Planning, Analysis and Design of Multistoreyed Apartment

Project Work 1988-89

P-37

Submitted by

Elangovan. P.
Murugesan. R.
Ramachandran. V.
Swaminathan. V.

Under the Guidance of

Prof. P. Jeyapaul, M.Sc., (STRUT)

Department of Civil Engineering
Kumaraguru College of Technology
Coimbatore - 641 006

Department of Civil Engineering
Kumaraguru College of Technology
Coimbatore - 641 006

Certificate

Name .... Roll. No. ....

This is the Bonafide Record of the Project titled

Planning, Analysis and Design of Multistoreyed Apartment

Done by

Mr. ....

In partial fulfilment of the requirements for the Degree of Bachelor of Engineering in Civil Engineering Branch of Bharathiar University, Coimbatore - 641 006 during the academic Year 1988-89

....

Head of the Department Staff in-charge

Submitted for the University Examination held on ....

University Register No. ....

....

Internal Examiner External Examiner

CONTENTS

PREFACE TO THE PROJECT WORK

In urban areas, multi-storeyed apartments are now in vogue due to increase in population and growth in different activities, vaccant land 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 practial application in the field.

Analysis and synthesis are higher level of learning according to blooms texanomy 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.

It is our duty to express our indebtedness to the authors of books on this subject to which references have been made.

Project members

P.ELANGOVAN, B.E.,

R.MURUGESAN,

V.RAMACHANDRAN,

V.SWAMINATHAN

ABSTRACT

This project work deals with planning, analysis, and design of a multistoreyed apartment building. It is a typical reinforced concrete framed structure for appartments with total no. of 20 flats i.e two separate five storeyed buildings with 2 flats in each storey.

Each flat of the left side building hav~~e~~ing 2 bed-rooms, one drawing cum dining room, one kitchen, one bathroom and one water closet.

Each flat of the right side building having one bed room, one living room, one store room, one kitchen room and one toilet.

Water tank is provided above the roof level of the left side building. Car parking is provided in the ground level of the right 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 limit state 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 members have been designed using limit state design procedure. High strength deformed bars have been used as reinforcement. Necessary architectural and structural drawings have been prepared.

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 sizes and their location

For the same 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 serves 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 no. of shelves should also be provided.

Bath and w.c

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 forrest room and should be fitted with shower, wash basin, shelves, brackets etc.

Staircases and Lifts

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 0.9m clear of railings and may range upto 1.5m. The stairs should be in two flights having a laading in the middle. As in this project, the number of stories are five, Lifts are also provided.

Water supply and Drainage

A common water tank is provided above the roof level of building. Water supply is given to each flat by proper network of pipelines. To receive the water from the corporation water supply a under ground water tank has been provided.

Adequatge 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 drinage 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 necessary holes, Silts etc, can be left in structural units. Recessed conduct type is suggested for wiring.

Flooring.

Nowadays it is advisable to use Mosaic tile floorings for better performance and appearance. The type, dimension and laying may be according to IS 809 - 1957.

Drawing

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

JOINERY DETAILS

a.	Bed room	.. 272 CM x 400 CM
b.	Drawing/Dining room	.. 272 CM x 455 CM
c.	Bed room	.. 300 CM x 280 CM
d.	W.C	.. 150 CM x 90 CM
e.	Bathroom	.. 190 CM x 110 CM
f.	Kitchen room	.. 300 CM x 200 CM
g.	Bed room	.. 272 CM x 317 CM
h.	Living room	.. 272 CM x 317 CM
i.	Toilet	.. 220 CM x 150 CM
J.	Store room	.. 232 CM x 272 CM
k.	Kitchen room	.. 232 CM x 190 CM
l.	Landing	.. 120 CM x 245 CM
m.	Landing	.. 210 CM x 245 CM

D_1	Door	.. 100 CM x 200 CM
D_2	Door	.. 90 CM x 200 CM
D_3	Door	.75 CM x 200 CM
W_1	Window	.. 150 CM x 130 CM
W_2	Window	.. 115 CM x 130 CM
W_3	Window	.. 90 CM x 130 CM
W_4	Window	.. 60 CM x 130 CM
W_5	Window	.. 120 CM x 75 CM
V	Ventilator	.. 90CM x 60 CM
O	Opening	.. 100 CM x 206 CM

DESIGN OF SLABS

Introduction

In our project the slab has been analysed by yield line theory and designed by Limit State Method. This method of analysis and design is more economical compared with the elastic methods of analysis and design.

Yield Moments

In this method the collapse moment is found by virtual work method. Then the required reinforcement Ast is found by moment of resistance of the section which is considered as under reinforced section.

$$Mu = 0.87\ fy\ Ast\ d\ (1 - \frac{Ast\ fy}{bdfck})$$

Notation for analysis

fy = Characteristic strength of reinforcement

fck = Characteristic compressive strength of concrete

b = Width of compressive face

D = total depth of slab

d = effective depth of slab

ϕ = angle defining the direction of yield line

α = Ratio of length and width of slab

m = Collapse moment of positive yield line / unit length m' = Collapse moment of negative yield line / unit length

μ = Ratio of moments in X and Y direction

Mu = Moment of resistance of a section without compression reinforcement

X_u = Depth of neutral axis.

Roof slab

(Continuous on two sides)

Size of slab = 3.40m x 2.9 m

Longer dimension Lx = 3.4 m

Shorter dimension Ly = 2.9 m

Thickness of slab continuous and spanning in two direction.

$$\frac{L}{40} = \frac{3.20 \times 100}{40} = 8 \text{ cm}$$

Assume thickness of slab as 10 cm

Design data

Mix used = M_{200}

Steel = TOR Steel

fck = 200 kg/cm^2

fy = 4150 kg/cm^2

Unit weight of concrete = 2400 kg/m^3

Loads

Live load = 150 kg/m^2

Self weight = 0.1 x 2400

= 240 kg/m^2

Weathering course = 100 kg/m^2

Total load = 490 kg/m^2

Limit load = 1.5 (D.L + L.L)

= 1.5 x 490

= 735 kg/m^2

Design

Assume m = m'

α = 1.76

μ = 0.919

$\tan \phi = \sqrt{3\mu + \frac{\mu^2}{\alpha^2}} - \frac{\mu}{\alpha}$

$= \sqrt{3 \times 0.919 + \left(\frac{0.919}{1.176}\right)^2} - \left(\frac{0.919}{1.176}\right)$

= 1.099

$M = \frac{W_u \tan^2\phi}{\mu \times 48} lu^2$

= 152.94 kg-m

$0.87\ fy\ Ast\ d\ (1 - \frac{Ast\ fy}{bd\ fck}) = Mu$

$0.87 \times 4150 \times Ast \times 8\ (1 - \frac{Ast \times 4150}{100 \times 8 \times 200})$

Ast = 152.94Kg.m

28884 Ast - 749.179Ast - 152.94×10^2 = 0

$Ast^2 - 38.55\ A\ t + 20.41 = 0$

Ast = 0.54 cm^2

Minimum % of steel = .12%

$= \frac{.12}{100} \times 100 \times 10$

= 1.2 cm^2

Reinforcement in shorter direction

Ast = 1.2 cm^2

Provide 8mm ϕ bars

Spacing = $\frac{100 \times \pi \times .8^2}{4 \times 1.2}$

= 41.8 cm

Maximum spacing 3d or 450mm whichever is less (ie) 3 x 8 = 24 or 45cm whichever is less.

Provide 8mm ϕ bars at 20 cm C/C.

Reinforcement in longer direction

Moment in longer direction = 0.919x152.94x100 kg-cm

= 14055.186 kg.cm

d = 8 - 0.8 = 7.2cm

Now,

$$.87 \times 4150 \times Ast \times 7.2 \left(1 - \frac{Ast \times 4150}{100 \times 7.2 \times 200}\right) = 14055.186$$

$$25995.6\ Ast - 749.179\ Ast^2 - 14055.186 = 0$$

$$Ast^2 - 34.69\ Ast + 18.76 = 0$$

$$Ast = \frac{34.69 \pm \sqrt{34.69^2 - 4 \times 18.76}}{2}$$

Ast = 0.549 cm^2

Minimum area of steel = $1.2\ cm^2$

Provide 8mm bars,

Spacing = $\frac{\pi \times 0.8^2}{4} \times \frac{100}{1.2}$

= 41.8 cm

Maximum spacing = 24 cm

Provide 8mm ϕ bars at 20cm C/C

The reinforcement details are given in fig.

Floor slab

Size of slab = 3.4 m x 2.9m

Thickness of slab continuous and spanning in two direction

$\frac{L}{40} = \frac{3.4 \times 1000}{40}$ = 8 cm

$\therefore$ Assume thickness of slab as 10 cm

Design data

Mix used = M_{200}

Steel = TOR steel

fck = $200\ kg/cm^2$

fy = $4150\ kg/cm^2$

Unit weight of concrete = $2400\ kg/m^3$

Load

Live load = $300\ kg/m^2$

Self weight = 0.1x1x2400

= $240\ kg/m^2$

Total load = 540 kg/m²

Limit load = 1.5 x 540

= 810 kg/m²

Design

Assume m = m'

α = 1.176

μ = 0.919

tan ϕ = 1.099

M = $\dfrac{W_u \tan^2\phi \; l_y^2}{48 \times \mu}$

M = $\dfrac{810 \times 1.099 \times 2.9^2}{0.919 \times 48}$

= 168.55 kg-m

Reinforcement in shorter direction

0.87 fy Ast d (1 - Ast fy/bd fck) = Mu

0.87x4150xAstx8(1-Astx4150/100x8x200)

= 168.55 x 100

28884 Ast - 749.149 Ast² - 168.55x100 = 0

Ast² - 38.55 Ast + 22.49 = 0

Ast = 0.59 cm²

Minimum steel = $\dfrac{0.12}{100}$ x 100 x 10 = 1.2 cm²

Provide 8mm ϕ bars at 20cm C/C.

Reinforcement in longer direction

Moment = 0.919x168.55x100

= 15489.745 kg cm

d = 7.2 cm

$$0.87 \times Ast \times 7.2 \left(1 - \frac{Ast \times 4150}{100 \times 8.2 \times 200}\right) = 15489.745 \text{ kg-cm}$$

$$25995.6\ Ast - 749.149\ Ast^2 - 15489.745 = 0$$

$$Ast^2 - 34.69\ Ast + 20.67 = 0$$

$$Ast = \frac{-34.69 + \sqrt{34.69^2 - 4 \times 20.67}}{2} =$$

$$= 0.61 \text{ cm}^2$$

Minimum area of steel = 1.2 cm^2

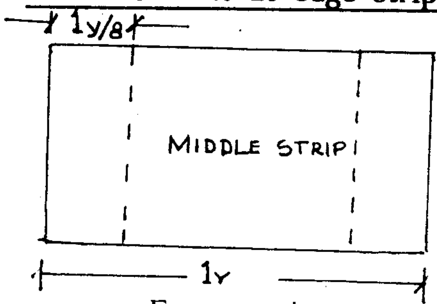
Hence provide 8mm Ø bars at 20 cm C/C

The design details of other slab are given below.

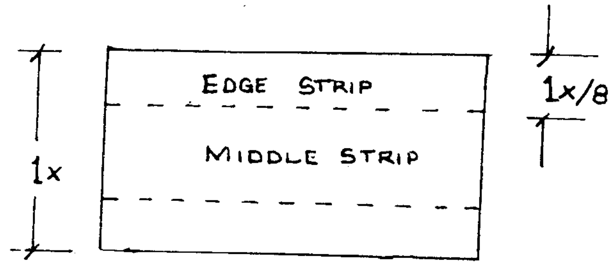
ROOF SLAB

S.No.	Size & slab	Live load	Dead load	Collapse load	Reinforcement in middle strip					
					Longer span			Shorter span		
	m	kg/m^2	kg/m^2	kg/m^2 1.5(L.L+D.L)	Moment kgm/m	Ast cm^2	spacing 8mmØ @	Moment kg-m/m	Ast cm^2	Spacing 8mm Ø @
1.	3.4 x 2.9	150	340	735	140.55	1.2	20cm	152	1.2	20cm
2.	2.6 x 1.7	150	340	735	46.94	1.2	20cm	59.65	1.2	20cm
3,	2.9 x 2.6	150	340	735	110.5	1.2	20cm	115.71	1.2	20cm
4.	2.6 x 2.2	150	340	735	75.278	1.2	20cm	86.23	1.2	20cm
5.	4.7 x 2.9	150	340	735	140.34	1.2	20cm	206.99	1.2	20cm
6.	3.25x 3	150	340	735	13.76	1.2	20cm	14.31	1.2	20cm
7.	2.5 x 2.1	150	340	735	69,21	1.2	20cm	73.42	1.2	20cm
8.	3.25x2.2	150	340	735	77.41	1.2	20cm	104.19	1.2	20cm

Reinforcement at edge strips



For span lx



for larger span ly

ly = 3.4m

lx = 2.89m

As per code IS - 456 -1983

Min. reinforcement required is 0.12% of total cross sectional area

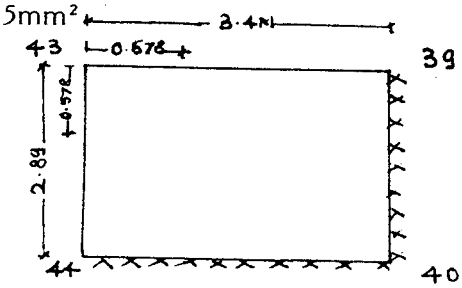
Min. steel area required in longer span : $\frac{0.12}{100} \times \frac{(3.4)}{8} \times 1000 \times 100$

$= 51 mm^2$

Provide 2 numbers of 8 mm Ø bars

Min. steel area required in shorter span $= \frac{0.12}{100} \times \frac{(2890)}{8} \times 100$

$= 43.35 mm^2$



Provide 2 nos of 8 mm Ø bars.

Torsion reinforcement at corners

At 43 $\frac{lx}{5} = \frac{2.89}{5} = 0.578m$

Torsion reinforcement at corners= $3/4 \times 1.2 = 0.9 \ Cm^2$

Provide 8mm Ø bars at a spacing of 50mm C/C in both directions at top and bottom.

At 39 Half the reinforcement as for corner A is to be provided provide 8 mm bars @ 100 mm C/C

S.No	Reinforcement in edge strip: Longer Span	Reinforcement in edge strip: Shorter span	Reinforcement at corners
1	2 Nos of 8 mm ∅/	2 Nos of 8 mm ∅	8 mm ∅ @ 50mm C/C at 43 & 100 mm C/C at 39 and 44
2	2 Nos of 8 mm ∅	2 Nos of 8 mm ∅	8 mm ∅ 50 mm C/C at 45 & 100 mm C/C at 42 & 44
3	2 Nos of 8 mm ∅	2 Nos of 8 mm ∅	8 mm∅ @ 100mm C/C at 42 & 38
4	2 Nos of 8 mm ∅	2 Nos of 8 mm ∅	8 mm ∅ @ 100 mm C/C at 38 & 26
5	2 Nos of 8 mm ∅	2 Nos of 8 mm ∅	8 mm ∅ @ 100 mm C/C at 30 & 12
6	2 Nos of 8 mm ∅	2 Nos of 8 mm ∅	8 mm ∅ @ 50 mm C/C at 36 & 100 mm at 35 & 33
7	2 Nos of 8 mm ∅	2 Nos of 8 mm ∅	8 mm ∅ @ 100 mm C/C at 33 & 29
8	2 Nos of 8 mm ∅	2 Nos of 8 mm ∅	8 mm ∅ @ 100 mm at 29 & 27

FLOOR SLAB

Sl No.	Size of Slabs m	Live load Kg/m^2	Dead load Kg/m^2	Collapse load 1.5 (D.L + L.L) Kg/m^2	Longer span			Shorter span		
					Moment Kg M/M	CM^2	Spacing 8mm ∅ @	Moment Kg M/M	CM^2	Spacing 8 mm ∅ @
1	3.4 x 2.9	300	240	810	158.75	1.2	20 CM	172.74	1.2	20 CM
2	2.6 x 1.7	300	240	810	51.66	1.2	20 CM	65.64	1.2	20 CM
3	2.9 x 2.6	300	240	810	121.71	1.2	20 CM	127.45	1.2	20 CM
4	2.6 x 2.2	300	240	810	82.96	1.2	20 CM	95.03	1.2	20 CM
5	4.77 x 2.9	300	240	810	154.67	1.2	20 CM	228.12	1.2	20 CM
6	3.25 x 3	300	240	810	151.46	1.2	20 CM	157.61	1.2	20 CM
7	2.5 x 2.1	300	240	810	76.27	1.2	20 CM	86.43	1.2	20 CM
8	3.25 x 2.2	300	240	810	85.31	1.2	20 CM	114.82	1.2	20 CM

ANALYSIS OF FRAMES:

Introduction:

Analysis of frames can be carried out by elastic and limit load analysis. Elastic analysis namely moment distribution method is used for the analysis by taking little portion of the frame calledSUBSTITUTING FRAME the moments can be calculated.

The analysis is carried out by the following steps.

(i) Preliminary dimension of beams

(ii) Preliminary dimension of columns.

(iii) Design moments for beams and columns by moment distribution method using substitute frame method.

Preliminary dimensions of beams;

To carry out the analysis of frame, the dimension of beams and columns are required, so that ;M.I, stiffness and distribution factors are calculated. So it is necessary to determine the size of the member first in analysis.

In building frames, beam size is usually governed by negative moments and shear, where the effective section is rectangular.

The total load on the beam is

(i) Self weight of beam

(ii) Load from slab

(iii) Load from walls supported by beam.

The following should kept in mind to get preliminary dimensions.

(i) Depth of beam is assumed as $\frac{L}{15}$ to $\frac{L}{20}$ of span

(b) Breadth of beam is assumed as 23 CM

(c) For loads, fixed end moment is calculated.

The M.R is calculated as fixed end moment divided by 1.5. From this moment the depth of beam is and can be compared to the assumed dimensions and beam depth is finalised.

Preliminary dimensions of Main Beams.

Roof beam (R_4 - 12) Support section

Assume 1 m (brick wall) height and thickness of parapet wall as 20CM

D.L of brick wall = 2000 x 1 x 0.2

= 400 Kg/m

Assume size of beam as 25 CM x 40 CM

Self weight of beam = 240 Kg/m

Total load = 640 Kg/m

Fixed end moment due to D.L $= \frac{640 \times 4.77^2}{12} = 1213.48$ Kg-m

F.E.M due to l.l $= \frac{W_1 a}{12L} (L^3 - a^2 (2L - a)]$

$= \frac{490 \times 1.47}{12 \times 4.77} (4.77^3 - 1.47^2(2 \times 4.77 - 1.47)$

= 1146.3 Kg. m

Total fixed end moment = 2359.78 Kg. m

$1.5 \times 9.11 \times 25d^2$ = 235978

$\therefore$ d = 26.28 CM

Take size of beam as 23 x 30 CM

S.F = 2714.89 Kg.

Floor beam

R_4 - 12 (Support section)

D.L of brick wall = 2000 x 2.8 x 0.2

= 1128 Kg/m

Assume beam size as = 25 x 30 CM

D.L of beam = 0.25 x 0.3 x 2400

= 180 Kg/m

Total D.L = 1308 Kg/m

F.E.M due to D.L = $\frac{1308 \times 4.77^2}{12}$ = 2480 Kg m

F.E.M due to L.L = $\frac{640 \times 1.47}{12 \times 4.77}(4.77^3-1.47^2(2\times4.77-1.47)$

= 1497.19 Kg-m

Total F.E.M = 3977.29 Kg-m

$1.5 \times 9.11 \times 25 \times d^2$ = 397729

d = 34.12CM

Take beam size as 23 CM x 40 CM

Shear force = 3981 Kg

The other main beams are similarly analysed and preliminary dimensions are tabulated.

TABLE

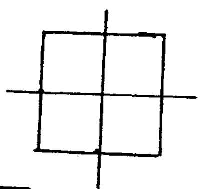
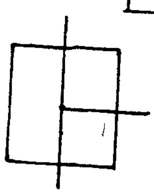
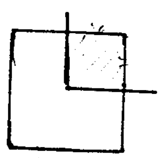
Beam No.	Span m	Assumed size Breadth x depth. CM x CM	Load intensity for Triangular Kg/m	Load intensity for Trapezoidal Kg/m	UDL + Sl. Wt. + walls.	Concentraled load Kg.	Total F.E.M	Depth CM	Size of beam CM x CM
1	2	3	4	5	6	7	8	9	10
ROOF LEVEL MAIN BEAM									
R_1-4	4.24	23 x 40	--	720.3	640	--	2359.78	26.78	23 x 30
R_4-12	4.77	23 x 40	--	720.3	640	--	2359.78	26.78	23 x 30
R_1 2	2.94	23 x 30	720.3	--	580	--	742.07	14.74	23 x 20
R_4-5	2.94	23 x 30	1440.6	--	180	--	778.2	15.08	23 x 20
R_2-3	3.25	23 x 20	--	739.9	520	--	591.05	12.58	23 x 20
R_3-7	3.02	23 x 30	739.9	--	510.4	--	739.4	15.4	23 x 20
R_7-11	4.65	23 x 30	--	514.5	580	2718.12	3650.0	28.38	23 x 30
R_{10}-11	3.25	23 x 30	--	534.1	180	1253.34	1835.05	24.16	23 x 30
R_2-5	4.24	23 x 35	739.9	720.3	210	2718.12	3239.8	29.8	23 x 30
R_5-10	3.45	23 x 35	739.9	720.3	210	2718.12	3239.8	29.8	23 x 30
R_{17}-26	6.8	23 x 40	--	396.9	640	1687.82	5075.9	38.5	23 x 40
R_{15}-17	5.2	23 x 40	--	396.9	640	4671.66	6610.4	43.9	23 x 45
R_{15}-24	6.8	23 x 40	--	396.9	640	1687.82	5075.9	38.5	23 x 40

1	2	3	4	5	6	7	8	9	10
R_{24-26}	5.2	23 x 50	--	1068.2	276	9344.0	10317.85	51.5	23 x 55
R_{10-14}	2.64	23 x 35	--	--	410	1222.0	751.56	14.8	23 x 20
FLOOR LEVEL MAIN BEAM									
R									
1-4	4.24	23 x 30	--	940.8	1308	--	3977.29	34.12	23 x 35
R_{4-12}	4.77	23 x 30	--	940.8	1308	--	3977.29	34.12	23 x 35
R_{1-2}	2.94	23 x 30	940.8	--	1308	--	1368.69	20.0	23 x 20
R_{4-5}	2.94	23 x 25	1881.6	--	1278	--	1767.56	22.75	23 x 25
R_{2-3}	3.25	23 x 30	--	966.4	1308	--	1719.98	22.4	23 x 25
R_{3-7}	3.02	23 x 30	966.4	--	1308	--	1453.23	20.62	23 x 20
R_{7-11}	4.65	23 x 30	--	672	1308	3426.14	5565.58	40.36	23 x 45
R_{10-11}	3.25	23 x 30	--	697.6	1293.6	1253.34	2240.73	26.7	23 x 30
R_{2-5}	4.24	23 x 40	940.8	966.4	1368	3426.14	6204.24	42.62	23 x 45
R_{5-10}	4.24	23 x 40	940.8	966.4	1368	3426.14	6204.24	42.62	23 x 45
R_{17-26}	6.8	23 x 55	--	518.4	1488	4632.86	10812.67	52.26	23 x 55
R_{15-17}	5.2	23 x 55	--	518.4	1428	11864	12798.7	54.5	23 x 55
R_{15-24}	6.8	23 x 55	--	518.4	1488	4632.86	10812.67	51.2	23 x 50
R_{24-26}	5.2	23 x 60	--	697.6	1688	23728	23582.2	62.4	23 x 60
R_{10-14}	2.64	23 x 23	--	--	615.96	3243	1427.9	21.3	23 x 30
R_{11-27}	2.64	23 x 23	--	627	1353	--	1072.64	18.47	23 x 20

Preliminary Dimensions for Columns:

Columns are mostly subjected to axial loads and biaxial moments. Initially the B.M is unknown. So that axial load coming on the column at a particular section is multiplied by a factor depending upon its position. The factor is selected from the table given below. The equivalent direct load obtained after multiplying the axial load by a suitable factor is divided by allowable direct compressive stress to obtain the C.S area from which size of the column can be fixed.

TABLE:

Type of column	Top	Top but one	Lower
	1.0	1.0	1.0
	4.5	2.0	1.4
	6.6	2.3	1.8

Column C_1 (A level)

Load from the beam R_{1-4} = 2715 Kg

R_{1-2} = 1382 Kg

Total Load = 4097 Kg

Self weight of column (Assume 10%) = 409.7 Kg

4506.7 Kg

Equivalent axial load = 6 x 4506.7

= 27040.2 Kg.

Allowable compressive stress for M_{200} = 50 Kg/Cm²

$\therefore$ Area of column required = $\frac{27040.2}{50}$ = 540.8 Cm²

Adopt column size as 23 x 25 CM

B level

Load from beam R_{1-4} = 3981 Kg

R_{1-2} = 2615 Kg

Total load = 6596 + 4097

= 10693 Kg

Self wt. of column = 1069.3 Kg.

Total axial load = 11762.3 Kg

Equivalent axial load = 11762.3 x 2.3

= 27053.29 Kg

Area of column required = $\frac{27053.29}{50}$ Kg = 541.12 Cm²

Adopt column size as 23 x 25 CM

C Level

Load from beam R_{1-4} = 3981 Kg

R_{1-2} = 2615 Kg

Total load = (3981 + 2615) x 2 + 4097 = 17289 Kg

Self weight of column = 1728.9 Kg.

19017.9 Kg

Area of column required = $\frac{19017.9}{50}$ = 684.64 CM²

Adopt size of column as 23 cm x 32 cm

D Level

Load from beam R_{1-4} = 3981 Kg

R_{1-2} = 2615 Kg

Total load = (3981 + 2615) 3 + 4097 = 23885 Kg

Self weight of column = 2388.5 Kg.

26273.5 Kg

Area of column required = $\frac{26273.5 \times 1.8}{50}$ = 945.84 Cm^2

Adopt size of column 23 x 45 Cm

E Level

Load from bram R_{1-4} = 3981 Kg

R_{1-2} = 2615 Kg

Total load = (3981 + 2615) 4 + 4097 = 30481 Kg

Self weight of column = 3048.1 Kg

40125.1 Kg

Area of column required = $\frac{40125.1 \times 1.8}{50}$ = 1444.5 CM^2

Adopt size of column as 30 x 50 CM

Similarly all other columns are analyse and priliminary dimension are tabulated.

TABLE

Column No.	Level	Load from Kg beams Kg	Self weight 10% Kg	Total load above level Kg	Total Load Kg	L.F	Eq. axial load Kg.	Area of column Cm²	Calculated size of column cm x cm	Provided size of column cm x cm
1	2	3	4	5	6	7	8	9	10	11
C_1	A-level	4097	409.7	--	4506.7	6	27040.2	540.8	23 x 25	
	B-level	6596	659.6	4506.7	11762.3	2.3	27053.29	541.1	23 x 25	
	C-level	6596	659.6	11762.3	19017.9	1.8	34232.2	684.64	23 x 32	23 x 40
	D-level	6596	659.6	19017.9	26273.5	1.8	47292.3	945.8	23 x 45	
	E-level	6596	659.6	26273.5	40125.1	1.8	72225.2	1444.5	30 x 50	
C_2	A-level	5099	509.9	--	5608.9	4.5	25240.05	504.8	23 x 23	
	B-level	10393	1039.3	5608.9	17041.2	2.0	34082.4	681.6	23 x 32	23 x 40
	C-level	10393	1039.3	17041.2	28473.5	1.4	39862.9	797.3	23 x 37	
	D-level	10393	1039.3	28473.5	39905.8	1.4	55868.12	1117.0	23 x 48	
	E-level	10393	1039.3	39905.8	58103.3	1.4	81344.62	1626.9	30 x 55	
C_3	A-level	2819	281.9	--	3100.9	6.0	18605.4	372	23 x 20	
	B-level	5672	567.2	3100.9	9329.1	2.3	21456.9	429	23 x 23	23 x 30
	C-level	5672	567.2	9329.1	15579.3	1.8	28042.7	560.8	23 x 27	

1	2	3	4	5	6	7	8	9	10	11
	D-level	5672	567.2	15579.3	21818.5	1.8	39273.3	785	23 x 37	
	E-level	5672	567.2	21818.5	31746.2	1.8	57143.6	1143	23 x 50	
C_4	A-level	6754	6754	--	7429.4	4.5	33432.3	668.6	23 x 32	
	B-level	11224	1122.4	7429.4	19778.8	2	39557.6	791	23 x 37	23 x 40
	C-level	11224	1122.4	19778.8	32122.2	1.4	44971.1	899	23 x 42	
	D-level	11224	1122.4	32122.2	44468.6	1.4		1245	23 x 54	
	E-level	11224	1122.4	44468.6	63996.4	1.4	89594.96	1791.9	35 x 50	
C_5	A-level	5780	578	--	6358	4.5	28611	572	23 x 27	
	B-level	12884	1288.4	6358	20530.4	2.0	41060.8	821	23 x 38	23 x 45
	C-level	12884	1288.4	20530.4	34702.8	1.4	48583.9	792	23 x 45	
	D-level	12884	1288.4	34702.8	48875.2	1.4	68425.3	1368	23 x 59	
	E-level	12884	1288.4	48875.2	71488.4	1.4	100083.76	2001.68	35 x 60	
C_6	A-level	7413	741.3	--	8154.3	4.5	36694.4	733	23 x 34	
	B-level	10383	1038.3	8154.3	19575.6	2.0	39151.2	783	23 x 36	23 x 40
	C-level	10383	1038.3	19575.6	30996.9	1.4	43395.7	867	23 x 40	
	D-level	10383	1038.3	30996.9	42418.2	1.4	59385.4	1087	23 x 47	
	E-level	10383	1038.3	42418.2	60366	1.4	84512.8	1690	30 x 55	

1	2	3	4	5	6	7	8	9	10	11
C_7	A-level	7334	733.4	8448	17360.2	4.5	78120.9	1562	30 x 52	
	B-level	10950	1095.0	17360.2	29405.2	2.0	58910.4	1176	30 x 40	
	C-level	10950	1095.0	29405.2	41450.2	1.4	58030.28	1160.6	30 x 39	
	D-level	10950	1095.0	41450.2	59495.2	1.4	83293.3	1498	30 x 50	
	E-level	10950	1095.0	59495.2	63179	1.4	88450.6	1769	35 x 55	
C_8	A-level	9578	957.8	8448	17966	1	17966	395.3	25 x 20	
	B-level	11114	1111.4	17966	31988	1	31988	639	25 x 30	23 x 50
	C-level	11114	1111.4	31988	44213.4	1	44213.4	884.4	25 x 40	
	D-level	11114	1111.4	44213.4	56438.8	1	56438.0	1128.7	25 x 50	
	E-level	11114	1111.4	56438.8	66259.4	1	66259.4	1325	25 x 55	
C_9	A-level	6652	665.2	--	7317.2	4.5	32927.4	658	23 x 32	
	B-level	11205	1120.5	7317.2	19642.7	2	39285.4	785	23 x 37	
	C-level	11205	1120.5	19642.7	31968.2	1.4	44785.5	895	23 x 42	23 x 45
	D-level	11205	1120.5	31968.2	44293.7	1.4	62011.2	1240	23 x 54	
	E-level	11205	1120.5	44293.7	63797.5	1.4	89316.5	1786.33	35 x 50	
C_{10}	A-level	9253	925.3	--	10178.3	6	61069.8	1221	40 x 40 x 23	
	B-level	16229	1622.9	10178.3	28107.2	2.3	64646.6	1292	42 x 42 x 23	

1	2	3	4	5	6	7	8	9	10	11
	C-level	16229	1622.9	28107.2	45882.1	1.8	82587.8	1653	50 x 50 x 23	50 x 50 x 23
	D-level	16229	1622.9	45882.1	63734	1.8	114721.2	2293	62 x 62 x 23	
	E-level	16229	1622.9	63734	92020.9	1.8	165637.62	3312	40 x 60 x 30	
C_{11}	A-level	9253	925.3	--	10178.3	6	61069.8	1221	40 x 40 x 23	
	B-level	16229	1622.9	10178.3	28107.2	2.3	64646.6	1292	42 x 42 x 23	50 x 50 x 23
	C-level	16229	1622.9	28107.2	45882.1	1.8	82587.8	1653	50 x 50 x 23	
	D-level	16229	1622.9	45882.1	63734	1.8	114721.2	2293	62 x 62 x 23	
	E-level	16229	1622.9	63734	966209	1.8	173917.62	3478	55 x 60 x 30	
C_{12}	A-level	17918.4	1791.84	--	19703.64	4.5	88666.4	1773.3	40 x 30 x 20	
	B-level	39280.4	3928.04	19703.64	62918.2	2	125837.8	2516	50 x 34 x 30	55 x 35 x 30
	C-level	39280.4	3928.04	62918.2	106128.44	1.4	148579.8	2971	55 x 35 x 30	
	D-level	39280.4	3928.04	106128.44	149337.7	1.4	209071.9	4182	55 x 35 x 30	
	E-level	39280.4	3928.04	149337.7	218248.4	1.4	305547.8	6110	70 x 70 x 40	
C_{13}	A-level	16158	1615.8	--	17773.8	6	106642.8	2133	45 x 30 x 30	
	B-level	37526	3752.6	17773.8	59052.4	2.3	135020	2716	45 x 35 x 30	55 x 35 x 30
	C-level	37526	3752.6	59052.4	100331	1.8	180595.8	3212	40 x 60 x 30	
	D-level	37526	3752.6	100331	141609.6	1.8	252897.3	4298	70 x 50 x 30	
	E-level	37526	3752.6	141609.6	202856.2	1.8	365141.16	7302	90 x 70 x 30	

Calculation of moment of Intertia for the section:

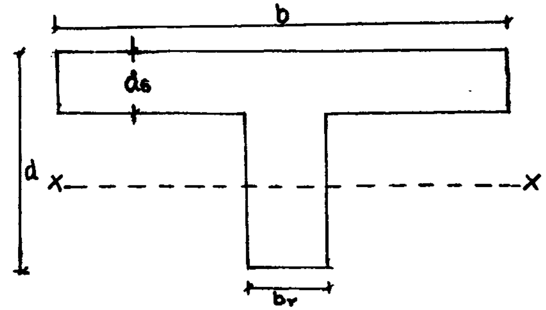
Introduction:

The moment of inertia is calculated for intermediate beams and end beams cast monolithically with the slab as T and L section respectively. In the case of lintal and plinth beam the M.I is calculated treating as rectangular sections.

For T and L beam the M.I is calculated using the table given in "Reinforced concrete Designs Hand book" by **Chas. E.REYNOLDS.**

For columns the M.I is calculated treating them as rectangular sections. Only gross section is considered for both beams and columns.

REYNOLD'S TABLE:



Moment of intertia of tee section

(Also applicable to ELL section and inverted channel)

Moment of inertia about x - x Axis

Through centroid $I_{xx} = C\ b_r d^3$

Values of C are tabulated

Ratio $\frac{Ds}{d}$	Ratio $\frac{br}{bs}$ 0.12	0.14	0.16	0.18	0.20	0.22	0.24	0.26	0.28	0.30	0.35	0.40	0.45	0.50
0.10	0.17	0.161	0.154	0.147	0.142	0.137	0.133	0.129	0.125	0.122	0.115	0.11	0.106	0.102
0.15	0.18	0.172	0.165	0.158	0.152	0.146	0.143	0.138	0.135	0.131	0.121	0.117	0.111	0.107
0.20	0.183	0.175	0.169	0.163	0.157	0.152	0.147	0.143	0.14	0.137	0.128	0.121	0.116	0.111
0.30	0.184	0.176	0.170	0.164	0.159	0.155	0.150	0.146	0.142	0.139	0.132	0.125	0.119	0.114
0.40	0.19	0.18	0.173	0.165	0.16	0.155	0.151	0.146	0.142	0.139	0.132	0.125	0.119	0.115
0.45	0.197	0.185	0.176	0.169	0.182	0.156	0.152	0.147	0.143	0.14	0.132	0.125	0.119	0.115
0.50	0.208	0.195	0.183	0.174	0.167	0.160	0.155	0.15	0.146	0.141	0.133	0.126	0.12	0.115

Calculation of moment of inertia for sections:

Beam $A_1 A_4$ (L Section)

Span = 4.24 m

Cross section = $23 \times 30\ Cm^2$

Flange width = $\frac{lo}{12} + 6W + 3\ Df$

$= \frac{0.7 \times 424}{12} + 23 + 3 \times 10 = 77.73Cm$

$\frac{ds}{d} = \frac{10}{40} = 0.25 \quad \frac{br}{bs} = \frac{23}{77.73} = 0.295$

C = 0.138

M.Ixx = $C\ br\ d^3 = 0.138 \times 23 \times 40^3 = 203136\ CM^4$

Beam $A_{24} A_{26}$ (T Section)

Span = 5.2 m

Cross section = 23 cm x 55 cm

Flange width = $\frac{lo}{6} + bw + 6\ Df$

$= \frac{0.7 \times 520}{6} + 23 + 6 \times 10 = 143.7Cm$

$\frac{ds}{d} = \frac{10}{65} = 0.154 \quad \frac{Br}{bs} = \frac{23}{143.7} = 0.16$

C = 0.165

MIxx = $0.165 \times 23 \times 65^3$

$= 1042201.8\ cm^4$

Column $A_1 B_1$ (Rectangular section)

Cross section = 23 cm x 40 cm

$$MI_{xx} = \frac{23 \times 40^3}{12} = 122666.67 \text{ cm}^4$$

Similarly for all the beams and columns the moment of intertia is calculated.

Calculation of distribution factors

The distribution factors are calculated in two perpendicular directions seperately as the moments are calculated.

The calculation of distribution factor is as follows. The M.I of the beam is divided by its length or span L to get the value of K (Relative stiffness). Similarly for all the section meeting at a joint is calculated. Then the distribution factor for a member is $K/\Sigma K$. Similarly all the distribution factors are calculated.

Dead load Analysis

As the magnitude and position for dead loads such as the self weight of the slab beam etc. are almost permanent and not varying, the analysis of the entire frame is done keeping all the loads using moment distribution with single cycle.

Calculation of FEM and simply supported bending moment at the mid span

Frame 1

Beam A_1 A_4

(i) Fixed end moment

Trapezoidal load from the slab $= \frac{340 \times 2.94}{2} = 499.8$ Kg/m

FEM $= \frac{WL^2 \ (1 - 2a^2 + a^3)}{12}$

$= \frac{499.8 \times 4.24^2}{12} (1 - 2 \times 0.574^2 + 0.574^3)$

$= 396.68$ Kg m

a. $= \frac{\mu \kappa^2}{2} \sqrt{\frac{3}{\kappa^2 \mu} + 1} - 1$

a $= \frac{1.664^2 \times 0.678}{2} \times \sqrt{\frac{3}{1.678 \times 1.664^2} + 1} - 1$

a $= 0.574$

Self weight of beam $= 165.6$ Kg/m

Parapet weight $= 400.$ Kg/m

565.6 Kg/m

FEM $= \frac{WL^2}{12} = \frac{565.6 \times 4.24^2}{12} = 847.344$ Kg -m

Total FEM $= 396.683 + 847.344$

$= 1244.027$ Kg-m

(ii) Simply supported at the mid span

B.M at the mid span due to traperiodal load $= \frac{WL^2}{24} (3 - 4a^2)$

$= \frac{499.8 \times .24^2}{24} (3 - 4 \times 0.574^2)$

$= 629.23$ Kg. m

B.M at the mid span due to

self weight + parapet weight $= \frac{WL^2}{8} = \frac{565.6 \times 4.24^2}{8}$

$= 1271.02$ Kg - M

Total simply supported BM at the

mid span $= 1900.25$ Kg M

Beam $A_1 A_2$

(i) Fixed end moment

Triangular load from slab $= 340 \times \frac{2.94}{2}$

$= 499.8$ Kg/m

FEM $= \frac{5}{96} WL^2 = \frac{5}{96} \times 499.8 \times 2.94^2$

$= 225.004$ Kg.m

Self weight of beam $= 110.4$ Kg/m

Parapet wt $= 400$ Kg/m

510.4 Kg/m

$\therefore$ FEM $= \frac{WL^2}{12} = \frac{510.4 \times 2.94^2}{12}$

$= 367.64$ Kg. m

Total FEM $= 592.645$ Kg. m

(ii) Simply supported B.M at the mid span

B.M at the mid span due to triangular load $= \frac{WL^2}{12} = \frac{499.8 \times 2.94^2}{12}$

$= 360.006$ Kg.m

B.M at the mid span due to self wt + parapet wt. $= \frac{510.4 \times 2.94^2}{8}$

$= 551.46$ Kg.m

Total B.M at the Mid span = 911.47 Kg. m

Similarly other fixed end moments and B.M at the centre for simply supported conditions are calculated and given in the table.

FRAME: 1 **TABLE:**

Joint	Member	D.F	F.E.M Kg.m	Distribution	Final moment Kg.m
A_1	$A_1 A_4$	0.52	1244.027	-646.89	597.132
A_4	$A_4 A_1$	0.36	-1244.027	-119.176	-1363.203
	$A_4 A_{12}$	0.32	1575.073	-105.934	1469.139
A_{12}	$A_{12} A_4$	0.29	-1575.073	0	-1575.073
	$A_{12} A_{30}$	0.29	1575.073	0	157.073
A_{30}	$A_{30} A_{12}$	0.32	-1575.073	105.934	-1469.139
	$A_{30} A_{34}$	0.36	1244.027	119.176	1363.203
A_{34}	$A_{34} A_{30}$	0.52	-1244.027	646.89	-597.132
B_1	$B_1 B_4$	0.44	2376.017	-1045.447	1130.569
B_4	$B_4 B_1$	0.31	-2376.017	-195.87	-2571.89
	$B_4 B_{12}$	0.28	3007.88	-176.92	2830.95
B_{12}	$B_{12} B_4$	0.25	-3007.88	0	-3007.88
	$B_{12} B_{30}$	0.25	3007.88	0	3007.88
B_{30}	$B_{30} B_{12}$	0.28	-3007.88	176.92	-2830.95
	$B_{30} B_{34}$	0.31	2376.017	195.87	2571.89
B_{34}	$B_{34} B_{30}$	0.44	-2376.017	1045.477	-1130.569
D_1	$D_1 D_4$	0.48	2376.017	-1140.49	1235.53
D_4	$D_4 D_1$	0.33	-2376.017	-208.51	-2584.531
	$D_4 D_{12}$	0.30	3007.88	-189.558	2818.321
D_{12}	$D_{12} D_4$	0.27	-3007.88	0	-3007.88
	$D_{12} D_{30}$	0.27	3007.88	0	3007.88
D_{30}	$D_{30} D_{12}$	0.30	-3007.88	189.558	-2896.321
	$D_{30} D_{34}$	0.33	2376.017	208.51	2584.531
D_{34}	$D_{34} D_{30}$	0.48	-2376.017	110.49	-1235.53

FRAME 2

A_3	$A_3 A_2$	0.71	723.917	-513.98	209.935
A_2	$A_2 A_3$	0.24	-723.917	31 505	-692.142
	$A_2 A_1$	0.63	592.645	82.70	675.346
A_1	$A_1 A_2$	0.82	-592.645	485.968	-106.676
B_3	$B_3 B_2$	0.66	1389.007	-196.74	472.26
B_2	$B_2 B_3$	0.43	-1389.007	116.95	-1272.05
	$B_2 B_1$	0.29	1117.014	78.876	1195.890
B_1	$B_1 B_2$	0.50	-1117.014	558.507	-558.507
D_3	$D_3 D_2$	0.70	1389.007	-972.30	416.70
D_2	$D_2 D_3$	0.44	-1389.007	119.764	-1269.326
	$D_2 D_1$	0.30	1117.014	81.596	1198.61
D	$D_1 D_2$	0.54	-1117.014	603.1875	-513.826

Frame 3

A_2	$A_2 A_3$	0.57	2426.25	-1382.96	1043.287
A_5	$A_5 A_2$	0.31	-2426.25	190.77	-2235.479
	$A_5 A_{10}$	0.36	1810.85	221.540	2032.390
A_{10}	$A_{10} A_5$	0.36	-1810.85	496.728	-1318.7274
	$A_{10} A_{14}$	0.20	432.715	275.627	708.342
A_{14}	$A_{14} A_{10}$	0.20	-432.715	-275.627	-708.342
	$A_{14} A_{31}$	0.36	1810.85	-496.128	1318.7274
A_{31}	$A_{31} A_{14}$	0.36	-1810.85	221.540	-2032.390
	$A_{31} A_{35}$	0.31	2426.25	-190.77	2235.479
A_{35}	$A_{35} A_{31}$	0.57	-2426.25	1382.96	-1043.287

B_2	$B_2 B_5$	0.62	4378.43	-2714.63	1663.803
B_5	$B_5 B_2$	0.33	-4378.43	413.457	-3964.973
	$B_5 B_{10}$	0.39	3125.53	488.631	3614.161
B_{10}	$B_{10} B_5$	0.43	-3125.53	786.226	-2339.30
	$B_{10} B_{14}$	0.10	1297.096	182.84	1479.94
B_{14}	$B_{14} B_{10}$	0.10	-1297.096	-182.84	-1479.939
	$B_{14} B_{31}$	0.46	3125.53	-841.079	2284.45
B_{31}	$B_{31} B_{14}$	0.39	-3125.53	-486.631	-3614.161
	$B_{31} B_{35}$	0.33	4378.43	-413.45	3964.973
B_{35}	$B_{35} B_{31}$	0.62	-4378.43	2714.63	-1663.803
D_2	$D_2 D_5$	0.66	4378.43	-2889.76	1488.67
D_5	$D_5 D_2$	0.34	-4378.43	425.986	-1352.444
	$D_5 D_{10}$	0.41	3125.53	513.689	3639.219
D_{10}	$D_{10} D_5$	0.40	-3125.53	731.373	-2394.157
	$D_{10} D_{14}$	0.10	1297.096	182.843	1480.374
D_{14}	$D_{14} D_{10}$	0.10	-1297.096	-182.843	-1479.924
	D14 D_{31}	0.49	3125.53	-895.88	2229.646
D_{31}	$D_{31} D_{14}$	0.41	-3125.53	-513.689	-3639.219
	$D_{31} D_{35}$	0.34	4378.43	-425.986	3953.24
D_{35}	D_{33} D_{31}	0.66	-4378.43	2889.76	-1488.67

FRAME 4

A_{17}	$A_{17}A_{15}$	0.46	3398.36	-1563.24	1835.115
A_{15}	$A_{15}A_{17}$	0.46	-3266.8	1502.73	-1764.072
B_{17}	$B_{17}B_{15}$	0.42	5480.88	-2301.96	3178.910
B_{15}	$B_{15}B_{17}$	0.42	-5338.24	2242.06	-3096.18
E_{17}	$E_{17}E_{15}$	0.40	5480.88	-2192.352	3288.528

FRAME 5

A_{15}	$A_{15}A_{24}$	0.34	4249.839	-1444.945	2804.894
A_{24}	$A_{24}A_{15}$	0.205	-4249.839	0	-4249.839
	$A_{24}A_{43}$	0.205	4249.839	0	4249.839
A_{43}	$A_{43}A_{24}$	0.34	-4249.839	1444.945	-2804.894
B_{15}	$B_{15}B_{24}$	0.30	8687.136	-2606.140	6080.995
B_{24}	$B_{24}B_{15}$	0.19	-8687.136	0	-8687.136
	$B_{24}B_{43}$	0.19	+8687.136	0	8687.136
B_{43}	$B_{43}B_{24}$	0.30	-8687.136	2606.1408	-6080.995

FRAME: 6

A_{26}	$A_{26}A_{24}$	0.38	5922.284	-2250.467	3671.816
A_{24}	$A_{24}A_{26}$	0.30	4509.98	-1713.77	2796.19
B_{26}	$B_{26}B_{24}$	0.27	13351.609	-3604.934	9746.675
B_{24}	$B_{24}B_{26}$	0.27	12538.47	-3385.38	9153.08
E_{26}	$E_{26}E_{24}$	0.26	13351.609	3471.41	9880.190

Live load analysis:

-Using Substitute Frames

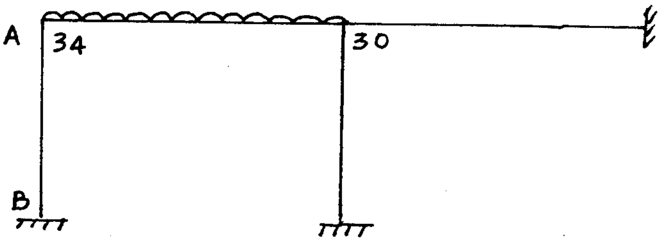
The live loads are of different magnitude and of different position for computing the bending moments at critical sections. The structure being indeterminate and exact analysis becomes tedious. In fact, the assumptions involved even in exact analysis leads to considerable error and this approximate method of analysis are reasonably accurate for practical purpose.

Theortically, a load applied at any point of the structure should cause forces and moments at all sections of frame, but a close study of this aspect has shown that the moment in any beam or columns are mainly due to the loads on span very close to it. The effect of loads on distant pannel is small. To facilitate the determination of moments in any member of a frame, it is usual to analyse only a small portion of frame consisting of the adjacent member only. Such small portion are termed as "substitute frames".Their form and the end conditions of their member are so adopted that the forces in question works out to be the greatest.

Fixed end moments and bending moment at the centre for simply supported conditions are calculated. The substitute frames for the critical beam and column section are choosen other substitute frames are analysed and the final moments are given in the table.

Substitute frame method

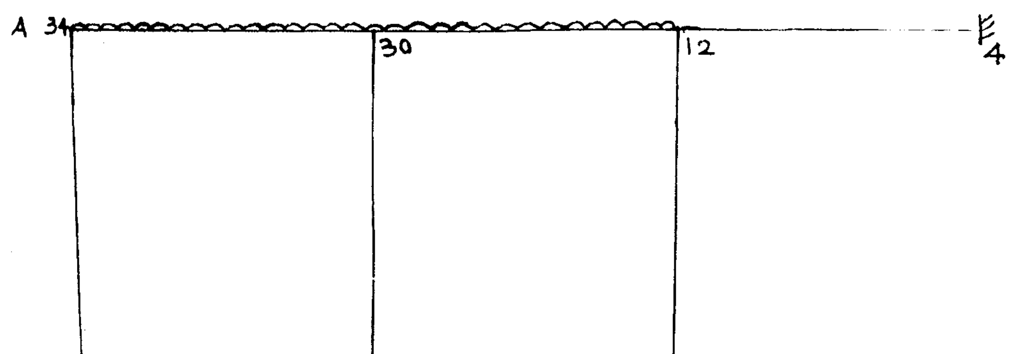
Case 1



Joint	A_{34}		A_{30}		
Member	$A_{34}B_{34}$	$A_{34}A_{30}$	$A_{30}A_{34}$	$A_{30}B_{30}$	$A_{30}A_{12}$
D.F	0.48	0.52	0.36	0.32	0.32
F.E.M	--	-264.69	264.69	--	--
Bal	127.051	137.639	-95.288	-84.701	-84.701
C/o	--	-47.644	68.82	--	--
Bal	22.869	24.775	-24.775	-22.022	-22.022
C/o	--	-12.388	-12.388	--	--
Bal	5.946	6.442	-4.460	-3.964	-3.964
C/o	--	-2.230	3.221	--	--
T.M	155.27	-157.866	221.375	-110.687	-110.687

Case ii

Joint	A_{34}		A_{30}			A_{12}		
Member	$A_{34}B_{34}$	$A_{34}A_{30}$	$A_{30}A_{34}$	$A_{30}B_{30}$	$A_{30}A_{12}$	$A_{12}A_{30}$	$A_{12}B_{12}$	$A_{12}A_{4}$
D.F	0.48	0.52	0.36	0.32	0.32	0.29	0.42	0.29
F.E.M	--	-264.69	264.69	--	-350.91	350.91	--	--
Bal	127.051	137.639	31.039	27.59	27.59	-101.764	-147.382	-101.764
C/o	--	15.520	68.820	--	50.882	13.795	--	--
Bal	-7.450	-8.070	-6.458	-5.740	-5.740	-4.001	-5.794	-4.001
C/o	--	-3.229	-4.035	--	-2.001	-2.870	--	--
Bal	1.550	1.679	2.173	1.932	1.932	0.832	1.205	0.832
C/o	--	1.087	0.840	--	0.416	0.966	--	--
F.M	121.151	-121.15	356.69	23.78	-378.59	256.86	-151.97	-104.93



TABLE

Substitute frame No	Joint	Member	F.E.M Kg.m.	D.F.	Final moment kg.m.
1	2	3	4	5	6
Frame :1.					
1.	A_{34}	$A_{34}A_{30}$	-264.69	0.52	-157.87
		$A_{34}B_{34}$	264.69	0.48	155.27
	A_{30}	$A_{30}A_{34}$	264.69	0.36	221.37
		$A_{30}B_{30}$		0.32	-110.685
		$A_{30}A_{12}$		0.32	-110.685
2.	A_{34}	$A_{34}A_{30}$	-264.69	0.52	-121.15
		$A_{34}B_{34}$		0.48	121.15
	A_{30}	$A_{30}B_{30}$		0.32	23.78
		$A_{30}A_{12}$	-350.91	0.32	-380.0
		$A_{30}A_{34}$	264.69	0.36	356.22
	A_{12}	$A_{12}A_{30}$	350.91	0.29	256.86
		$A_{12}B_{12}$		0.42	151.97
		$A_{12}A_{14}$		0.29	-104.92
3.	A_{30}	$A_{30}A_{34}$		0.36	126.33
		$A_{30}B_{30}$		0.32	112.29

		$A_{30}A_{12}$	-350.91	0.32	-238.62
	A_{12}	$A_{12}B_{30}$	350.91	0.29	407.06
		$A_{12}B_{12}$		0.42	0
		$A_{12}A_{4}$	-350.91	0.29	-407.06
	A_{4}	$A_{4}A_{12}$	+350.91	0.32	238.62
		$A_{4}B_{4}$		0.32	-112.29
		$A_{4}A_{1}$		0.36	-126.33
4.	A_{34}	$A_{34}B_{34}$	-264.69	0.48	163.09
		$A_{34}A_{30}$		0.52	-163.09
	A_{30}	$A_{30}A_{34}$	264.69	0.36	201.9
		$A_{30}B_{30}$		0.32	-133.58
		$A_{30}A_{12}$		0.32	-68.31
	A_{12}	$A_{12}A_{30}$		0.29	76.02
		$A_{12}B_{12}$		0.42	200.64
		$A_{12}A_{4}$	-350.91	0.29	-276.66
	A_{4}	$A_{4}A_{12}$	350.91	0.32	283.01
		$A_{4}B_{4}$		0.32	-133.05
		$A_{4}A_{1}$		0.36	-149.84

5.	A_{30}	$A_{30}A_{34}$		0.36	149.78
		$A_{30}B_{30}$		0.32	134.14
		$A_{30}A_{12}$	-350.91	0.32	-281.92
	A_{12}	$A_{12}A_{30}$	350.91	0.29	276.94
		$A_{12}B_{12}$		0.42	-200.25
		$A_{12}A_{4}$		0.29	-76.69
	A_{4}	$A_{4}A_{12}$		0.32	66.37
		$A_{4}B_{4}$		0.32	131.53
		$A_{4}A_{1}$	-264.69	0.36	-261.73
	A_{1}	$A_{1}A_{4}$	264.69	0.52	160.27
		$A_{1}B_{1}$		0.48	-160.25
6.	B_{34}	$B_{34}A_{34}$		0.28	176.25
		$B_{34}C_{34}$		0.28	176.25
		$B_{34}B_{30}$	-529.38	0.44	-352.50
	B_{30}	$B_{30}B_{34}$	529.38	0.31	458.11
		$B_{30}A_{30}$		0.205	-136.08
		$B_{30}C_{30}$		0.205	-136.08
		$B_{30}B_{12}$		0.28	-185.88

7	B_{34}	$B_{34}C_{34}$		0.28	141.99
		$B_{34}A_{34}$		0.28	142.0
		$B_{34}B_{30}$	-529.38	0.44	-283.97
	B_{30}	$B_{30}B_{34}$	529.38	0.31	687.29
		$B_{30}A_{30}$		0.205	31.28
		$B_{30}C_{30}$		0.205	31.28
		$B_{30}B_{12}$	-701.82	0.28	-745.85
	B_{12}	$B_{12}B_{30}$	701.82	0.25	541.42
		$B_{12}A_{12}$		0.25	-180.5
		$B_{12}C_{12}$		0.25	-180.5
		$B_{12}B_{4}$		0.25	-180.5
8.	B_{30}	$B_{30}B_{34}$		0.31	217.56
		$B_{30}C_{30}$		0.205	143.57
		$B_{30}A_{30}$		0.205	143.87
		$B_{30}B_{12}$	-701.82	0.28	-505.31
	B_{12}	$B_{12}B_{30}$	701.82	0.25	800.08
		$B_{12}C_{12}$		0.25	0
		$B_{12}A_{12}$		0.25	0
		$B_{12}B_{4}$	-701.82	0.25	-800.08

	B_4	B_4B_{12}	701.82	0.28	505.31
		B_4C_4		0.205	-143.87
		B_4A_4		0.205	-143.87
		$B_4\ B_1$		0.31	-217.86
9.	B_{34}	$B_{34}A_{34}$		0.28	180.08
		$B_{34}C_{34}$		0.28	180.08
		$B_{34}B_{30}$	-529.38	0.44	-360.15
	B_{30}	$B_{30}B_{34}$	529.38	0.31	429.26
		$B_{30}A_{30}$		0.205	154.7
		$B_{30}C_{30}$		0.205	-154.7
		$B_{30}B_{12}$		0.28	-120.55
	B_{12}	$B_{12}B_{30}$		0.25	237.25
		$B_{12}A_{12}$		0.25	191.72
		$B_{12}C12$		0.25	191.72
		$B_{12}B_4$	-701.82	0.25	- 620.64
	B_4	B_4B_{12}	701.82	0.28	570.65
		B_4A_4		0.205	-162.47
		B_4C_4		0.205	-162.47
		B_4B_1		0.31	-245.7

	B_{14}	$B_{14}B_{12}$	701.82	0.28	570.65
		$B_{14}A_4$		0.205	-162.47
		B_4C_4		0.205	-162.47
		B_4B_1		0.31	-245.7
10.	B_{30}	$B_{30}B_{34}$		0.31	251.42
		$B_{30}A_{30}$		0.205	·166.36
		$B_{30}C_{30}$		0.205	166.36
		$B_{30}B_{12}$	-701.82	0.28	-583.98
	B_{12}	$B_{12}B_{30}$	701.82	0.25	583.6
		$B_{12}A_{12}$		0.25	-228.75
		$B_{12}C_{12}$		0.25	-228.75
		$B_{12}B_4$		0.25	-126.05
	B_4	B_4B_{12}		0.28	107.22
		B_4A_4		0.28	158.5
		B_4C_4		0.205	158.5
		B_4B_1	-529.38	0.31	-424.24
	B_1	B_iB_4	529.28	0.44	360.16
		B_1A_1		0.28	-180.07
		B_1C_1		0.28	-180.07

11.	A_{34}	$A_{34}B_{34}$	-264.69	0.48	191.06
		$A_{34}A_{30}$		0.52	-191.07
	A_{30}	$A_{30}A_{34}$	264.69	0.36	231.25
		$A_{30}A_{34}$		0.32	-146.48
		$A_{30}A_{12}$		0.32	-847.18
	B_{34}	$B_{34}A_{34}$		0.28	215.55
		$B_{34}C_{34}$		0.28	158.39
		$B_{34}B_{30}$	-529.38	0.44	-373.99
	B_{30}	$B_{30}B_{34}$	529.38	0.31	462.32
		$B_{30}A_{30}$		0.205	-168.8[illegible]
		$B_{30}C_{30}$		0.205	-124.05
		$B_{30}B_{12}$		0.28	-169.45
12.	A_{30}	$A_{30}A_{34}$		0.36	115.94
		$A_{30}B_{30}$		0.32	178.23
		$A_{30}A_{12}$	-350.91	0.32	-294.16
	A_{12}	$A_{12}A_{30}$	350.91	0.29	312.96
		$A_{12}B_{12}$		0.42	-222.22
		$A_{12}A_4$		0.29	-90.73

	B_{30}	$B_{30}B_{34}$		0.31	228.55
		$B_{30}A_{30}$		0.205	204.44
		$B_{30}C_{30}$		0.205	151.66
		$B_{30}B_{12}$	-701.82	0.28	-587.47
	B_{12}	$B_{12}B_{30}$	701.82	0.25	620.14
		$B_{12}A_{12}$		0.25	-251.42
		$B_{12}C_{12}$		0.25	-184.36
		$B_{12}B_{4}$		0.25	-184.36
13.	B_{34}	$B_{34}A_{34}$		0.28	152.83
		$B_{34}C_{34}$		0.28	228.06
		$B_{34}B_{30}$	-529.38	0.44	-380.91
	B_{30}	$B_{30}B_{34}$	529.38	0.31	465.64
		$B_{30}A_{30}$		0.205	-120.32
		$B_{30}C_{30}$		0.205	-180.95
		$B_{30}B_{12}$		0.28	-166.93
	C_{34}	$C_{34}B_{34}$		0.28	152.83
		$C_{34}D_{34}$		0.28	228.06
		$C_{34}C_{30}$	-529.38	0.44	-380.91
	C_{30}	$C_{30}C_{34}$	529.38	0.31	465.64
		$C_{30}B_{30}$		0.205	-120.32
		$C_{30}D_{30}$		0.205	-180.95

		$C_{30}C_{12}$		0.20	-166.93
14.	B_{30}	$B_{30}B_{34}$		0.31	222.36
		$B_{30}A_{30}$		0.205	147.01
		$B_{30}C_{30}$		0.205	220.6
		$B_{30}B_{12}$	-701.82	0.28	-590.02
	B_{12}	$B_{12}B_{30}$	701.82	0.25	623.99
		$B_{12}A_{12}$		0.25	-178.3
		$B_{12}C_{12}$		0.25	-267.34
		$B_{12}B_{4}$		0.25	-178.3
	C_{30}	$C_{30}C_{34}$		0.31	222.36
		$C_{30}B_{30}$		0.205	147.01
		$C_{30}D_{30}$		0.205	220.6
		$C_{30}C_{12}$	-701.32	0.28	-590.02
	C_{12}	$C_{12}C_{30}$	701.82	0.25	623.99
		$C_{12}B_{12}$		0.25	-178.3
		$C_{12}D_{12}$		0.25	-267.34
		$C_{12}C_{4}$		0.25	-178.3

FRAME 2

1	A_1	A_1B_1		0.18	25.80
		A_1A_2	-99.26	0.82	-25.81
	A_2	A_2A_1	99.26	0.63	56.53
		A_2B_2		0.13	-17.86
		A_2A_3		0.24	-36.66
2.	A_1	A_1A_2	-99.26	0.82	15.57
		A_1B_1		0.18	-15.56
	A_2	A_2A_1	99.26	0.63	164.88
		A_2B_2		0.13	6.05
		A_2A_3	-133.29	0.24	-170.94
	A_3	A_3B_3	+ 133.29	0.71	40.08
		A_3B_3		0.29	-40.07
3.	A_2	A_2A_1		0.63	117.35
		A_2B_2		0.13	24.22
		A_2A_3	-133.29	0.24	-141.57
	A_3	A_3A_2	133.29	0.71	44.94
		A_3B_3		0.29	-44.93

4.	B_1	B_1A_1		0.25	58.61
		B_1C_1		0.25	58.61
		B_1B_2	-198.52	0.50	-117.26
	B_2	B_2B_1	198.52	0.29	181.3
		B_2A_2		0.14	-35.73
		B_2C_2		0.14	-35.73
		B_2B_3		0.43	-109.79
5.	B_1	B_1A_1		0.25	45.77
		B_1C_1		0.25	45.77
		B_1B_2	-198.52	0.50	-91.51
	B_2	B_2B_1	198.52	0.29	278.62
		B_2A_2		0.14	15,91
		B_2C_2		0.14	15.91
		B_2B_3	-266.58	0.32	-212.89
	B_3	B_3B_2	266.58	0.66	98.4
		B_3A_3		0.17	-49.19
		B_3C_3		0.17	-49.19
6.	B_2	B_2B_1		0.29	108.29
		B_2A_2		0.14	52.28
		B_2C_2		0.43	52.28
		B_2B_3	-266.58	0.43	-212.89

	B_3	B_3B_2	266.58	0.66	166.56
		B_3A_3		0.17	- 58.27
		B_3C3		0.17	- 58.27
7.	A_1	A_1B_1		0.18	47.34
		A_1A_2	-99.76	0.82	-47.4
	A_2	A_2A_1	99.26	0.63	63.43
		A_2B_2		0.13	-33.25
		A_2A_3		0.24	-30.16
	B_1	B_1A_1		0.25	65.54
		B_1C_1		0.25	56.03
		B_1B_2	-198.52	0.50	121.58
	B_2	B_2B_1	198.52	0.29	189.05
		B_2A_2		0.14	-42.49
		B_2C_2		0.14	-34.3
		B_2B_3		0.43	-105.24
8.	A_2	A_2A_1		0.63	97.04
		A_2B_2		0.13	44.24
		A_2A_3	-133.29	0.24	-141.27

	A_3	A_3A_2	133.29	0.71	62.67
		A_3B_3		0.29	-62.67
	B_2	B_2B_1		0.29	103.4
		B_2A_2		0.14	60.45
		B_2C_2		0.14	49.86
		B_2B_3	-266.56	0.43	-213.77
	B_3	B_3B_2	266.56	0.66	128.05
		B_3A_3		0.17	-73.2
		B_3C_3		0.17	-54.84
9.	B_1	B_1A_1		0.25	51.79
		B_1C_1		0.25	77.11
		B_1B_2	-198.52	0.50	-128.9
	B_2	B_2B_1	198.52	0.29	181.66
		B_2C_1		0.14	-32.58
		B_2C_2		0.14	-48.97
		B_2B_3		0.43	-100.08
	C_1	C_1B_1		0.25	51.79
		C_1D_1		0.25	77.11
		C_1C_2	-198.52	0.50	-129.9

	C_2	C_2C_1	198.52	0.29	181.66
		C_2B_2		0.14	-32.58
		C_2D_2		0.14	48.97
		C_2C_3		0.43	-100.08
10.	B_2	B_2B_1		0.29	99.31
		B_2A_2		0.14	47.94
		B_2C_2		0.14	71.45
		B_2B_3	-266.58	0.43	-218.72
	B_3	B_3B_2	266.58	0.66	132.09
		B_3A_3		0.17	-53.24
		B_3C_3		0.17	-78.84
	C_2	C_2C_1		0.29	99.31
		C_2B_2		0.14	47,94
		C_2D_2		0.14	71.94
		C_2C_3	-266.58	0.43	-218.72
	C_3	C_3C_2	266.58	0.66	132.09
		C_3B_3		0.17	-53.24
		C_3D_3		0.17	-78.84

FRAME: 3

1.	A_{35}	$A_{35}B_{35}$	-566.78	0.43	301.70
		$A_{35}B_{31}$		0.57	-301.71
	A_{31}	$A_{31}A_{35}$	708.61	0.31	622.01
		$A_{31}B_{31}$		0.33	-297.48
		$A_{31}A_{14}$		0.36	-324.51
2.	A_{35}	$A_{35}B_{35}$	-566.78	0.43	264,02
		$A_{35}B_{31}$		0.57	-264,04
	A_{31}	$A_{31}A_{34}$	708.61	0.31	708.12
		$A_{31}B_{31}$		0.33	-106.04
		$A_{31}A_{14}$	-491.57	0.36	-674.05
	A_{14}	$A_{14}A_{31}$	410.19	0.36	277.41
		$A_{14}B_{14}$		0.44	-156.35
		$A_{14}A_{10}$		0.20	-71.07
3.	A_{31}	$A_{31}B_{31}$		0.33	173.33
		$A_{31}A_{35}$		0.31	162.82
		$A_{31}A_{14}$	-491.57	0.36	-336.2
	A_{14}	$A_{14}A_{31}$	410.10	0.36	433.51
		$A_{14}B_{14}$		0.44	-84.68
		$A_{14}A_{10}$	-283..25	0.20	-348,74

	A_{10}	$A_{10}A_{14}$	283.25	0.20	211.62
		$A_{10}B_{10}$		0.44	-116.4
		$A_{10}A_5$		0.36	-95.25
4.	A_{35}	$A_{35}B_{35}$		0.43	303.61
		$A_{35}A_{31}$	-566.78	0.57	-303.61
	A_{31}	$A_{31}A_{35}$	708.61	0.31	597.49
		$A_{31}B_{31}$		0.33	-323.56
		$A_{31}A_{14}$		0.36	-273.92
	A_{14}	A_{14} A_{31}		0.36	7.07
		A_{14} B_{14}		0.44	211.3
		A_{14} A_{10}	-283.25	0.20	-218.37
	A_{10}	A_{10} A_{14}	283.25	0.20	261.69
		A_{10} B_{10}		0.44	-143.94
		A_{10} A_5		0.36	117.76
5.	A_{31}	A_{31} A_{35}		0.31	182.49
		A_{31} B_{11}		0.33	194.26
		A_{31} A_{14}	-491.57	0.36	-376.79
	A_{14}	A_{14} A_{31}	410.19	0.36	308.25
		A_{14} B_{14}		0.44	-248.98
		A_{14} A_{10}		0.20	-59.25

	A_{10}	$A_{10}\ A_{14}$		0.20	53.93
		$A_{10}\ B_{10}$		0.44	237.29
		$A_{10}\ A_5$	-410.19	0.36	-291.24
	A_5	$A_5\ A_{10}$	491.57	0.36	282.27
		$A_5\ B_5$		0.33	-145.55
		$A_5\ A_2$		0.31	-136.73
6.	B_{35}	$B_{35}\ A_{35}$		0.19	270.81
		$B_{35}\ C_{35}$		0.19	270.81
		$B_{35}\ B_{31}$	-1133.56	0.62	-541.67
	B_{31}	$B_{31}\ B_{35}$	1417.22	0.33	1233.56
		$B_{31}\ A_{31}$		0.14	-257.75
		$B_{31}\ C_{31}$		0.14	-257.75
		$B_{31}\ B_{14}$		0.39	-718.06
7.	B_{35}	$B_{35}\ A_{35}$		0.19	234.08
		$B_{35}\ C_{35}$		0.19	234.08
		$B_{35}\ B_{31}$	-1133.56	0.62	-468.17
	B_{31}	$B_{31}\ B_{35}$	1417.22	0.33	1580.14
		$B_{31}\ A_{31}$		0.14	-89.38
		$B_{31}\ C_{31}$		0.14	-89.38
		$B_{31}\ B_{14}$	-983.14	0.39	-1401.36

	B_{14}	$B_{14}\ B_{31}$	820.38	0.46	380.16
		$B_{14}\ A_{14}$		0.22	-154.88
		$B_{14}\ C_{14}$		0.22	-154.88
		$B_{14}B_{10}$		0.10	-70.407
8	B_{31}	$B_{31}\ B_{35}$		0.33	363.78
		$B_{31}\ A_{31}$		0.14	154.33
		$B_{31}\ C_{31}$		0.14	154.33
		$B_{31}\ B_{14}$	-983.14	0.39	-672.44
	B_{14}	$B_{14}\ B_{31}$		0.46	781.74
		$B_{14}\ A_{14}$		0.22	-117.68
		$B_{14}\ C_{14}$		0.22	-117.68
		$B_{14}\ B_{10}$	-470.25	0.10	-546.38
	B_{10}	$B_{10}\ B_{14}$	470.25	0.10	399.89
		$B_{10}\ A_{10}$		0.22	-97.76
		$B_{10}\ C_{10}$		0.22	-97.76
		$B_{10}\ B_{5}$		0.46	-204.40
9.	B_{35}	$B_{35}\ C_{35}$		0.19	274.22
		$B_{35}\ A_{35}$		0.19	274.22
		$B_{35}\ B_{31}$	-1133.56	0.62	-548.43

	B_{31}	B_{31} B_{35}	1417.22	0.33	1175.1
		B_{31} A_{31}		0.14	-282.55
		B_{31} C_{31}		0.14	-282.55
		B_{31} B_{14}		0.39	-608.[illegible]
	B_{14}	B_{14} B_{31}		0.46	30.05
		B_{14} A_{14}		0.22	189.41
		B_{14} C_{14}		0.22	189.41
		B_{14} B_{10}	-470.25	0.10	-408.85
	B_{10}	B_{10} B_{14}	470.25	0.10	457.88
		B_{10} A_{10}		0.22	-111.93
		B_{10} C_{10}		0.22	-111.93
		B_{10} B_{5}		0.46	-234.03
10.	B_{31}	B_{31} B_{35}		0.33	404.37
		B_{31} C_{31}		0.14	171.58
		B_{31} A_{31}		0.14	171.58

		B_{31} B_{14}	-931.14	0.39	-747.47
	B_{14}	B_{14} B_{31}	820.38	0.46	542.17
		B_{14} C_{14}		0.22	-242.33
		B_{14} A_{14}		0.22	-242.33
		B_{14} B_{10}		0.10	-57.48
	B_{10}	B_{10} B_{14}		0.10	57.48
		B_{10} C_{10}		0.22	242.33
		B_{10} A_{10}		0.22	242.33
		B_{10} B_{5}	-820.38	0.46	-542.17
	B_{5}	B_{5} B_{10}	983.14	0.39	747.47
		B_{5} C_{5}		0.14	-171.58
		B_{5} A_{5}		0.14	-171.58
		B_{5} A_{2}		0.33	-407.37
11.	A_{35}	A_{35} B_{35}		0.43	362.55
		A_{35} A_{31}	-566.78	0.57	-362.56
	A_{31}	A_{31} A_{35}	708.61	0.31	636.65
		A_{31} B_{31}		0.33	-364.82
		A_{31} A_{14}		0.36	-271.84

	B_{35}	B_{35} A_{35}		0.19	366.22
		B_{35} C_{35}		0.19	243.90
		B_{35} B_{31}	-1133.56	0.62	-610.15
	B_{31}	B_{31} B_{5}	1417.22	0.33	1250.23
		B_{31} A_{31}		0.14	-361.85
		B_{31} C_{31}		0.14	-234.65
		B_{31} B_{14}		0.39	-653.69
12	A_{31}	A_{31} A_{35}		0.31	151.52
		A_{31} B_{31}		0.33	237.64
		A_{31} A_{14}	-491.57	0.36	-389.21
	A_{14}	A_{14} A_{31}	410.19	0.36	348.20
		A_{14} A_{10}		0.20	-84.09
		A_{14} B_{14}		0.44	-264.10
	B_{31}	B_{31} B_{35}		0.33	367.37
		B_{31} C_{31}		0.14	155.85
		B_{31} A_{31}		0.14	237.37
		B_{31} B_{14}	-983.14	0.39	-761.01
	B_{14}	B_{14} B_{31}	820.38	0.46	599.19
		B_{14} B_{10}		0.10	-94.33
		B_{14} C_{14}		0.22	-207.50
		B_{14} A_{14}		0.22	-297.35

13.	A_{14}	A_{14} A_{31}		0.36	94.22
		A_{14} B_{14}		0.44	162.61
		A_{14} A_{10}	-283.25	0.20	-256.87
	A_{10}	A_{10} A_{14}	283.25	0.20	256.87
		A_{10} B_{10}		0.44	-162.64
		A_{10} A_{5}		0.36	-94.22
	B_{14}	B_{14} B_{31}		0.46	-199.95
		B_{14} C_{14}		0.22	-152.79
		B_{14} A_{14}		0.22	-152.79
		B_{14} B_{10}	-470.25	0.10	448.36
	B_{10}	B_{10} B_{14}	470.25	0.10	448.36
		B_{10} A_{10}		0.22	-152.79
		B_{10} C_{10}		0.22	-152.79
		B_{10} C_{5}		0.46	-199.95
14.	B_{35}	B_{35} A_{35}		0.19	245.84
		B_{35} C_{35}		0.19	360.99
		B_{35} B_{31}	-1131.56	0.62	-606.82
	B_{31}	B_{31} B_{35}	1417.22	0.33	1250.78
		B_{31} C_{31}		0.14	-353.8
		B_{31} A_{31}		0.14	-236.94
		B_{31} B_{14}		0.39	-660.05

	C_{35}	C_{35} B_{35}		0.19	360.99
		C_{35} D_{35}		0.19	245.84
		C_{35} C_{31}	-1133.56	0.62	-606.82
	C_{31}	C_{31} C_{35}	1417.22	0.33	1250.78
		C_{31} D_{31}		0.14	-353.8
		C_{31} C_{14}		0.39	-660.05
15.	B_{31}	B_{31} B_{35}		0.33	382.63
		B_{31} A_{31}		0.14	162.32
		B_{31} C_{31}		0.14	239.14
		B_{31} B_{14}	-983.14	0.39	-784.49
	B_{14}	B_{14} B_{31}	820.38	0.46	531.97
		B_{14} A_{14}		0.22	-240.81
		B_{14} C_{14}		0.22	-180.59
		B_{14} B_{10}		0.10	-109.45
	C_{31}	C_{31} C_{35}		0.33	382.63
		C_{31} B_{31}		0.14	162.32
		C_{31} D_{31}		0.14	239.13
		C_{31} C_{14}	-983.14	0.39	-784.49

	C_{14}	C_{14} C_{31}	820.38	0.46	531.97
		C_{14} B_{14}		0.22	-240.81
		C_{14} D_{14}		0.22	-180.59
		C_{14} C_{10}		0.10	-109.45
16.	B_{14}	B_{14} B_{31}		0.46	204.11
		B_{14} A_{14}		0.22	97.63
		B_{14} C_{14}		0.22	146.26
		B_{14} B_{10}	-470.25	0.10	-447.98
	B_{10}	B_{10} B_{14}	470.25	0.10	447.98
		B_{10} A_{10}		0.22	-146.26
		B_{10} C_{10}		0.22	-97.63
		B_{10} B_{5}		0.46	-204.4
	C_{14}	C_{14} C_{31}		0.46	204.4
		C_{14} B_{14}		0.22	97.63
		C_{14} D_{14}		0.22	146.26
		$C_{14}C_{10}$	-470.25	0.10	-447.98
	C_{10}	C_{10} C_{14}	470.5	0.10	447.98
		C_{10} B_{10}		0.22	-146.26
		C_{10}-D_{10}		0.22	-97.63
		C_{10} C_{5}		0.46	-204.40

FRAME: 4

1.	A_{15}	A_{15} B_{15}	-1306.82	0.54	727.95
		A_{15} A_{17}		0.46	-727.95
	A_{17}	A_{17} A_{15}	1114.74	0.46	762.58
		A_{17} B_{17}		0.56	-762.57
2.	B_{15}	B_{15} A_{15}		0.29	763.25
		B_{15} C_{15}		0.29	763.65
		B_{15} B_{17}	-2073.64	0.42	-1527.31
	B_{17}	B_{17} B_{15}	2229.48	0.42	1602.5
		B_{17} A_{17}		0.29	-801.3
		B_{17} C_{17}		0.29	-801.3
3.	A_{15}	A_{15} B_{15}		0.54	838.8
		A_{15} A_{17}	-1036.82	0.46	-838.8
	A_{17}	A_{17} A_{15}	1114.74	0.46	832.86
		A_{17} B_{17}		0.54	-832.88
	B_{15}	B_{15} A_{15}		0.29	935.53
		B_{15} C_{15}		0.29	667.63
		B_{15} B_{17}	-2073.64	0.42	-1606.16

	B_{17}	B_{17} B_{15}	2229.48	0.42	1637.37
		B_{17} A_{17}		0.29	-958.92
		B_{17} C_{17}		0.29	-680.72
4.	B_{15}	B_{15} C_{15}		0.29	974.67
		B_{15} A_{15}		0.29	649.49
		B_{15} B_{17}	-2073.64	0.42	-1624.41
	B_{17}	B_{17} B_{15}	2229.48	0.42	1708.68
		B_{17} A_{17}		0.29	-684.56
		B_{17} C_{17}		0.29	-1024.11
	C_{15}	C_{15} B_{15}		0.29	974.67
		C_{15} D_{15}		0.29	649.49
		C_{15} C_{17}	-2073.64	0.42	-1624.41
	C_{17}	C_{17} C_{15}	2229.48	0.42	1708.68
		C_{17} B_{17}		0.29	-648.56
		C_{17} D_{17}		0.29	-1024.11

FRAME: 5

1.	A_{43}	$A_{43}\ B_{43}$		0.66	680.34
		$A_{43}\ A_{24}$	-920.43	0.34	-680.34
	A_{24}	$A_{24}\ A_{43}$	920.43	0.205	868.89
		$A_{24}\ B_{24}$		0.59	-644.83
		$A_{24}\ A_{15}$		0.205	-244.06
2.	A_{43}	$A_{43}\ B_{43}$		0.66	607.48
		$A_{43}\ A_{24}$	-920.43	0.34	-607.48
	A_{24}	$A_{24}\ A_{43}$	920.43	0.205	1076.91
		$A_{24}\ B_{24}$		0.59	
		$A_{24}\ A_{15}$	-920.43	0.205	-1076.91
	A_{15}	$A_{15}\ A_{24}$	920.43	0.34	607.48
		$A_{15}\ B_{15}$		0.66	-607.48
3.	B_{43}	$B_{43}\ A_{43}$		0.35	714.69
		$B_{43}\ C_{43}$		0.35	714.69
		$B_{43}\ B_{24}$	-1840.86	0.30	-1429.41
	B_{24}	$B_{24}\ B_{43}$	1840.86	0.19	1736.0
		$B_{24}\ A_{24}$		0.31	-664.38
		$B_{24}\ C_{24}$		0.31	-666.38

		B_{24} B_{15}		0.19	-407.2
4.	B_{43}	B_{43} A_{43}		0.35	644.3
		B_{43} C_{43}		0.35	644.3
		B_{43} B_{24}	-1840.86	0.30	-1288.6
	B_{24}	B_{24} B_{43}	1840.86	0.19	2111.99
		B_{24} A_{24}		0.31	
		B_{24} C_{24}		0.31	
		B_{24} B_{15}	-1840.86	0.19	-2111.99
	B_{15}	B_{15} B_{24}	1840.86	0.30	1288.6
		B_{15} A_{15}		0.35	-644.3
		B_{15} C_{15}		0.35	-644.3
5.	A_{43}	A_{43} B_{43}		0.66	782.18
		A_{43} A_{24}	-920.43	0.34	-782.18
	A_{24}	A_{24} A_{43}	920.43	0.205	825.35
		A_{24} B_{24}		0.59	-612.52
		A_{24} A_{15}		0.205	-212.83
	B_{43}	B_{43} A_{43}		0.35	635.69
		B_{43} C_{43}		0.35	863.25
		B_{43} B_{24}	-1840.86	0.3	-1497.95

	B_{24}	B_{24} B_{43}	1840.86	0.19	1699.11
		B_{24} A_{24}		0.31	-650.28
		B_{24} C_{24}		0.31	-650.28
		B_{24} B_{15}		0.19	-398.55
6.	B_{43}	B_{43} A_{43}		0.35	601.46
		B_{43} C_{43}		0.35	897.84
		B_{43} B_{24}	-1840.86	0.30	-496.32
	B_{24}	B_{24} B_{43}	1840.86	0.19	1752.03
		B_{24} A_{24}		0.31	-561.81
		B_{24} C_{24}		0.31	-848.64
		B_{24} B_{15}		0.19	-344.37
	C_{43}	C_{43} B_{43}		0.35	601.46
		C_{43} D_{43}		0.35	897.84
		C_{43} C_{24}	-1840.86	0.30	-1496.32
	C_{24}	C_{24} C_{43}	1840.86	0.19	1752.03
		C_{24} B_{24}		0.31	-561.81
		C_{24} D_{24}		0.31	-848.64
		C_{24} C_{15}		0.19	-344.37

FRAME: 6

1.	A_{24}	A_{24} B_{24}		0.62	1696.3
		A_{24} A_{26}	-2207.48	0.38	-1696.31
	A_{26}	A_{26} A_{24}	2362.12	0.38	1777.44
		A_{26} B_{26}		0.62	-1777.43
2.	B_{24}	B_{24} A_{24}		0.365	1873.6
		B_{24} C_{24}		0.365	1873.6
		B_{24} B_{26}	-4414.16	0.27	-3747.23
	B_{26}	B_{26} B_{24}	4730.02	0.27	3950.86
		B_{26} A_{26}		0.365	-1975.43
		B_{26} C_{26}		0.365	-1975.43
3.	A_{24}	A_{24} B_{24}		0.62	1923.31
		A_{24} A_{26}	-2207.48	0.38	-1923.22
	A_{26}	A_{26} A_{24}	2362.12	0.38	1975.35
		A_{26} B_{26}		0.62	-1975.35
	B_{24}	B_{24} A_{24}		0.365	2292.3
		B_{24} C_{24}		0.365	1736.46
		B_{24} B_{26}	-4730.02	0.27	-4028.76

	B_{26}	B_{26} B_{24}	4414.16	0.27	3845.15
		B_{26} A_{26}		0.365	-1616.31
		B_{26} A_{26}		0.365	-2228.85
4.	B_{24}	B_{24} A_{24}		0.365	1548.15
		B_{24} C_{24}		0.365	2323.25
		B_{24} B_{26}	-4414.16	0.27	-3871.37
	B_{26}	B_{26} B_{24}	4730.03	0.27	4091.38
		B_{26} A_{26}		0.365	-1638.46
		B_{26} C_{26}		0.365	-2452.86
	C_{24}	C_{24} B_{24}		0.365	1548.15
		C_{24} D_{24}		0.365	2323.25
		C_{24} C_{26}	-4414.16	0.27	-3871.37
	C_{26}	C_{26} C_{24}	4730.02	0.27	4091.38
		C_{26} B_{26}		0.365	-1638.46
		C_{26} D_{26}		0.365	-2452.86

DESIGN OF BEAMS

Introduction:

The beams constructed at the lintal levels are designed as rectangular sections whereas other beams are designed as 'T' section or 'L' section at the centre and rectangular section at the support section using limit state design procedure, as per IS 456-1978.

The assumptions in the design of beam are intended to ensure ductile failure, that is the tensile reinforcement had to undergo a certain degree of inelastic deformation before concrete fails in compression.

The limiting moment and the area of steel are calculated for the beam A_1 - A_4 in the support section and at the mid section as model calculation. For other beams the limiting moments are calculated using the design aids to IS 456-1978 and the area of steels are callculated and are found in the tables.

Typical design calculations are given for each type of beam in the following pages.

DESIGN OF BEAMS

Beam A_1 - A_4

From first principles at the support section

Mix used = M200

Steel = = Tor steel

Size of the beam = 23 cm x 30 cm

Factored moment at A_1 = 1.5 x 755.002 = 1132.5 Kg - m

" " " A_4 = 1.5 (1363 - 203 + 356.23)

= 2579.15 Kg-m

Assuming 12 mm Ø bars with 30 mm clear cover

effective depth d = 30 -3-1 2/2 = 26.4 cm

$\frac{Mulim}{bd^2}$ = 2.76 N/mm²

= 27.6 Kg/cm²

Mulim = 27.6 x 23 x 26.4^2 = 442430.21 Kg/cm

= 4424.3 Kg-m

$\therefore$ Mulim = Mu

Therefore the section is to be designed as a single reinforced rectangular section

From IS 456 - 1978

$$Mu = 0.87 \; fy \; Ast \; d\left(1 - \frac{Ast \; fy}{bd \; fck}\right)$$

$$257915 = 0.87 \times 4150 \times Ast \times 26 \; A \left(1 - \frac{Ast \; 4150m}{23 \times 26.4 \times 200}\right)$$

$\therefore$ Ast = 3.016 Cm²

As per IS 456 - 1978 code minimum reinforcement

Ast $= \frac{0.85}{4150} \times 23 \times 26.4$

$= 1.243\ Cm^2$

No. of rods $= \frac{3.016 \times 4}{\Pi \times 1.2^2} = 2.67$

Provide 3 rods of 12mm Ø

From Design Aids

Beam $A_1 A_4$

Factored moment Mu $= 257915$ Kg cm

$\frac{MU}{bd^2} = \frac{257915}{23 \times 264^2} = 16.09\ Kg/cm^2$

$= 1.6\ N/mm^2$

Pt $= 0.494\%$

$\therefore$ Ast $= \frac{0.494 \times 230 \times 264}{100} = 299.96\ mm^2$

$= 2.99\ Cm^2$

This is nearly the same as that computed from first principles.

Design of L - Beam $A_1 A_4$ at the mid section

D.L B.M at the centre $= 920.08$ Kg - m

L.L. B.M at the centre $= 235.04$ Kgm

Factored moment $= 1.5\ (920.08 + 235.04)$

$= 1732.68$ Kg-m

Assume xu < Df

For L beam bf $= \frac{10}{12} + bw + 3\ Df$

$= \frac{0.7 \times 424}{12} + 23 + 3 \times 10 = 77.73$ cm

Mu $= 0.87\ fy\ Ast\ d\ (1 - \frac{Ast\ fy}{bd\ fck})$

From this Ast $= 1.33\ cm^2$

$$\frac{xu}{d} = \frac{0.87\ fu\ Ast}{0.36\ fck\ bd}$$

$$= \frac{0.87 \times 4150 \times 1.33}{0.36 \times 200 \times 77.73 \times 36.4}$$

$$= 0.024 < \frac{Xumax}{d}$$

Minimum reinforcement

$$\frac{Ast}{bd} = \frac{0.85}{fy}$$

$$Ast = \frac{0.85}{4150}\ V\ 1348.5 = 0.28\ cm^2$$

Provide 2 bars of 12 mmØ

Beam A_1A_4

$$\frac{m_u}{bd^2} = \frac{173268}{23 \times 36.4^2} = 5.69$$

From design aids rto is 456 - 1978

Pt $= 0.163\ \%$

Ast $= 1.36\ cm^2$

It is nearly equal to the area of reinforce ment calculated from the first principles.

DISIGN OF T-beam A24 Z26 From first Principles

Net B.M. at the centre = 7232 . 515 kg.m.

Factored moment = 19848.77 kg.,m

Assume $X_u < D_f$

For T - beam, bf $= \frac{lo}{6} + bw + 6\,Df$

$= \frac{0.7 \times 550}{6} + 23 + 6 \times 10 = 147.17$ cm

Mu $= 0.87 fy\, Ast\, d \left(1 - \frac{Ast\, fy}{bf\, d\, fck}\right)$

$1084877 = 0.87 \times 4150\, Ast \times 61.4 \left(1 - \frac{Ast \times 4150}{147.17 \times 61.4 \times 200}\right)$

$\therefore$ Ast $= 4.95\ cm^2$

$\frac{X_u}{d} = \frac{0.87 \times 4150 \times 4.95}{0.36 \times 200 \times 147.17 \times 61.4} = 0.027 < \frac{X_{umax}}{d}$

Provide 2 bars of 20 mm. ϕ

For beam $A_{24}\,A_{26}$

$\frac{Mu}{bd^2} = \frac{1084877}{23 \times 61.4^2} = 12.5$

From design aids to IS 456 - 1978

Pt = 0.376%

- Ast = 5.3 Cm^2

This is nearly same as the area of reinforcement calculated from the first principles.

Check for support section.

Frame 3 Joint A_{14}

D.L B.M a_{14} due to beam $A_{14} A_{10}$ = -708.342 Kg -m

D.L.B.M at A_{14} due to beam $A_{14} A_{31}$ = 1314.72 74 Kg m

606.385 Kg-m

L.L.B.M at A_{14} due to beam $A_{14} A_{10}$ = -348.79 Kg-m

L.L B.M at A_{14} due to beam $A_{14} A_{31}$ = 433.51 Kg-m

84.72 Kg-m

Factored moment = 1.5 (84.72 + 606.385)

= 1036.66 Kg-m

$\frac{\text{Mu limit}}{bd^2}$ = 2.76

$\therefore$ Mulim = $276 \times 23 \times 21.4^2$ = 290713.2 Kg-cm

Mulim > Mu

$\frac{Mu}{bd^2}$ = 9.84 Kg/cm²

Pt = 0.295%

$\therefore$ ast = 1.46 cm²

Provide 12mm∅ of 2 Nos

Design for shear reinforcement

to find the shearforce at B_1

shear force due to $B_1 B_2$

Area of the triangle = $2.94 \times \frac{2.94}{2} \times \frac{1}{2} = 2.16 m^2$

Live load = $2.16 \times 300 = 648$ Kg

Self wt. of the slab = $2.16 \times 10 \times 2400$

= 518.4 Kg

Self wt. of the beam = $0.23 \times 0.35 \times 1 \times 2400$

= 193.2 Kg

Wall weight = $0.20 \times 2.8 \times 2.94 \times 2000$

= 3316.32 Kg

Total load = 4675.92 Kg

Shear force at B_1 = 2337.96 Kg

Shearforce due to beam B_1 B_4

Area of trapozoidal = $\frac{4.24 + 1.3}{2} \times 1.47$

= $4.0719 m^2$

Live load = 4.0719×300 = 1221.57 Kg

Self wt. of the slab = $4.5119 \times 1 \times 2400$ = 977.256 Kg

Self wt. of beam = $0.23 \times 0.35 \times 2400$ = 193.2 Kg

Wall weight = $0.2 \times 2.82 \times 4.24 \times 2000$

= 4782.72 Kg

Total load = 7174.746 Kg

S.F at B_1 = 3587.373 Kg

Total shear force at B_1 = 5925.33 Kg

Shear stress = $\frac{V}{bd}$ = $\frac{5925.33}{23 \times 31.4}$

= 8.2 Kg/cm^2

$\frac{Ast}{bd} \times 100$ = $\frac{3 \times 1.13 \times 100}{23 \times 31.4}$ = 0.469 %

From IS 456 - 1978

Permissible shear stress in concrete = 0.28 N/mm²

= 2.8 Kg/cm²

Shear force to be provided by the stirrups

= 5925.33 - 2.8 x 23 x 31.4

= 3903.17 Kg

$$S_v = \frac{2 \times \pi \times (0.6)^2 \times 1400 \times 31.6}{4 \times 3903.17}$$

= 6.4 Cm

Provide 6 mm Ø 2 legged stirrups at 6 mm C/C

Control of Deflection

For beam A_{26} A_{24}

Span = 5.2 m

Depth = 56.4 cm

$$\frac{\text{Span}}{\text{Depth}} = \frac{520}{56.4} = 9.22$$

$$\frac{100\ Ast}{bd} = \frac{100 \times (2)^2 \times \pi \times 5}{4 \times 23 \times 56.4} = 1.21\%$$

From chart 22 of design aids to IS code 456 - 1978

Max. span /eff. depth = 19.8 > 9.22

Hence safe.

Beam	Factored moment Kg -m	Size cm x cm	$\frac{Mu}{bd^2}$ Kg Cm²	Pt	Ast cm²	Dia of rod mm	No. of rods	Cross section at mid span.
1	2	3	4	5	6	7	8	9
Frame 1								
$A_1 A_4$	2579.15	23 x 30	16.09	0.494	2.99	12	3	L
$A_4 A_{12}$	2973.2	23 x 30	18.55	0.584	3.55	12	4	L
$A_{12} A_{30}$	2973.2	23 x 30	18.55	0.584	3.55	12	4	L
$A_{30} A_{34}$	2577.15	23 x 30	16.09	0.494	2.99	12	3	L
$B_1 B_4$	4388.77	23 x 35	21.56	0.701	5.06	12	5	L
$B_4 B_{12}$	5711.74	23 x 35	25.19	0.843	6.12	12	6	L
$B_{12} B_{30}$	5711.94	23 x 35	25.19	0.848	6.12	12	6	L
$B_{30} B_{34}$	4888.77	23 x 35	21.56	0.701	5.06	12	5	L
Frame 2								
$A_1 A_2$	1260.34	23 x 20	20.37	0.655	2.47	12	3	L
$A_2 A_3$	1294.62	23 x 20	20.93	0.678	2.56	12	3	L
$B_1 B_2$	2211.77	23 x 25	20.99	0.678	2.56	12	3	L
$B_2 B_3$	2373.81	23 x 25	22.54	0.737	3.63	12	4	L

1	2	3	4	5	6	7	8	9
Frame 3								
$A_2 A_5$	3023.4	23 x 30	18.66	0.602	3.655	12	4	T
$A_5 A_{10}$	4059.66	23 x 30	25.33	0.852	5.173	12	5	T
$A_{10} A_{14}$	1585.7	23 x 20	25.63	0.866	3.27	12	3	T
$A_{14} A_{31}$	4059.66	23 x 30	25.33	0.852	5.173	12	5	T
$A_{31} A_{35}$	3024.4	23 x 30	18.86	0.602	3.655	12	4	T
$B_2 B_5$	6817.67	23 x 45	17.29	0.54	5.17	12	5	T
$B_5 B_{10}$	7523.28	23 x 45	19.08	0.604	5.75	12	6	T
$B_{10} B_{14}$	4792.0	23 x 30	29.8	0.955	5.8	12	6	T
$B_{14} B_{31}$	7523.28	23 x 45	17.08	0.604	5.75	12	6	T
$B_{31} B_{35}$	6817.67	23 x 45	17.29	0.54	5.17	12	5	T
Frame 4								
$A_{15} A_{17}$	6893.095	23 x 45	17.84	0.565	5.33	20	2	L
$B_{15} B_{17}$	11108.91	23 x 55	18.57	0.534	6.85	20	3	L

1	2	3	4	5	6	7	8	9
Frame 5								
$A_{15} A_{24}$	7990.12	23 x 40	26.80	0.919	7.61	20	3	L
$A_{24} A_{43}$	7990.12	23 x 40	26.80	0.919	7.61	20	3	L
				Pt Pc				
$B_{15} B_{24}$	16198.69	23 x 50	33.28	1.13 0.18	11.96 1.9	20	4 + 1	L
$B_{24} B_{43}$	16198.69	23 x 50	33.28	1.13 0.18	11.96 1.9	20	4 + 1	L
Frame 6								
$A_{24} A_{26}$	10848.77	23 x 55	12.5	0.376	5.3	20	2	T
$B_{24} B_{26}$	26424.423	23 x 60	20.06	0.883	13.49	20	5	T

ANALYSIS AND DESIGN OF SECONDARY BEAMS

Depending upon the planning of the apartmental building secondary beams are also provided as shown in fig. In this project the secondary beams are analysed by elastic method and designed by working stress method using high strength steel (Fe 415) and concrete mix M_{200}

Notations for analysis:

Ast	Area of c.s of steel in tension
b	breadth of slab
d	eff. depth of slab
D	total depth of slab
bf	flange width of T beam
x	depth of neutral. axis
F	Maximum shear force
y	lever arm
C	stress in concrete
t	stress in steel

Design data

Mix used	= M_{200}
Steel	= Fe_{415}
M.R	= 94 Π bd^2
jd	= 0.904d
st	= 2300 Kg/cm^2

Design of SB_1 (Floor level)

Assume size of beam as 23 cm x 45 cm

Total load from the slab = 640 Kg/m^2

Dead weight of beam = 248.4 Kg/m

-Ve F.E.M due to L.L = $\frac{2327 \times 1.15 \times 2.1^2}{3.25^2} + \frac{1.51 \times 640}{12 \times 3.25}(325^2 - 1.51^2)$

$(2 \times 3.21 - 1.51) + \frac{5}{96} \times 1.05 \times 640 \times 3.25^2$

= 1117.29 + 568.65 + 1144.2 = 2830.14 Kg-m

-Ve F.E.M due to DL = $\frac{248.4 \times 3.25^2}{12}$ = 218.64 Kg-m

Total F.E.M = 3048.78 Kg-m

$1.5 \times 9.11 \times 23\, d^2$ = 304878 Kg-cm

d = 31.2 cm

∴ provide 23 cm x 45 cm size

Ast reqd = $\frac{304878}{2300 \times 0.904 \times 42}$ = 3.5 cm^2

Provide 4 nos of 12 mmØ bars

Actual area = 4.52 cm^2

Check for shear

Shear force = $\left(\frac{1.51}{2}(0.23 + 3.25) + \frac{1.24}{2}(0.78 + 3.25) + \frac{248.4 \times 3.25}{2} \right)$

= 2005.38 Kg

Shear stress = $\frac{2005.38}{23 \times 42}$ = 2.07 Kg/cm²

$\tau_c bd$ = 3 x 23 x 42 = 2898 Kg

Vs = 528.14 Kg

Spacing of vertical Stirrups = $\frac{\sigma st.xAsv \times d}{Vs}$ = $\frac{2300 \times 0.56 \times 42}{528.14}$

= 103 cm

Provide 6 mm Ø 2 legged vertical stirrups @ 20 cm C/C

Section at mid span

Net B.M at centre = 3160.15 - 3934.74

= -774.59 Kg - m

Provide 2 nos of 12 mm Ø

Design of SB_1 (Roof level)

Assume size of beam as 23 cm x 23 cm

Total load from the slab = 490 Kg/m²

Dead wt. of beam = 0.23 x 0.23 x 2400

= 127 Kg/m

-Ve FEM due to L.L = $\frac{W_1}{12L}$ (L^3 - a^2 (2L - a)

= $\frac{490 \times 1.51}{12 \times 3.25}$ (3.25^2 -1.51^2 (2x3.25-1.51)

= $\frac{+490 \times 1.24}{12 \times 3.25}$ (3.25^2-1.25^2(2x3.25-1.24)

=435.41 + 408.8 = 844.22 Kg-m

-ve F.E.M due to DdL $= \frac{Wl^2}{12} =$ 111.79 Kg-m

Total - ve F.E.M = 844.22 + 111.79 + 956 Kg-m

1.5 m = F.F.M

$1.5 \times 9.11 \times 23 \times d^2$ = 95600 Kgcm

$\therefore$ d = 17.44 Cm

Provide D = 23 cm eff. depth d = 20 cm

$Ast = \frac{M}{tjd} = \frac{95600}{2300 \times 0904 \times 20} = 2.29 cm^2$

Provide 3 12 mm ϕ bars

Actual area provided = $3.39\ cm^2$

$S.F = \frac{1.51}{2}(0.23 + 3.25) + \frac{1.24}{2}(0.78 + 3.25)\ 490 + \frac{127 \times 3.25}{2}$

= 2718.12 Kg

Check for shear

$\tau v = \frac{V}{bd} = \frac{2718.12}{23 \times 20} = 5.9\ Kg/cm^2$

Allowable shear stress = $3.5\ Kg/cm^2$

$\therefore$ Shear reinforcement is necessary

$Vs = \frac{\sigma sv \quad Asv\ d}{Sv}$ = 1108.12 Kg

Provide 6 mm ϕ of 2 legged stirrups

$Spacing = \frac{2300 \times \frac{2\ \pi \times 0.6^2}{4} \times 20}{1108.12} = 23.47 cm$

adopt 20cm C/C

At mid section T - beam

Net B.M $= -956 + 8667 = -89.3$ Kg -M

Provide 2 bars of 12 mm Ø

Span/3 = 108 Cm, C/C = 275 Cm, 12 ds + dbr = 143 cm

Adopt bf 108cm

$$M\ Ast\ (d-x) = \frac{bx^2}{2}$$

$$\frac{108\ x^2}{2} = 13 \times 2.26\ (30 - x)$$

$$\therefore\ x = 3.78\ cm$$

Check for stresses

M.R $= C/2\ \ bx\ \ (d-x/3)$

$8930 = C/2\ \ (108)\ \ 3.78\ \ (30 - 1.26)$

$\therefore$ C $= 1.52$ Kg /cm²

$$C/x = \frac{t/m}{d-x} \quad \therefore\ t = 150.24\ Kg/cm^2 < 2300\ Kg/cm^2$$

Hence safe.

The other secondary beems are also designed similarly and design perametors are tabulated as given in the table. The reinforcement details are given in the figure.

At mid section

Beam	Flange Width	Simply supported B.Mat the centre	F.E.M at the support	Net B.M at the centre	required Ast.	Dia of the rods.	No. of rods	Stress in concrete	Stress in steel	Spacing of sheear reinforcement 6mm ∅ 2 legged stirrups
	cm	Kg-m	Kg m	Kg m	Cm^2	mm		Kg/cm^2	Kg/Cm^2	CM
1	2	3	4	5	6	7	8	9	10	11
Roof level										
SB_2	108	866.7	956	-89.70	--	12	2	1.52	150.24	25
SB_4	98	1156.98	728.06	428.92	0.85	10	2	10.57	1025	25
SB_{22}	96.3	608.05	692.85	-84.78	--	10	2	2.12	208.97	25
SB_{33}	77	251.6	363.012	-111.56	--	10	2	4.04	337.11	25
SB_{11}	143	4366.24	6390.25	-2023.28	--	10	2	12.57	2230+	25
SB_{44}	77	251.6	263.072	-111.56	--	10	2	4.04	337.11	25
Floor level										
SB_2	108	3160.15	3934.74	-774.59	--	12	2	13.20	1220.87	25
SB_4	98	2723.17	1758.9	964.26	1.397	12	2	13.6	1262.26	25
SB_{22}	96.33	2196.6	1697.5	500.1	0.72	10	2	8.32	914.98	25
SB_{33}	77	1173.8	973.8	200	0.36	12	2	4.26	262.63	25
SB_{11}	143	18619.36	13664.4	4954.96	3.7	12	4	16.3	1741	25
SB_{44}	77	1173.8	973.8	200	0.36	12	2	4.26	262.63	25
SB_5	53	1243.5	822.7	419.6	1.07	12	2	28.3	2140	25
SB_6	63	250.9	157.4	91.49	0.18	12	2	2.46	158.7	25

DESIGN OF COLUMN

All compression members are to be designed for a minimum excentricity of load in two principle directions

$$Cmin = \frac{1}{500} + \frac{D}{30}$$ subjected to a minimum of 2 cm

In our case the columns are su bjected to axial load and bi-axial bending. Exact design of members subject to axial load and biaxial bending is extremely laborious. Therefore the code permits the design of such members by the following equation

$$\left(\frac{Mux}{Mux_1}\right)^{\alpha n} + \left(\frac{Muy}{Muy}\right)^{\alpha n} \leq 1.0$$

where Mux, Muy are the moments about x and y axes respectively due to design loads.

Mux, Muy, are the maximum uniaxial moments capacities with an axial load Pu, bending about X and Y axes respectively and n is an exponent whose value depends on Pu/Puz

Where Puz = 0.45 fck Ac + 0.75 fya

Pu/puz	n
0.2	1.0
0.8	2.0

For intermediate values linear inter polation may be done

In this project limit state design method has been used to design the columns subjected to biaxial bending and axial thrust. Only rectangular sections with equal moments of reinforcement on each side have been considered.

Assumptions in the design:

1. The maximum compressive strain in concrete in axial compression is taken as 0.002.
2. The maximum compressive strain at the lightly compressed extreme fibres in concrete subjected to axial compression and bending and where there is no tension on the section shall be (0.0035 - 0.75) times the strain at the least compressed extreme fibre.

Design of columns

Column C_1 (A_1 B_1)

Size of column 23 cm x 45 cm

Axial load =	= 4096.89 Kg	
Factored load	= 1.5 x 4096.89	= 6145.335 Kg

From A_1 A_2

D.L B.M about xx axis	= -106.676 Kg-m
L.L B.M about xx axis	= -47.4
	= -154.076 Kg-m

From $A_1 A_4$

D.L B.M about Y Y axis = 597.132 Kg-m

L.L B.M about Y-Y axis = 191.070 Kg-m

788.202 Kg-m

Factored moment Mux = 154.076 x 1.5 = 231.114 Kg-m

Factored moment Muy = 788.202.15 = 1182.303 Kg-m

First trial

Assume the reinforcement percentage P = 0.8

$\frac{P}{fck} = \frac{0.8}{20} = 0.04$

Uniaxial moment capacity of the section about xx axis

d' = 40 mm + 10mm (Using 20mm Ø bars)

d' = 5 cm

$\frac{d'}{D} = \frac{5}{23} = 0.217$

$\frac{Pv}{fck\ .b.D} = \frac{6145.335}{200 \times 23 \times 40} = 0.03$ (refer chart 32)

$\frac{Mu}{fck\ b\ D^2} = 0.06$

$Mux_1 = 0.06 \times 200 \times 23^2 \times 40$

$= 2539.20$ Kg-m

Uniaxial moment capacity of the section about yy axis

$\frac{d'}{D} = \frac{5}{40}$ = 0.125

$\frac{Mu}{fck\ b\ d^2}$ = 0.0675

Muy, = 0.0675 x 200 x 23 x 40^2

= 4968 Kg-m

Calculation of Puz

Referring to chart 63 in design aid to I.S code 456 - 1978 for P = 0.8; fy = 4150; fck = 200

$\frac{Puz}{Ag}$ = 11.5 N/mm^2 = 115 Kg/cm^2

Puz = 115 x 40 x 23

= 105800

$\frac{Pu}{Puz} = \frac{6145.335}{105800}$ = 0.058

$\frac{Mux}{Mux_1} = \frac{231.114}{2439.20}$ = 0.091

$\frac{Muy}{Muy_1} = \frac{1182.303}{4968}$ = 0.238

Referring to chart 64

$\frac{Mux}{Mux_1}$ = 0.77 corresponding the $\frac{Pu}{Pue}$ and $\frac{Muy}{Muy_1}$

This is greater than 0.091

∴ provide minimum reinforcement of 0.8%

Ast $\frac{0.8}{100}$ x 23 x 40 = 7.36 CM^2

Using 16mm Ø bars

no. of bars $= \dfrac{7.36}{\dfrac{\Pi \times 1.6^2}{4}} - 4$

Provide 4 nos of 16mm 0 bars

Column C_1 (E_1 F_1)

Size of column = 23 x 40 cm

Axial load = 33527.93 kg

Factored load = = 1.5 x 33527.93

= 50291.895 Kg

From E_1 E_2

D.L B.M about xx axis = -558.50 Kg-m

L.L B.M about xx axis = -128.900 Kg-m

-687.407 Kg-m

From E_1 E_4

D.L B.M about yy axis = + 1130.569 Kg-m

D.L B.M about yy axis = 380.910 Kg-m

1511.479 Kg-m

Mux = 687.407 x 1.5 = 1031.11 Kg-m

Muy = 1511.479 x 1.5 = 2267.22 Kg-m

Assume P = 0.8%

$\dfrac{P}{fck} = \dfrac{0.8}{20} = 0.04$

Uniaxial moment capacity of the section about xx axis

d' = 5 cm

$\frac{d'}{D}$ = 5/23 = 0.217

$\frac{Pu}{fck\ b\ D} = \frac{50291.895}{200 \times 23 \times 40}$ = 0.27

$\frac{Mu}{fck\ b\ D^2}$ = 0.082

Mux_1 = 0.082 x 200 x 23^2 x 40 = 34702.4 Kg - cm

= 3470.24 Kg-m

Uniaxial bending moment about yy axis

$\frac{d'}{D} = \frac{5}{40}$ = 0.125

$\frac{Mu}{fck\ bd^2}$ = 0.093

Muyl = 0.093 x 200 x 23 x 40^2

= 6844.80 Kg-m

$\frac{Puz}{Ag}$ = 115 Kg/cm^2

Puz = 115 x 40 x 23 = 105800 Kg

$\frac{Pu}{Puz} = \frac{50291.895}{103800}$ = 0.475

$\frac{Mux}{Mux_1} = \frac{1031.11}{3470.24}$ = 0.297

$\frac{Muy}{Muy_1} = \frac{2267.22}{6844.80}$ = 0.33

Corresponding to $\frac{Pu}{Puz}$ and $\frac{Mu}{Muy_1} \times \frac{M\mu x}{Mux_1}$ = 0.85

Provide minimum reinforcement of 0.8%

6 bars of 16mm ∅

Provide 6 mm ∅ lateral ties at 24 cm C/C

Column C_2 (E_2 F_2)

Size of column	=	23 cm x 40 cm
Axial load	=	49552.51
Factored load = 1.5 x 49552.51	=	74328 • 23 Mg

From E_2 E_1

D.L B.M about xx axis	=	1198.61 Kg-m
L.L B.M about xx axis	=	181.66 Kg-m
		1380.27 Kg-m

From E_2 E_3

D.L B.M about xx axis	=	-1269.33 Kg-m
L.L. B.M about xx axis	=	-100.08 Kg-m
		- 1369.41 Kg-m

From E_2 E_5

D.L B.M about yy axis	=	1488.67 Kg-m
L.L. B.M about yy axis	=	606.82 Kg-m
		2095.49 Kg-m

Mux = 1.5 (1380.27 - 1369.41) = 16.29 Kg-m

Muy = 1.5 (2095.49) = 3143.24 Kg-m

Assume b = 0.8%

$\frac{b}{fck} = \frac{0.8}{20} = 0.04$

Uniaxial moment capacity of the section about a xx axis

$\frac{d'}{D} = \frac{5-}{23} = 0.217$

$\frac{Pu}{fck\ bD} = \frac{5}{23} = 0.217$

$\frac{Pu}{fck\ bD} = \frac{74328.23}{200 \times 23} \times 40 = 0.404$

$\frac{Mu}{fck\ bd^2} = 0.06$

$Mux_1 = 0.06 \times 200 \times 40 \times 23^2 = 2539.20$ Kg-m

Uni axial moment capacity of the section about yy axis

$\frac{d'}{D} = \frac{5}{40} = 0.125$

$Muy_1 = 0.061 \times 200 \times 40^2 \times 23$

$= 4489.6$ Kg-m

$\frac{Puz}{Ag} = 115;\quad Puz = 105800$ Kg

$\frac{Pu}{Puz} = \frac{74328.23}{105800} = 0.7$

$\frac{Mux}{Mux_1} = \frac{16.29}{2539.20} = 0.0064$

$\frac{Muy}{Muy_1} = \frac{3143.24}{4489.60} = 0.7$

$\frac{Mux}{Mux_1} = 0.67$ (Corresponding to above $\frac{Pu}{Puz}$ & $\frac{Muy}{Muy_1}$)

Provide minimum reinforcement

Provide 6 bars of 16mm ∅

Adopt 6 mm ∅ lateral ties at 24 cm C/C

Column C_4 (E_4 F_4)

Size of column	=	23 x 40 cm
Axial load	=	51649.78 Kg
Factored load	=	51649.78 x 1.5
	=	77474.67 Kg

From E_4 E_1

D.L Moment about xx axis	=	2584.53
L.L Moment about xx axis	=	222.36
		2806.89 Kg-m

From E_4 E_{12}

D.L moment about xx axis	=	-2896.32
L.L moment about xx axis	=	- 590.-2
		-3486.34 Kg-m

From E_4 E_5

D.L moment about yy axis	=	1027.64
L.L moment about yy axis	=	335.3
		1362.94 Kg-m

Mux = 1.5 (2806.89 - 3486.34) = 1019.175 Kg

Muy = 1.5 x 1362.94 = 2044.41 Kg-m

Assume P = 0.8%

Uniaxial moment capacity of the section about xx axis

$$\frac{P}{f_{ck}} = \frac{0.8}{20} = 0.04$$

$$\frac{d'}{D} = \frac{5}{40} = 0.125$$

$$\frac{P_v}{f_{ck}\ b\ D} = \frac{77474.67}{200 \times 23 \times 40} = 0.42$$

$$\frac{M_u}{f_{ck}\ b\ D^2} = 0.054$$

$Mux_1 = 0.054 \times 200 \times 23 \times 40^2 = 3974.4$Kg.m

Uniaxial moment capacity of the section about Y Y - axis

$$\frac{d'}{D} = \frac{5}{23} = 257$$

$$\frac{Mu}{fck\ b\ D^2} = 0.05$$

$Muy_1 = 0.05 \times 200 \times 40 \times 23^2 = 2116$Kg.m

$Puz = 105800$Kg

$$\frac{Pu}{Puz} = \frac{77474.67}{105800} = 0.73$$

$$\frac{Mux}{Mux_1} = \frac{1019.175}{3974.4} = 0.26$$

$$\frac{Muy}{Muy_1} = \frac{2044.41}{2116} = 0.97$$

$$\frac{Mux}{Mux_1} = 0.27$$

Provide reinforcement of 0.8%

Provide 6 bars of 16mmØ

Adopt 6mmØ lateral ties at 24cm c/c

Column C_5 (E_5F_5)

Size of column	=	23 x 45
Axial load	=	83114.71Kg
Factored load	=	1.5 x 83114.71
	=	124672.07Kg

From E_5E_2

D.L. B.M. about xx axis	=	-3952.44
L.L. B.M. about xx axis	=	-382.63
		-4335.07 kg.m

From E_5E_{10}

D.L. B.M. about xx axis	=	3639.22
L.L. B.M. about xx axis	=	784.49
		-4423.71 kg.m

From E_5E_4

D.L. B.M. about yy axis	=	1027.64
L.L. B.M. about yy axis	=	335.3
		1362.94Kg.m

$Mux = 1.5\ (4423.71 - 4335.07) = 132.9$ Kg.m

$Muy = 1.5 \times 1362.94 = 2044.41$ Kg.m

Assume P = 2%

$$\frac{P}{fck} = \frac{2}{20} = 0.1$$

Uniaxial moment capacity of the section about xx axis

$$\frac{d'}{D} = \frac{5}{45} = 0.11$$

$$\frac{Pu}{fck\ b\ D} \qquad \frac{124672.07}{200 \times 23 \times 45} = 0.60$$

$$\frac{Mu}{fck\ b\ D^2} = 0.07$$

$Mux_1 = 0.07 \times 200 \times 23 \times 45^2 = 6520.5$ Kg.m

Uniaxial moment capacity of the section about yy axis.

$$\frac{d'}{D} = \frac{5}{23} = 0.217$$

$$\frac{Mu}{fck\ b\ D^2} = 0.06$$

Muy_1 = $0.06 \times 200 \times 40 \times 23^2$ = 2539.2Kg.m

$\frac{Puz}{Ag}$ = 115

Puz = $115 \times 23 \times 45$ = 119025Kg

$\frac{PU}{Puz}$ = $\frac{124672.07}{119025}$ = 1.05

$\frac{Mux}{Mux_1}$ = $\frac{132.96}{6520.5}$ = 0.02

$\frac{Muy}{Muy_1}$ = $\frac{2044.41}{2539.2}$ = 0.81

Ast = $\frac{2 \times 23 \times 45}{100}$ = 20.7cm²

Provide 8 bars of 20mm ∅

Adopt 6mm ∅ lateral ties at 24cm c/c

Column C_{10} (E_{10} F_{10}) L Column

Size of column	=	50cm x 50cm x 23cm
Axial load	=	74449Kg
Factored load	=	111673.5Kg

From E_{15} E_{17}

D.L. B.M. about xx axis	=	-3096.18
L L. B.M. about xx axis	=	-1624.41

		-4720.59Kg.m

From E_{15} E_{24}

D.L. B.M. about yy axis	=	6080.995
L.L. B.M. about yy axis		1496.320
		7577.32Kg.m

Mux = 1.5 (4720.59) = 7080.89Kg.m

Muy = 1.5 (7577.32) = 11365.98Kg.m

Assume P = 2%

Uniaxial moment capacity of the section about xx axis

$$\frac{p}{fck} = \frac{2}{20} = 0.1$$

$$\frac{d'}{D} = \frac{5}{50} = 0.1$$

$$\frac{Pu}{fck\ b\ D} = \frac{111673.5}{200 \times 23 \times 50} = 0.49$$

$$\frac{Mu}{fck\ b\ D^2} = 0.11$$

Mux_1 = $0.11 \times 200 \times 23 \times 50^2$ = 12650Kg.m

Uniaxial moment capacity of the section about yy axis

$$\frac{d'}{D} = \frac{5}{50} = 0.1$$

Muy_1 = 12650.Kg.m

Puz = $150 \times 50 \times 23$ = 172500Kg

$$\frac{Pu}{Puz} = \frac{111673.5}{172500} = 0.65$$

$$\frac{Mux}{Mux_1} = \frac{7080.89}{12650} = 0.56$$

$$\frac{Muy}{Muy_1} = \frac{11365.98}{12650} = 0.898$$

It is Unsafe.

Second trial

Assume P = 3%

Uniaxial moment capacity of the section about xx axis

$$\frac{P}{fck} = \frac{3}{20} = 0.15$$

$$\frac{d'}{D} = \frac{5}{20} = 0.1$$

$$\frac{Pu}{fck\ b\ D} = \frac{111673.5}{200 \times 23 \times 50} = 0.49$$

$$\frac{Mu}{fck\ b\ D^2} = 0.175$$

Mux_1 = $0.175 \times 200 \times 23 \times 50^2$ = 20125Kg.m

Uniaxial moment capacity about yy axis is same as that of xx axis.

Muy_1 = 20125Kg.m

Puz = $180 \times 23 \times 50$ = **207000**Kg.

$$\frac{Pu}{Puz} = \frac{111673.5}{207000} = 0.54$$

$\frac{Mux}{Mux_1} = \frac{7080.89}{20125} = 0.35$

$\frac{Muy}{Muy_1} = \frac{11365.98}{20125} = 0.56$

(Corresponding to $\frac{Pu}{Puz}$ and $\frac{Muy}{Muy_1}$, $\frac{Mux}{Mux_1} = 0.74$)

Provide reinforcement $= \frac{3}{100} \times 23 \times 50 = 34.5cm^2$

Provide 12 bars of 20mm ∅

Adopt 6mm ∅ lateral ties at 24cm c/c

Column C_{12} : T-Column

Size of column	=	55cm x 35cm x 30cm
Axial load	=	166252Kg
Factored load	=	249393Kg

From E_{24} E_{15}

D.L. B.M. about xx axis	=	6080.995Kg.m
L.L. B.M. about xx axis	=	1496.32Kg.m
		7577.32Kg.m

From E_{24} E_{43}

D.L. B.M. about xx axis	=	-6080.995
L.L. B.M. about xx axis	=	-1498.32
		-7577.32Kg.m

From E_{24} E_{26}

D.L. B.M. about yy axis	=	9153.08
L.L. B.M. about yy axis	=	2871.37
		13024.45Kg.m

Mux = 0

Muy = 1.5 (13024.45) = 19536.68Kg.m

Assume P = 2%

$$\frac{P}{fck} = \frac{2}{20} = 0.1$$

Uniaxial moment capacity of the section about yy axis

$$\frac{d'}{D} = \frac{5}{65} = 0.08$$

$$\frac{Pu}{fck\ b\ D} = 0.639°$$

$$\frac{Mu}{fck\ b\ D^2} = 0.055$$

Muy_1 = 13942.5Kg.m

Assume P = 3%

$$\frac{P}{fck} \quad \frac{3}{20} = 0.15$$

Uniaxial moment of the section about xy axis

$$\frac{d'}{D} = \frac{5}{65} = 0.08$$

$$\frac{Pu}{fck\ b\ D} = 0.639$$

$$\frac{Mu}{fck\ b\ D^2} = 0.12$$

$$Muy = 0.12 \times 200 \times 30 \times 65^2 = 30420 Kg.m$$

$$Puz = 180 \times 30 \times 65 = 351000$$

$$\frac{Pu}{Puz} = 0.7105$$

$$\frac{Mux}{Mux_1} = 0$$

$$\frac{Muy}{Muy_1} = \frac{19536.68}{30420} = 0.64$$

Corresponding to $\frac{Pu}{Puz}$ and $\frac{Muy}{Muy_1}$; $\frac{Mux}{Mux_1} = 0.72$

Provide reinforcement $= \frac{3}{100} \times 30 \times 65 = 58.5 cm^2$

Provide 18Nos of 20mm ∅ bars.

The reinforcement details are given in figure.

DESIGN OF FOUNDATION

INTRODUCTION:

R.C.C. footings are provided to transmit the load of the structure supported by columns to the soil. The design is based on the assumption that the foundation is rigid so that the variation of pressure under the foundation will be linear. The pressure on the soil should not exceed the bearing capacity of soil.

The isolated rectangular footings are designed using elastic design procedure foundation is laid at a depth of 1.38M below the plinth beam.

DATA:

Bearing capacity of the soil	= 15 + 1 M^2
Mix used	= M 200
Steel	= Tor Steel
σ cbc	= 70 kg/cm^2
σst	= 2300 kg/cm^2
Modular ratio	= 13

Design contents K = 0.282, j= 0.903 and Ø = 9.11 30cm thick of levelling course is provided for all the footings keeping the bottom of the levelling 2.38m below ground level.

DESIGN:

FOOTING 1:

FOOTING FOR COLUMN C_1

Size of the column	= 23 cm x 40 cm
Total load from the column	= 33527.95 kg.
Assume self wt. of footing	= 10% = 3352.79 kg.
Total axial load	= 36880.72 kg

MOMENT OF FOOTING:

From the design of column C_1

Moment about xx axis Mux = 687.41 kg m

Moment about yy axis Muy = 1511.48 kg m

Eccentricity in yy axis = $\frac{Mux}{W} = \frac{687.41}{36880.72} = 1.8$ cm

Eccentricity in xx axis = $\frac{Muy}{W} = \frac{1511.48}{36880.72} = 4.1$ cm.

Area of footing required = $\frac{36880.72}{15 \times 1000} = 2.46\ M^2$

Size of footing = 1.6M x 1.6 M

Net upward pressure acting on the footing

$$= \frac{33527.93}{1.6 \times 1.6} = 13096.85 \text{ kg/M}^2$$

$$= 1.31 \text{ kg/cm}^2$$

The footing will project 70.3 cm from xx -axis and 64.1 cm from yy axis

1. BM at X_n XX, $M = \dfrac{1.309 \times 160 \times 70.3^2}{2}$

$$= 51.75 \times 10^4 \text{ kg - cm}$$

Effective depth required $= \sqrt{\dfrac{51.75 \times 10^4}{9.11 \times 40}} = 37.68$ cm

ii. BM at the X_n yy, $M = \dfrac{1.309 \times 160 \times 64.1^2}{2 @}$

$$= 4.3 \times 10^5 \text{ kg cm.}$$

Effective depth required $d = \sqrt{\dfrac{4.3 \times 10^5}{9.11 \times 23}} = \underline{45.32 \text{ cm}}$

iii. Depth required for punching shear

= 1.31 (160 x 160 - 23 x 40)

= 32084 kg.

Area of resisting = 2(23x40)dp x 10 = 126 dp

10 x 126 dp = 32084

Therefore <u>dp = 25.66 cm.</u>

Take overall depth = 55 cm with effective depth of 55 - 8 = 47 cm.

Area of steel parallel to xx axis $= \dfrac{4.3 \times 10^5}{2300 \times 0.903 \times 47}$

$= 4.4\ cm^2$

Provide 12mm dia. bars

No. of rods $= \dfrac{4.4 \times 4}{\pi \times 1.2^2} = 3.89$

Provide 4 rods of 12mm dia.

Area of steel parallel to yy axis $= \dfrac{51.75 \times 10^4}{2300 \times 903 \times 45.8}$

$= \underline{5.44\ cm^2}$

Provide 12mm dia. bars

No. of bars $= \dfrac{5.44}{\pi \times 1.2^2/4} = 4.81$

Adopt 5 bars of 12 mm dia. Min Ast $= \dfrac{0.12}{100} \times 1600 \times 470$

$= 902.4 mm^2$

CHECK FOR BOND:

Sufficient anchor length should be provided.

CHECK FOR SHEAR:

Critical section is taken at a distance euqal to the effective depth from the face of the column.

Shear force at the critical x_n = 1.31 x 17.1 x 160

= 3584.16 kg.

Let the depth of footing at the end be 20 cm.

Depth of the footing at the critical section

$$= 20 + \frac{35}{64.1} \times 17.1 = 29.34 \text{ cm.}$$

Breadth of footing at the critical section

= 23 + 2 x 50.225

= 123.45 cm

$$\text{Shear stress} = \frac{3584.16}{123.45 \times 29.34 \times 0.903}$$

$= 1.09 < 5$ kg|cm²

Hence safe.

The arrangement of column on the footing is as shown in the figure.

FOOTING 2

FOR COLUMN C_2

Size of the column = 23 x 40 cm²

Total load from column = 49552.51 kg

Assume self wt. of footing as = 10%

Total axial load = 54507.76 kg.

MOMENT OF FOOTING:

From the design of column C_2

Moment about xx axis = Mu_x = 10.86 kg.m.

$\simeq$ 11 kg. m

Moment about yy axis = Mu_y = 2095.5 kg.m.

Eccentricity in xx axis = $\frac{Mu_y}{W} = \frac{2095.5}{54507.76} = 3.84$ cm

Eccentricity in yy axis = $\frac{Mu_x}{W} = \frac{11}{54507.76} = 0.02$ cm.

(0.02) It is neglected

Area of footing required = $\frac{54507.76}{15 \times 1000}$

$= 3.64\ cm^2$

Size of footing = 2 m x 2 m

Net upward pressure acting on the footing

$= \frac{49552.51}{2 \times 2} = 12388.13\ kg/m^2$

$= 1.24\ kg/cm^2$

The footing will project 88.5 cm from xx axis and 83.4 cm from yy axis

1. BM at the section xx = $1.24 \times \frac{200.88.5^2}{2}$

= 971199 kg.cm.

2. Effective depth required = d = $\sqrt{\frac{97.12 \times 10^4}{9.11 \times 40}}$

= 51.63 cm.

(ii) BM at the section yy

$$M = \frac{1.24 \times 200 \times 83.4^2}{2} = 86.25 \times 10^4 \text{ kg.m}$$

$$\text{Effective depth reqd. } d = \sqrt{\frac{86.25 \times 10^4}{9.11 \times 23}}$$

$$= 64.16 \text{ cm.}$$

(iii) Depth required for punching shear

$$= 1.24\ (200 \times 200 - 23.40)$$

$$= 48459.2$$

Area of resisting = 2(23 + 40)dp = 126 dp

$$10 \times 126 \text{ dp} = 48459.2$$

$$\text{dp} = \frac{48459.2}{1260} = 38.46 \text{ cm.}$$

Take overall depth = 75 cm with effective depth on

75 - 8 = 67 cm.

$$\text{Area of steel parallel to xx axis} = \frac{86.25 \times 10^4}{2300 \times 0.903 \times 67}$$

$$= 6.19 \text{ cm}^2$$

Provide 12mm dia. bars

$$\text{No. of rods} = \frac{6.19}{\frac{\pi \times 1.2^2}{4}} = 5.48$$

Adopt 6 bars of 12mm dia.

$$\text{?Area of steel parallel to yy axis} = \frac{97.119 \times 10^4}{2300 \times 0.903 \times 65.8}$$

$$= 7.1 \text{ cm}^2$$

Provide 12mm dia. bars

No. of rods $= \dfrac{7.1}{\times \frac{1.2^2}{4}} = 6.28$

Minimum reinforcement $= \dfrac{0.12}{100} \times 2000 \times 670$

$= 1608\ mm^2$

Adopt 8 bars of 12mm dia.

CHEKC FOR BOND:

Sufficient anchor length should be provided.

CHECK FOR SHEAR:

Critical section is taken at a distance equal to the effective depth from the face of the column.

SF at critical section $= 1.24 \times 200 \times 21.5$

$= 5332$ kg.

Let the depth of the footing at the end be 20 cm.

Depth of the footing at the critical section $= 20 + \dfrac{55}{88.5} \times 21.5$

$= 33.36$ cm

Breadth of footing at

the critical section $= 23 + 2 \times \frac{88.5}{55} \times 33.36$

$= 130.36$

Shear stresses $= \frac{5332}{130.36 \times 33.36} = 1.23$ kg/cm²

< 5 kg/cm²

Hence safe.

The reinforcement and other details on the footing is as shown in figure

FOOTING 3 FOR COLUMN C_4

Size of column = 23 cm x 40 cm

Load from column = 51649.78 kg

Assume self wt. of footing as 10 % of axial load

Total axial load = 51649.78 + 5164.978

= 56814.758

Moment about xx axis = Mux = 679.45 kg-m

Moment about yy axis = Muy = 1362.94 kg-m

Eccentricity in xx axis $= \frac{1362.94 \times 100}{56814.758} = 2.39$ cm

Eccentricity in yy axis $= \frac{679.45 \times 100}{56814.758} = 1.195$ cm

Area of footing required $= \frac{56814.758}{15 \times 1000}$

$= 3.787$ m²

Size of footing 2M x 2M

Net upward pressure

acting on the footing = $\frac{51649.78}{200 \times 200}$

= 1.29 kg cm²

The footing will project 81.195 cm from xx axis and 90.89 cm from yy axis

(i) BM at the section XX = $1.29 \times 200 \times \frac{81.195^2}{2}$

= 850449.02 kg.cm.

Effective depth required = $\sqrt{\frac{850449.02}{9.11 \times 23}}$ = 63.71 cm

(ii) BM at the section yy = $1.29 \times 200 \times \frac{90.89^2}{2}$

= 1065668 kg-cm

Effective depth required = $\sqrt{\frac{1065668}{9.11 \times 40}}$

= 50.078 cm

(iii) Depth required for punching shear

= 1.29 (200 x 200 - 40 x 23)

= 50413.2 kg

Area of resisting = 2(23+40)dp = 126 dp

10.126 x dp = 50413.2

Therefore dp = 40.01cm

Take overall depth = 75 cm with effective depth

= 75 - 8 = 67 cm.

Reinforcement parallel to xx axis = $\frac{1065665}{2300 \times 0.903 \times 67}$

$= 7.65 \text{ cm}^2$

Provide 8 bars of 12mm dia.

Area of steel parallel to yy axis

$= \frac{850449.02}{2300 \times 0.903 \times 65.8}$

$= 6.223 \text{ cm}^2$

Provide 7 bars of 12mm dia.

Minimum reinforcement = $\frac{0.12}{1000} \times 2000 \times 670$

Provide 8 bars of 16 mm dia. = 1608 mm^2

CHEKC FOR BOND:

Sufficient anchor length should be provided.

CHECK FOR SHEAR:

Critical section is taken at a distance equal to the effective depth from the face of the column.

SF at the critical section = $1.29 \times 200 \times 23.89$

$= 6163.62$ kg

Let the depth of footing at the end be 20 cm.

Depth of footing at the critical section

$$= 200 + \frac{55}{90.89} \times 23.89$$

$$= 34.456 \text{ cm}$$

Breadth of footing at the critical section

$$= 40 + \frac{78.805}{90.89} \times 67$$

$$\frac{81.195}{90.89} \times 67$$

$$= 157.94 \text{ cm.}$$

Shear stresses $= \dfrac{6163.62}{157.94 \times 34.456 \times 0.903}$

$= 1.254 \text{ kg/cm}^2 < 5 \text{ kg/cm}^2$

Hence safe.

The arrangement of column on footing is shwon in figure.

FOOTING 4 FOR COLUMN C_5

Size of the column = 23 cm x 45 cm

Axial load from the column = 83114.71 kg

Assume self wt. of the footing as 10 % of the axial load.

Total load = 83114.71 + 8311.471

= 91426.18 kg

Moment about xx axis = 88.64 kg-m

Moment about yy axis = 1362.94 kg-m

Eccentricity in the xx axis = $\frac{1362.94 \times 100}{91426.181}$ = 1.49 cm.

Eccentricity in the yy axis = $\frac{88.64 \times 100}{91426.181}$ 0.09 cm

Area of the footing reqd. = $\frac{91426.181}{15 \times 1000}$ = 6.09 m^2

Size of footing = 2.5M x 2.5M

Net upward pressure on the footing = $\frac{83114.71}{250 \times 250}$

= 1.38 kg/cm^2

The footing will project 102.59 cm from xx axis

and 114.99 cm from yy axis.

(1) BM at the section xx axis = $1.33 \times 250 \times \frac{102.59^2}{2}$

= 1749732.7 kg.cm

Effective depth required = $\sqrt{\frac{1749732.7}{9.11 \times 23}}$ = 91.38 cm

(ii) BM at the section YY axis = $1.33 \times \frac{114.99^2}{2} \times 250$

= 219873.9 kg.cm.

Effective depth required = $\sqrt{\frac{219873.9}{9.11 \times 45}}$ = 73.22 cm

(iii) Depth required for punching shear

Punching shear = 1.33 (250 x 250 - 45 x 23)

= 81748.45 kg

Area of resisting = 2(23 + 45) dp

= 136 dp x 10 = 81748.45

dp = 60.11 cm

Take overall depth = 100 cm

With effective depth = 92 cm

Area of steel parallel to xx axis = $\frac{2198273.9}{2300 x 903 x 92}$

= 11.5 cm^2

Provide 12 nos. of 12mm dia. bars

Area of steel parallel to yy -axis =

$\frac{1749732.7}{2300 \times 0.903 \times 90.8}$

Min Ast = $\frac{0.12}{100} \times 2500 \times 920 = 2760\ mm^2$

Provide 10 nos. of 20mm dia. bars

CHECK FOR BOND:

Sufficient anchor length should be provided.

CHECK FOR SHEAR:

The critical section is taken at a distance euqal to the effective depth from the face of the column.

S.F. at the critical section = 22.99 x 250 x 1.33

= 7644.175 kg

Let the depth of footing at the end be 20 cm.

Depth of footing at the critical section

$$20 + \frac{80}{114.99} \times 22.99 = 35.99 \text{ cm}$$

Breadth of footing at the critical section

$$= 45 + \frac{102.59}{114.99} \times 92 + \frac{102.41}{114.99} \times 92$$

= 209 cm.

$$\text{Shear stresses} = \frac{7644.175}{209 \times 0.903 \times 35.94}$$

$$= 1.13 \text{ kg/cm}^2 < 5 \text{ kg/cm}^2$$

Hence safe.

The arrangement of column on footing is as shown in figure .

FOOTING 5 FOR COLUMN 10:

Size of column = 50 cm x 50 cm x 23 cm

Size equivalent square column 42 cm x 42 cm

Axial load from column = 74449 kg

Assume self wt. of footing as 10 % of axial load

Total axial load = 74449 + 7444.9

= 81893.9 kg

Moment about XX axis = 4720.59 kg.m

Moment about YY axis= 7577.32 kg.m

Eccentricity in the xx axis = $\frac{7577.32 \times 100}{81893.9}$ = 5.76 cm

Area of footing required = $\frac{81893.9}{15 \times 1000}$ = 5.46 cm^2

Size of footing = 2.4 x 2.4 M

Net upward pressure acting on the footing

$$= \frac{74449}{240 \times 240} = 1.29 \text{ kg/cm}^2$$

The footing will project 104.76 cm from XX axis and 108.25 cm from YY axis.

1. BM about XX axis = $1.29 \times \frac{210 \times 104.76^2}{2}$ = 2973034.7Kg.cm

   Effective depth required $= \sqrt{\frac{2973034.72}{9.11 \times 42}}$ = 88.12 cm

2. BM about yy axis = $\frac{1.29 \times 210 \times 108.25^2}{2}$

   = 1587211.6 kg.cm.

   Effective depth required $= \sqrt{\frac{1587211.6}{9.11 \times 42}}$ = 64.4 cm

3. Depth required for punching

   Punching shear = 1.29 (240 x 240 - 42 x 42)

   = 72028.44 kg

Area of resisting = 4 x 42 x dp = 168 dp

1680 dp = 72028.44 kg

dp = 42.87 cm

Take overall depth of footing = 100 cm with effective depth = 92 cm.

Area of steel parallel to XX axis

$$= \frac{1587211.6}{2300 \times 0.903 \times 92} = 8.3 \text{ cm}^2$$

Provide 8 bars of 12mm dia.

Area of steel parallel to yy axis $= \frac{297034.72}{2300 \times 0.903 \times 92}$

$= 15.7 \text{ cm}^2$

Min. Ast $= \frac{0.12}{100} \times 2400 \times 920 = 2649.6 \text{ mm}^2$

Provide 14 bars of 12mm dia.

Provide 10 nos. of 20mm dia.

CHECK FOR BOND:

Provide sufficient anchor length.

CHECK FOR SHEAR:

Critical section is taken at a distance equal to the effective depth from the face of the column.

Shear force at the critical section = 16.25 x 1.2 x 210

= 4402.13 kg.

Let the depth of footing at the end be 20 cm.

Depth at critical section = $20 + \frac{80}{108.25} \times 16.25$

= 32 cm.

Breadth at the critical section

$= 42 + \frac{104.76}{108.25} \times 92 + \frac{83.24}{108.25} \times 92$

= 210.27 cm.

Shear stresses = $\frac{4402.13}{210.27 \times 32 \times 0.903} = 0.72\ kg/cm^2$

$< 5\ kg/cm^2$

Hence safe.

The arrangement of column on footing is as shown in figure

FOOTING 6: FOR COLUMN C12

Size of footing = 55cm x 35 cm x 30 cm

Size of equivalent square column 52 cm x 52 cm.

Axial load = 166262 kg.

Assume self wt. of footing as 10 %

Total load = 166262 + 16626.2 = 182888.2 kg.

BM about XX axis = 0

BM about YY axis = 13536.68 kg.m.

Eccentricity in the YY direction = 0

Area required for footing = $\frac{182888.2}{15 \times 1000} = 12.29\ m^2$

Size of footing = 3.5 m x 3.5 M

Net upward pressure acting

on the footing = $\frac{166262}{350 \times 350} = 1.35\ kg/cm^2$

The footing will project 156.4 cm in XX direction and 149 cm in YY direction.

1. BM about XX axis = $\frac{149^2 \times 350 \times 1.35}{2}$

$= 5244986.3$ kg cm

Effective depth required = $\sqrt{\frac{5244986.3}{9.11 \times 52}}$

$= 105.22$ cm

2. BM about YY axis = $1.35 \times \frac{156.4^2}{2} \times 350$

$= 5778901.8$ kg.m.

Effective depth required = $\sqrt{\frac{5778901.8}{9.11 \times 52}}$

$= 110.45$ cm.

3. Depth required for punching

Punching shear = 1.35 (350 x 350 - 52 x 52)

= 161724.6 kg.

Area of resisting = 4X52 dp

$= 208$ dp

2080 dp $= 161724.6$

dp $= 77.75$ cm

Take overall depth of footing = 120 cm with

effective depth = 112 cm.

Area of steel parallel to XX direction

$$= \frac{5778901.8}{2300 \times 0.903 \times 112}$$

= 24.84 cm.

Provide 8 nos. of 20mm dia.

Area of steel parallel to YY direction

$$= \frac{5244986.3}{2300 \times 0.903 \times 110} = 23 \text{ cm}^2$$

Provide 8 nos. of 20mm dia.

$$\text{Min Ast} = \frac{0.12}{100} \times 3500 \times 1120$$

$$= 4704$$

Provide 16 nos. of 20 mm dia.

CHEKC FOR BOND:

Sufficient anchor length should be provided.

CHECK FOR SHEAR:

Critical section is taken at a distance equal to the effective depth from the face of the column.

S.F. at the critical section $= 1.35 \times 44.4 \times 350$

$= 20979$ kg

Let the depth of footing at the end be 25 cm.

Depth at the critical section

$$= 25 + \frac{(120 - 25)}{156.4} \times 44.4$$

$$= \underline{51.96 \text{ cm}}$$

Breadth at the critical section

$$= 52 + \frac{149}{156.4} \times 112 \times 2$$

$$= 265.40 \text{ cm}$$

$$\text{Shear stress} = \frac{20979}{265.40 \times 51.96} = 1.52 \text{ kg/cm}^2$$

$$< 5 \text{ kg/cm}^2$$

Hence safe.

The arrangement of column on the footing and reinforcement details are shown in figure.

STAIRCASE DESIGN

A doglegged stair case is planned and provided as shown in the figure.

Let the rise and tread be taken as 160mm and 250mm respectively. The width of each flight be 1.12 M.

Height of the stair = 2.82 + 0.10 = 2.92 M

Height of each flight = $\frac{2.92}{2}$ = 1.46 M

No. of rise required = $\frac{1.46}{0.160}$ = 9.12

Provide 9 rises in each flight

Actual rise of the step = $\frac{1.46}{9}$ = 16.22 cm

No. of treads in each flight = (9-1) = 8

Space for the treads = 8 x 250 = 2000 mm = 2 M

Space left for passage = 3.00 - 2 M = 1.00 ,M

Loads on each flight:

The effective horizontal span = 2 + 0.23 = 2.23 M

Let the thickness of waist slab = 120 mm.

Weight of the slab (W) on the slope = 0.12x2400

= 288 kg/m²

Dead wt. of horizontal area $W_1 = \frac{W \times \sqrt{R^2 + T^2}}{T}$

$$= \frac{288\sqrt{(0.1622)^2 + (0.25)^2}}{0.25} = 343.305\ kg/m^2$$

Dead weight of stress is given by

$$W_2 = \frac{R \times 2400}{2 \times 1} = \frac{0.1622 \times 2400}{2 \times 1} = 194.64\ kg/m^2$$

Total dead weight = $W_1 + W_2$ = 537.945 kg/m²

Live load = 300kg/m²

Weight of finishing = 10 kg/m²

Total load per metre run = 847.945 kg/m

DESIGN OF WAIST SLAB:

$$M = \frac{Wl^2}{8} = \frac{847.945 \times (2.23)^2}{8} = 527.093\ kg.m.$$

$$d = \sqrt{\frac{52709.3}{9.11 \times 100}} = 7.61\ cm.$$

Provide 12 cm overall thick

Effective depth = 12 - 2 = 10 cm

REINFORCEMENT DETAILS:

$$Mu = 0.87\ fy.\ Ast\ d\ \left(\frac{1 - Ast\ fy}{bd\ fck}\right)$$

5270 9.3 x 1.5 = 0.87 x 4150 x Ast x 10

$$\left(\frac{1 - Ast \times 4150}{100 \times 10 \times 200}\right)$$

Therefore Ast = 2.3 cm²

Use 10mm dia. bars spacing = 34.15 cm.

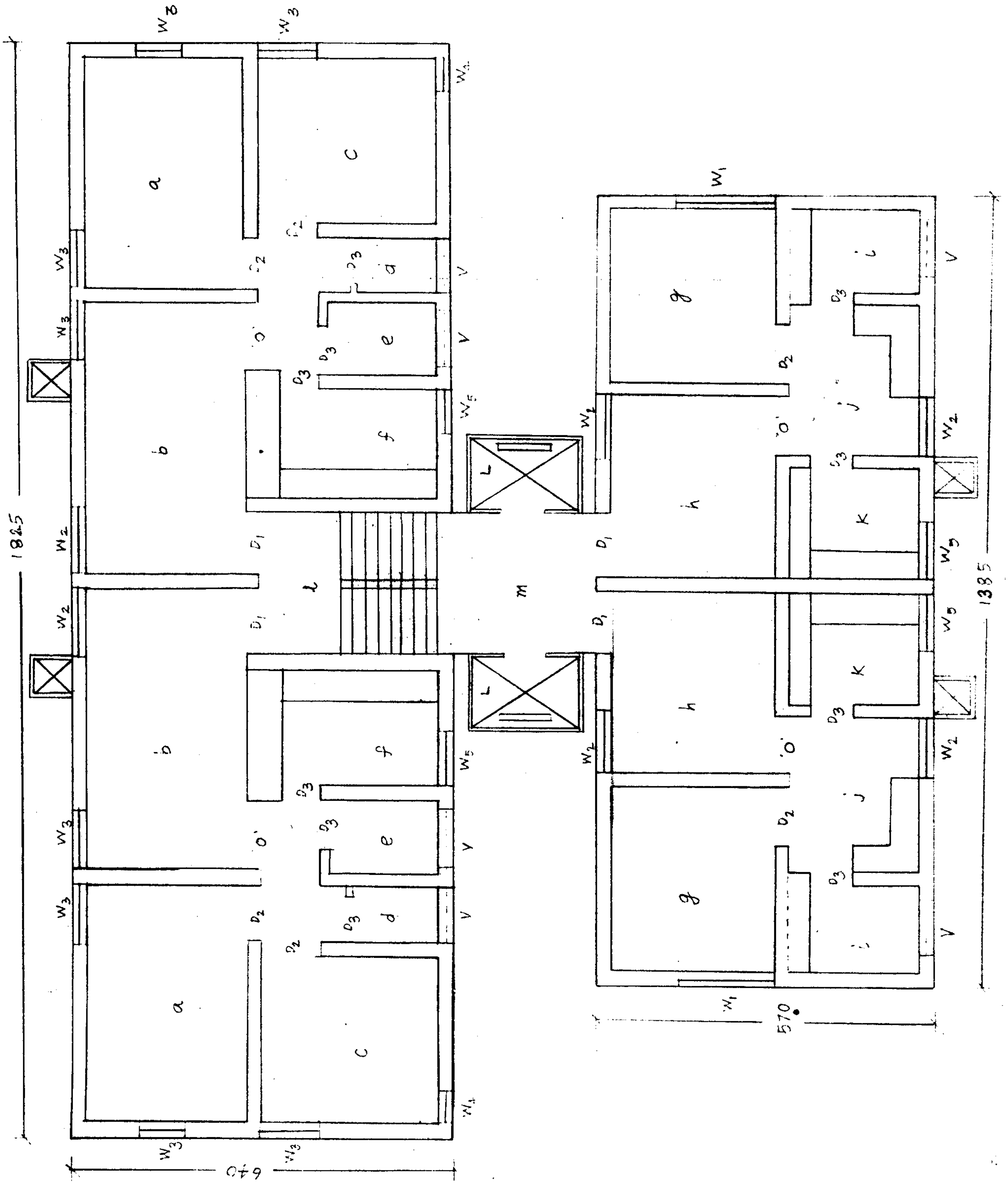
Provide 10 mm dia. bars @ 30 cm c/c

DISTRIBUTION STEEL:

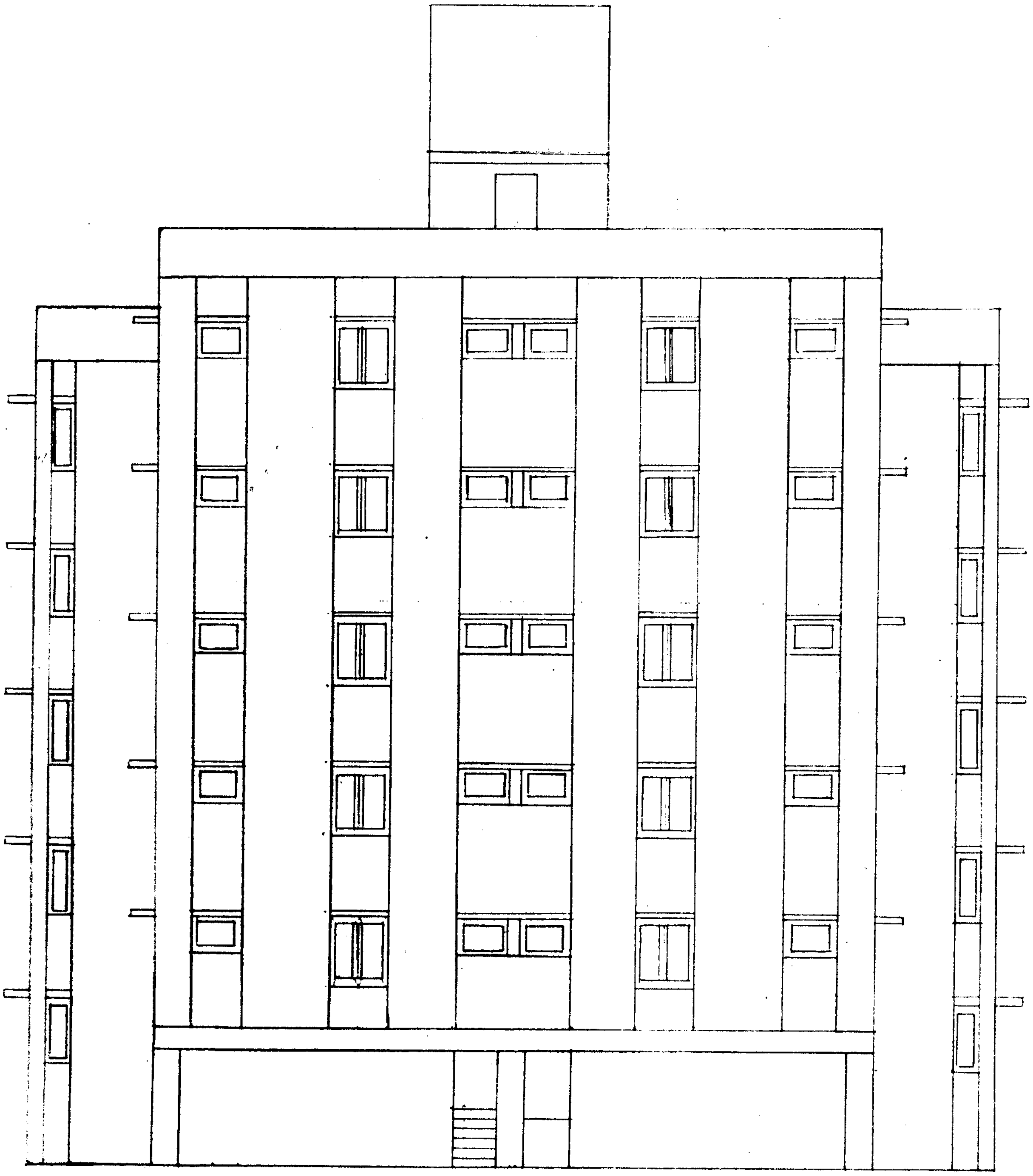
$$Ast = \frac{0.15}{100} \times 100 \times 12$$

$$= 1.8 \text{ cm}^2$$

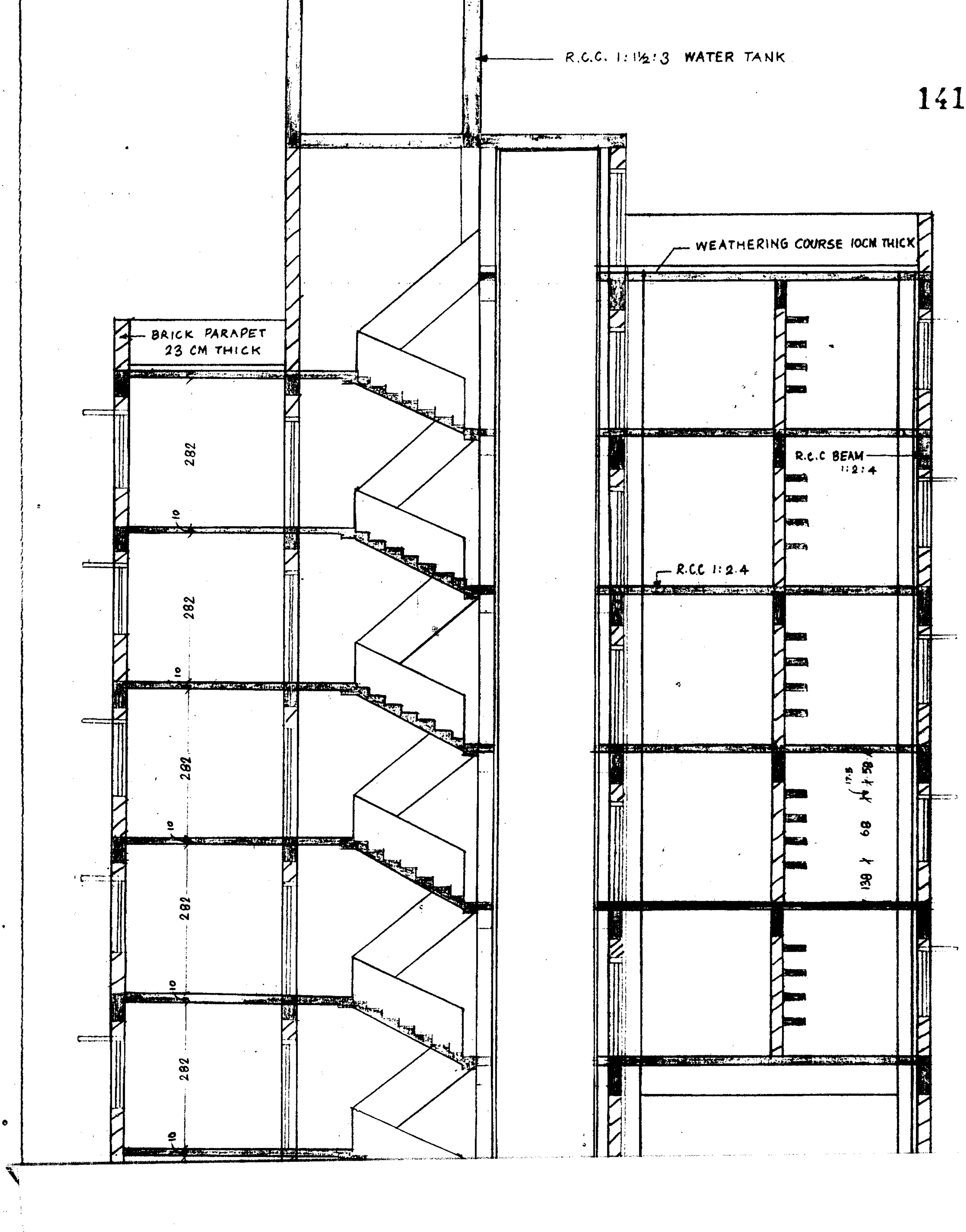
Provide 6 mm dia. bars @ 13 cm c/c



ALL DIMENSIONS ARE IN CM SCALE:1:100

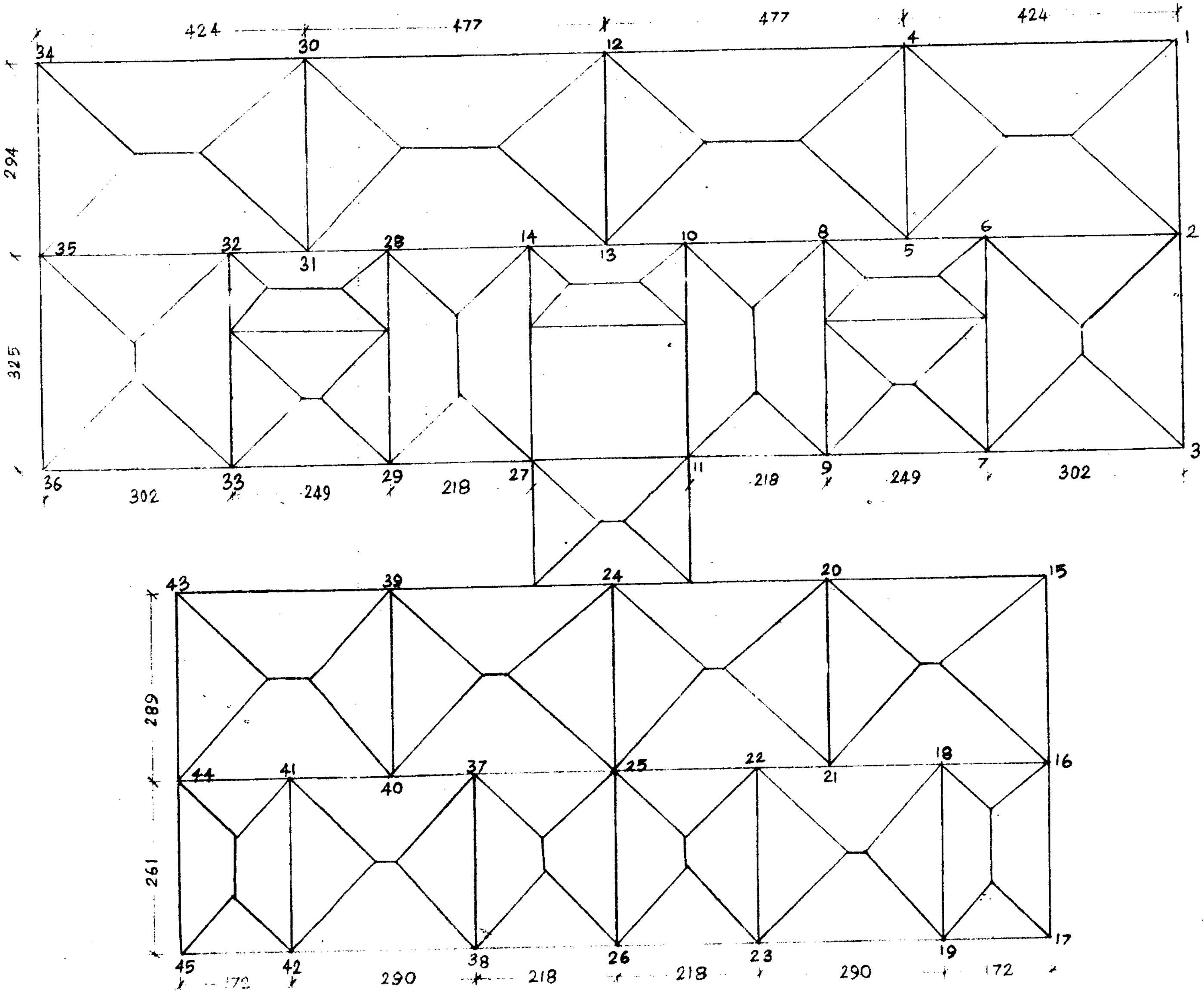


ELEVATION SCALE 1:100


R.C.C. 1:1½:3 WATER TANK
WEATHERING COURSE 10CM THICK
BRICK PARAPET
23 CM THICK
282
10
282
10
282
10
282
10
282
10
R.C.C BEAM
1:2:4
R.C.C 1:2.4
17.5
138 × 68 × 58

LOAD DISTRIBUTION DIAGRAM FOR ALL FLOORS

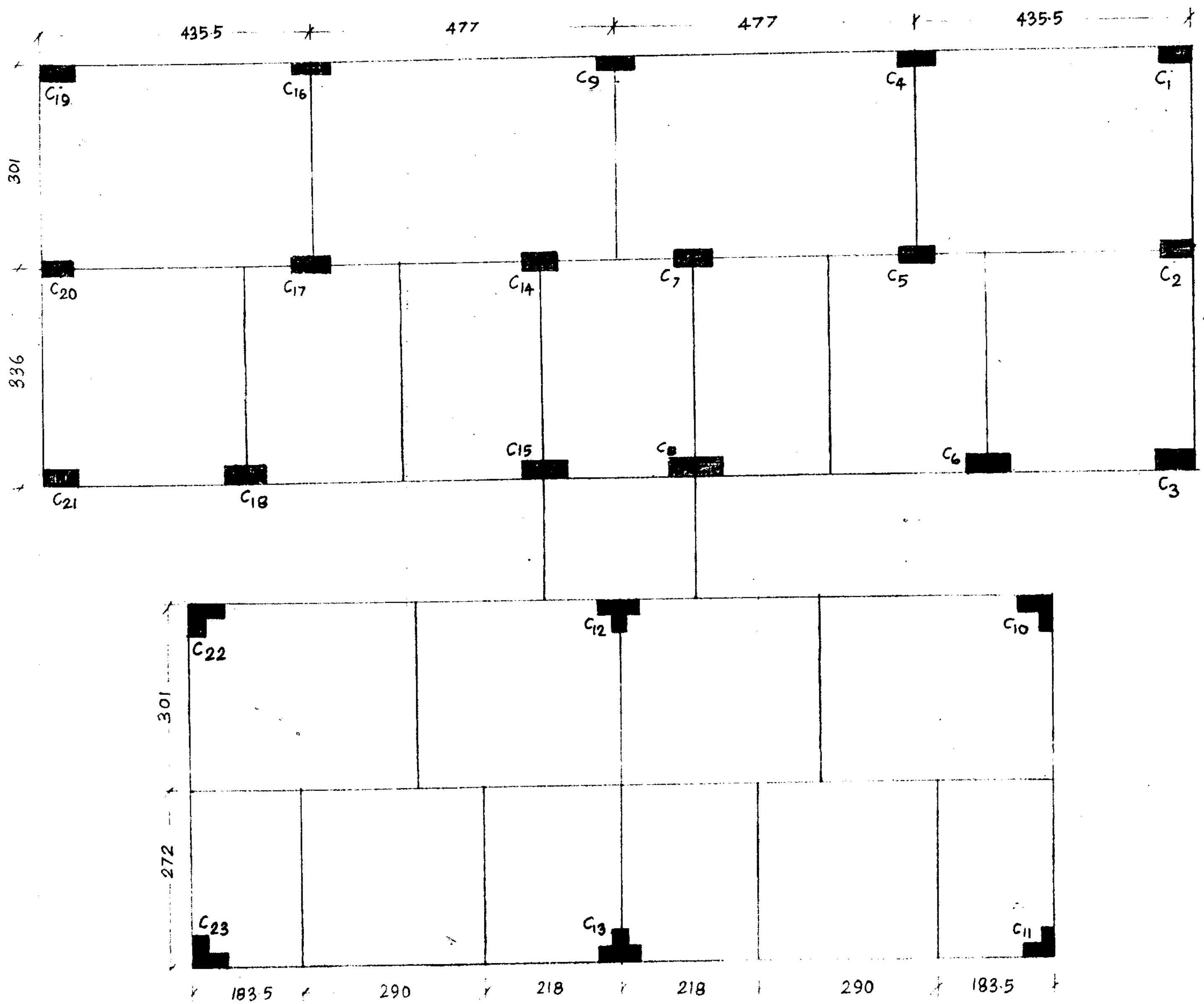
SCALE 1:100 ALL DIMENSIONS ARE IN CM.



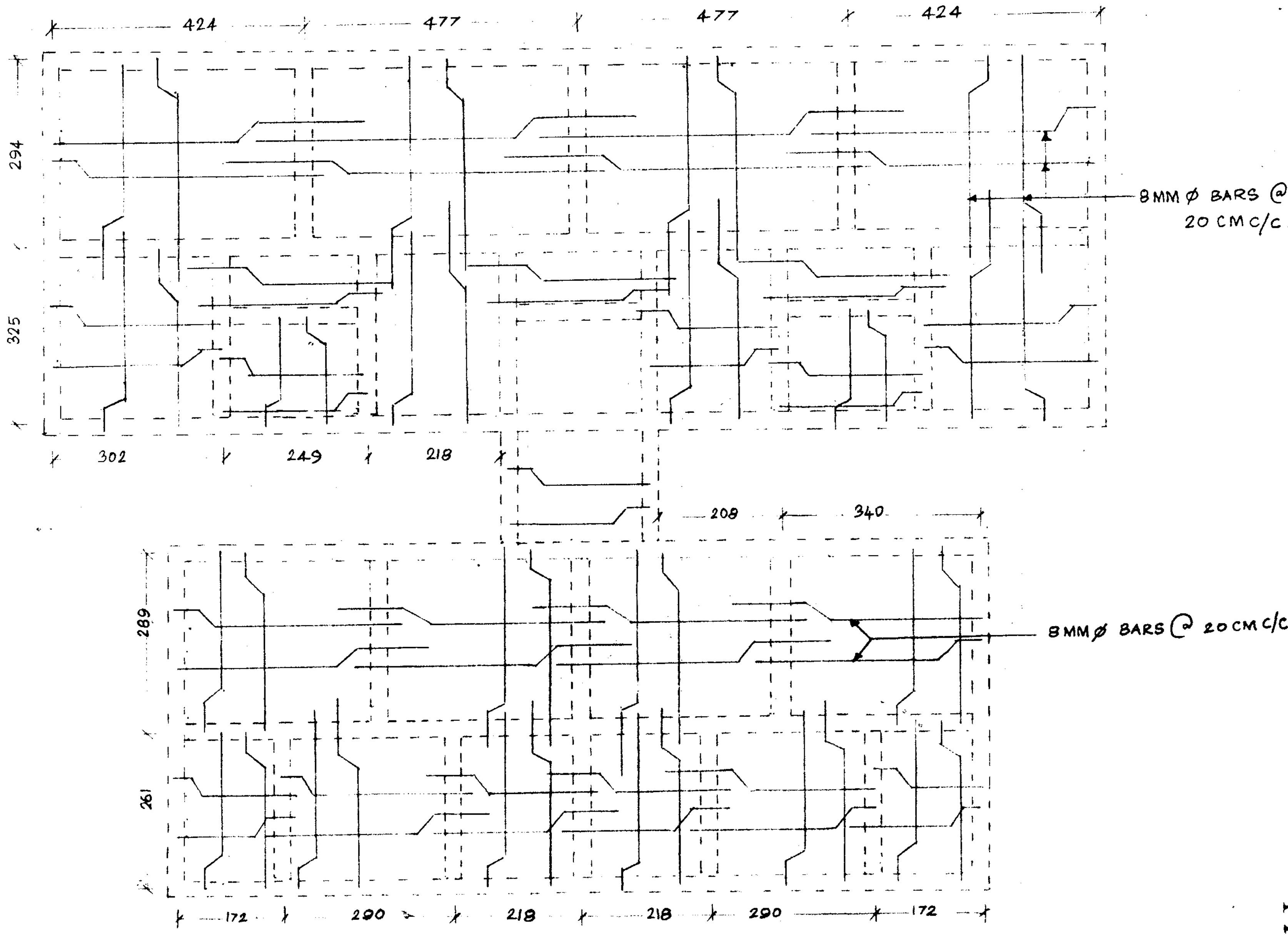
KEY DIAGRAM

ALL DIMENSIONS ARE IN CM

SCALE 1:100

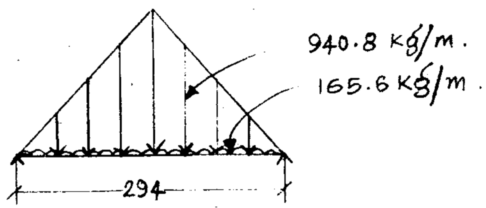


DETAILS OF SLAB REINFORCEMENT

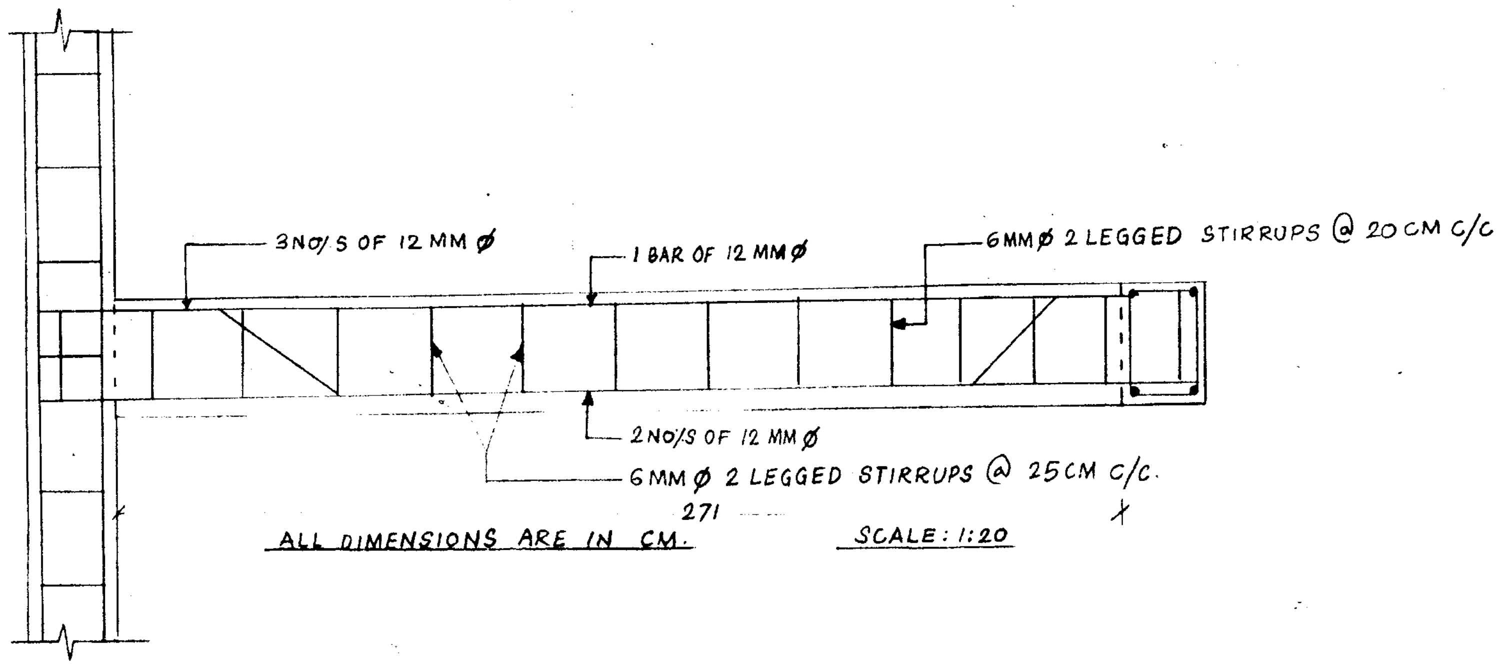


ALL DIMENSIONS ARE IN CM. SCALE 1:100

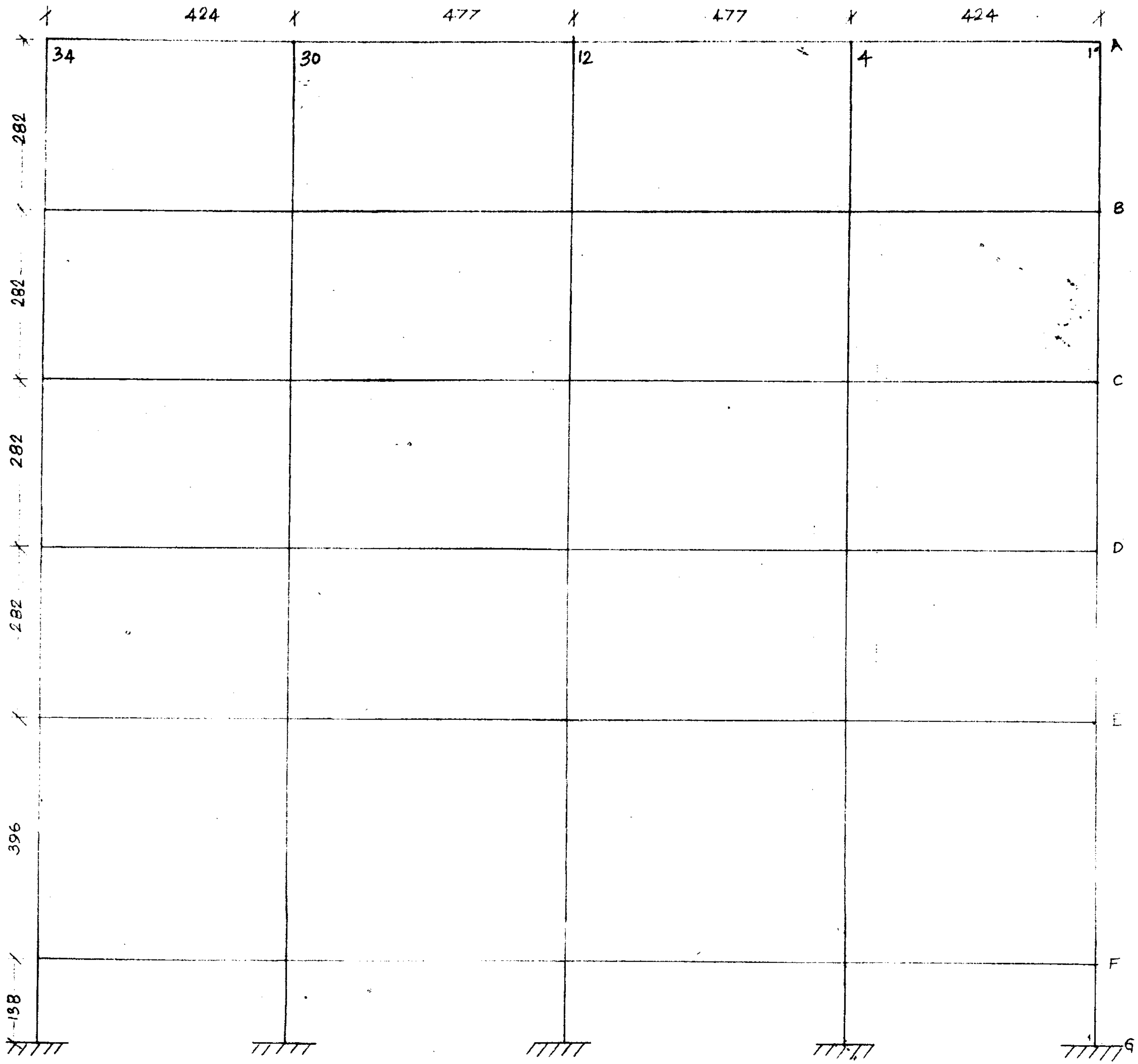
SECONDARY BEAM - SB4



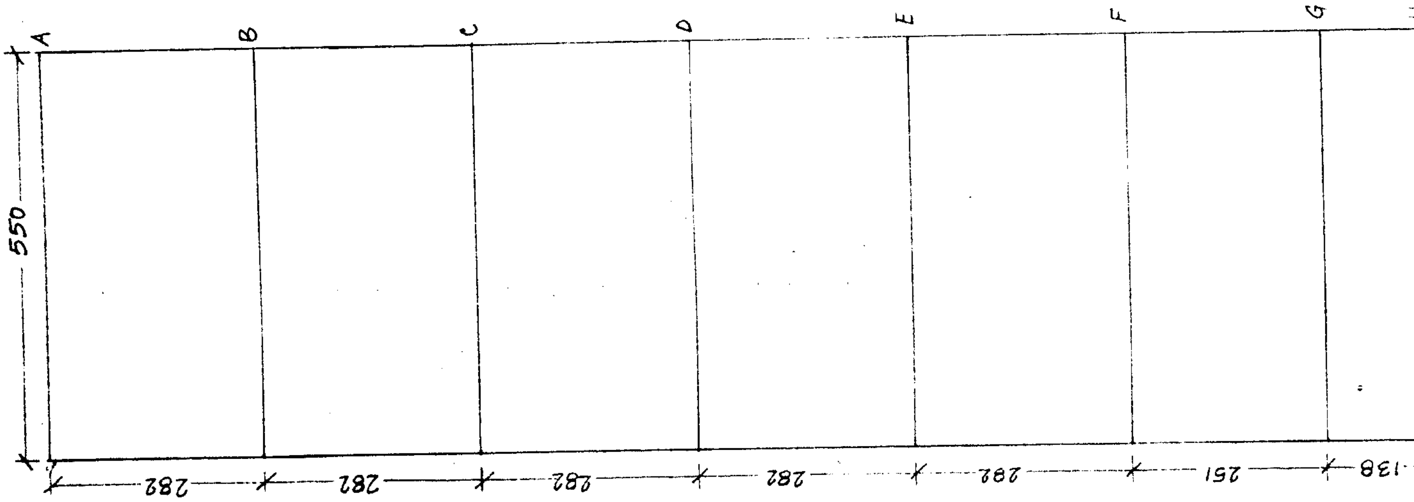
LOADING DIAGRAM. SCALE: 1:80.

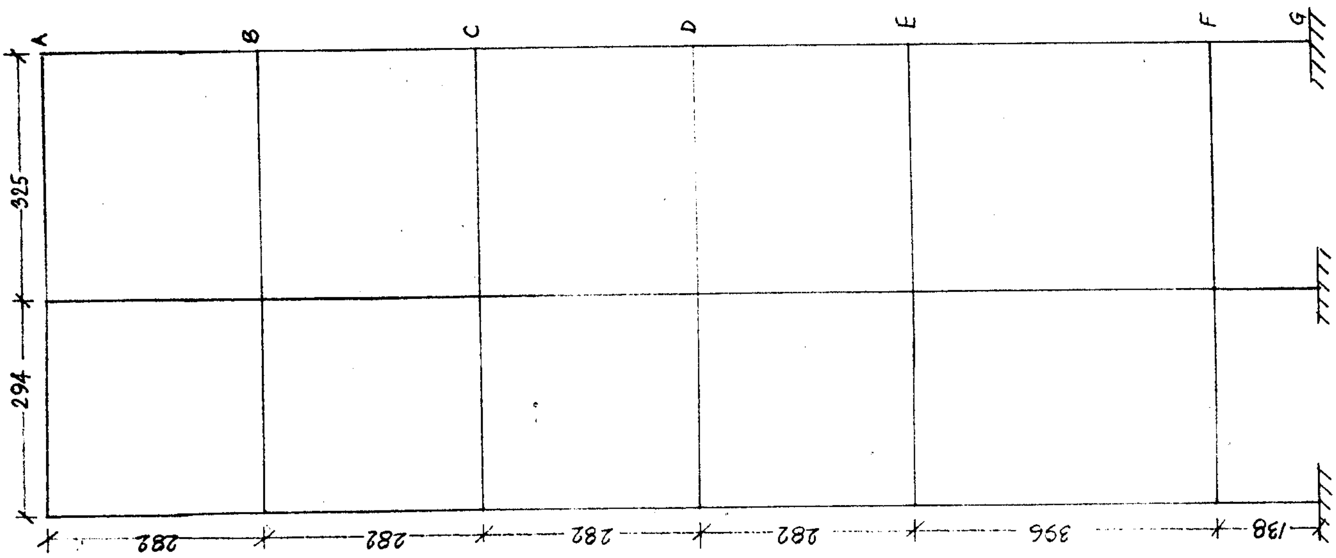


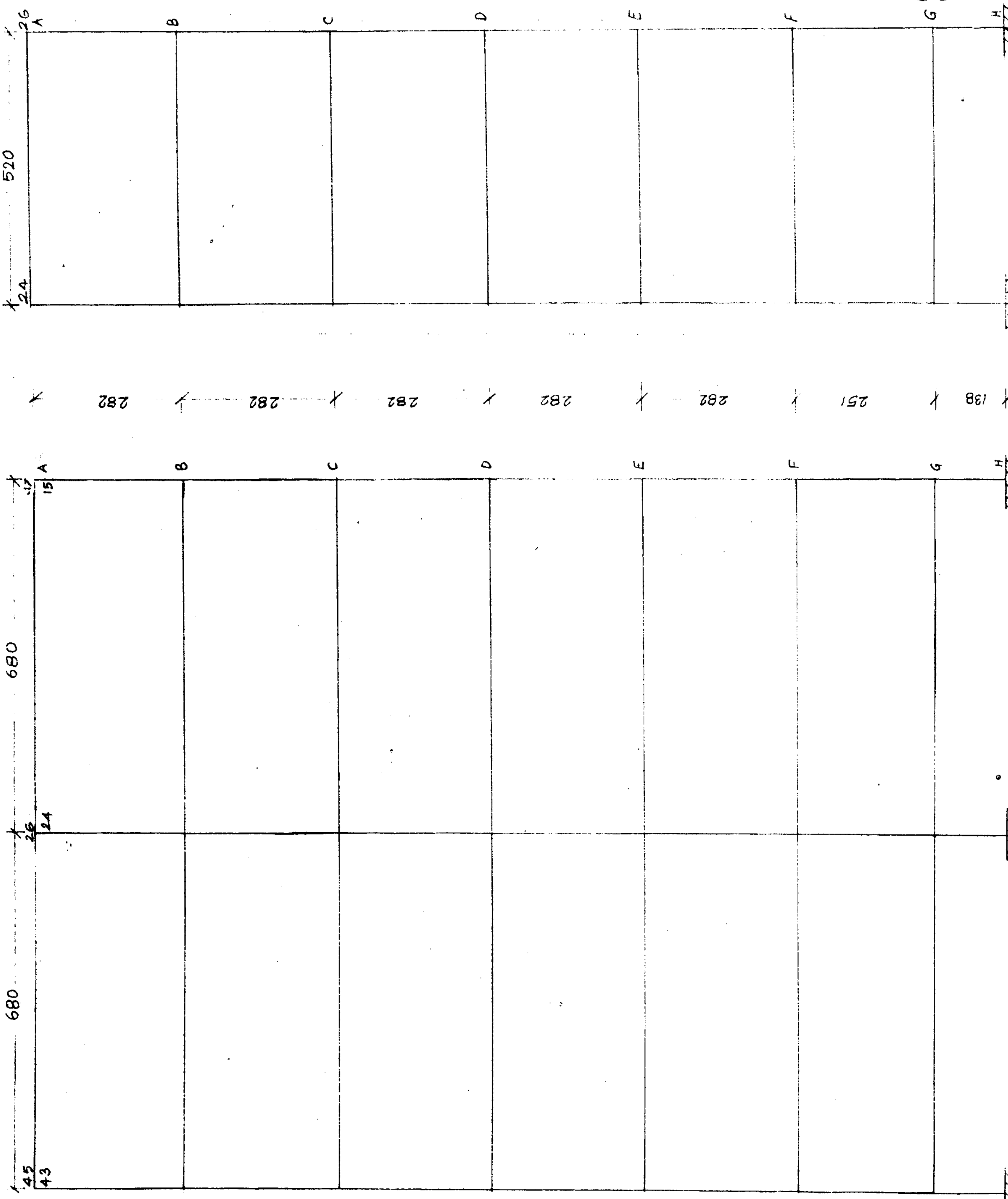
ALL DIMENSIONS ARE IN CM. SCALE: 1:20

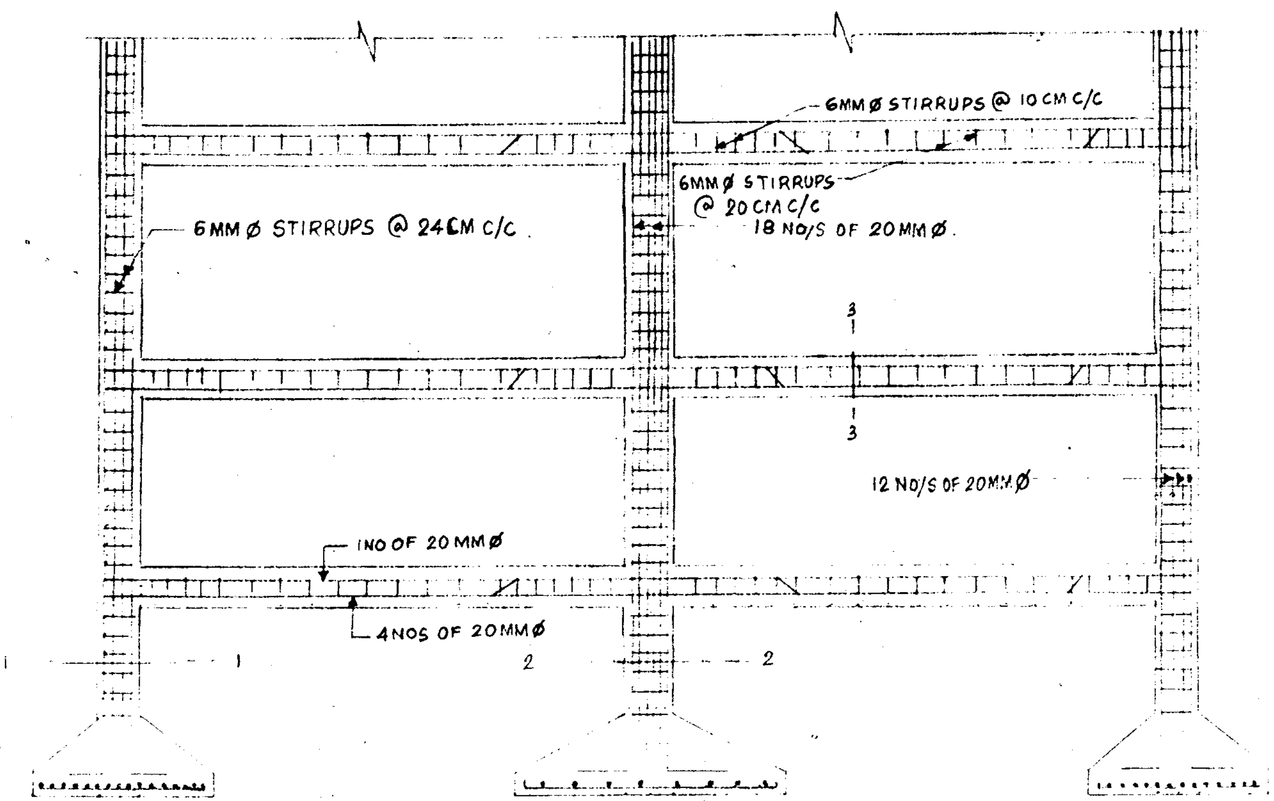


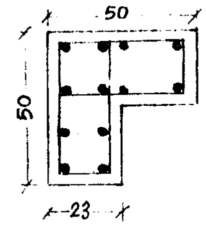
FRAME NO:1. ALL DIMENSIONS ARE IN CM. SCALE:1:100.


A
B
C
D
E
F
G
550
282
282
282
282
282
251
138

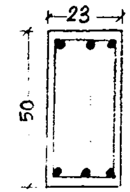

A
B
C
D
E
F
G
325
294
282
282
282
282
396
138


A
B
C
D
E
F
G
H
520
680
680
282
282
282
282
282
251
138

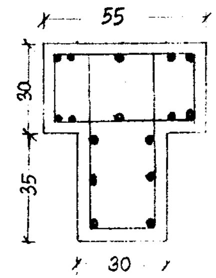




SECTION ON 1-1



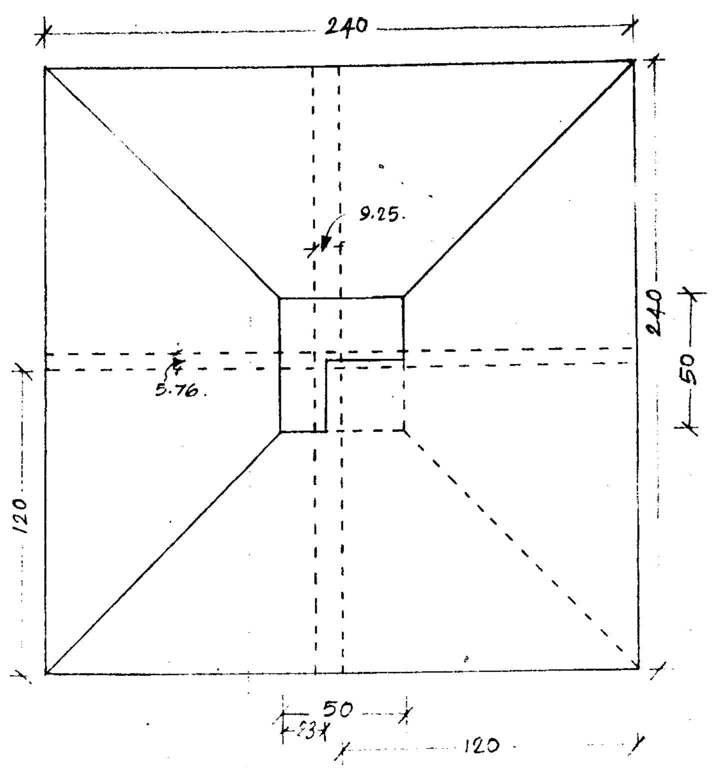
SECTION ON 3-3



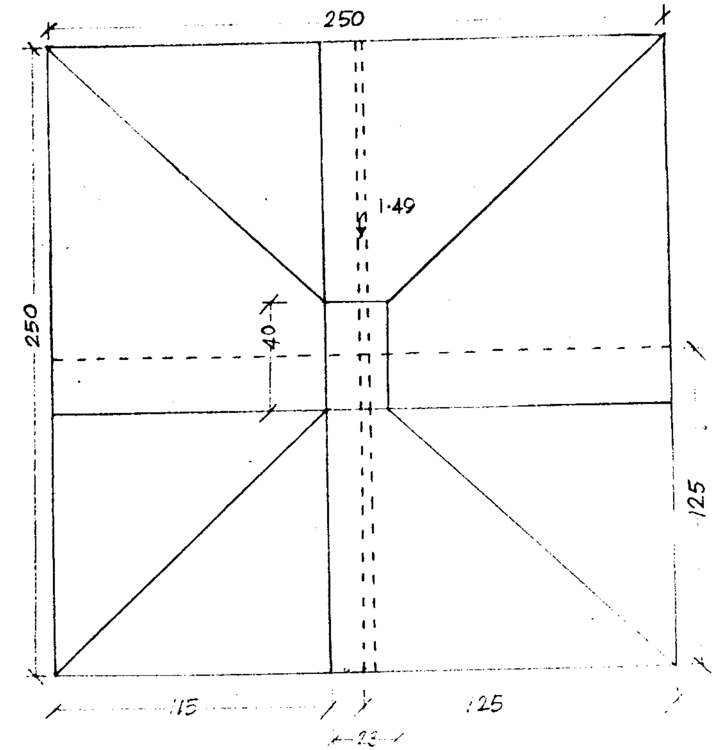
SECTION ON 2-2

REINFORCEMENT DETAILS FOR BEAMS AND COLUMNS:

FOR FRAME NO.5 ALL DIMENSIONS ARE IN CM; SCALE 1:100.

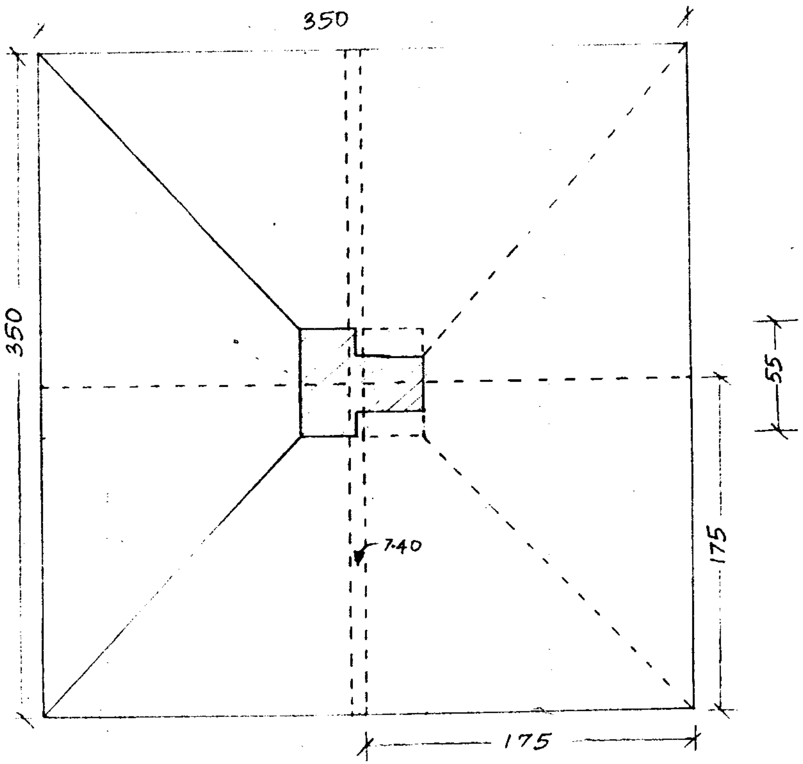


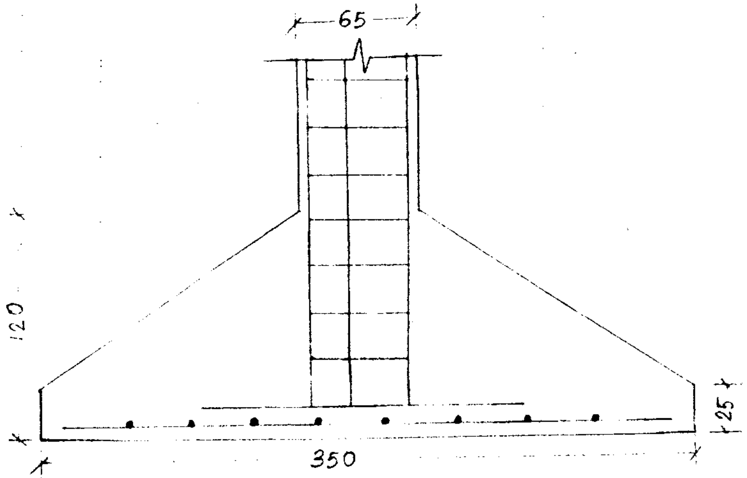
FOOTING FOR COLUMN-C10

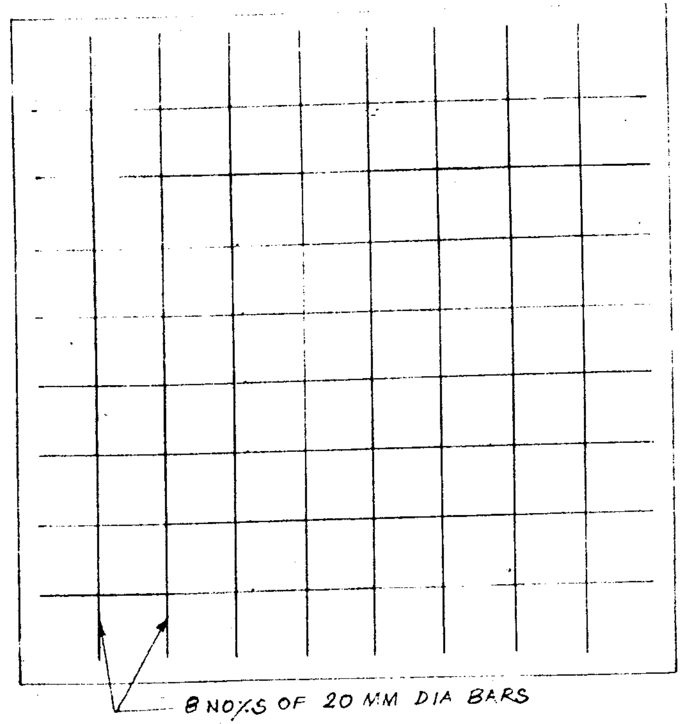


FOOTING FOR COLUMN-C5.

ALL DIMENSIONS ARE IN CM. SCALE: 1:30







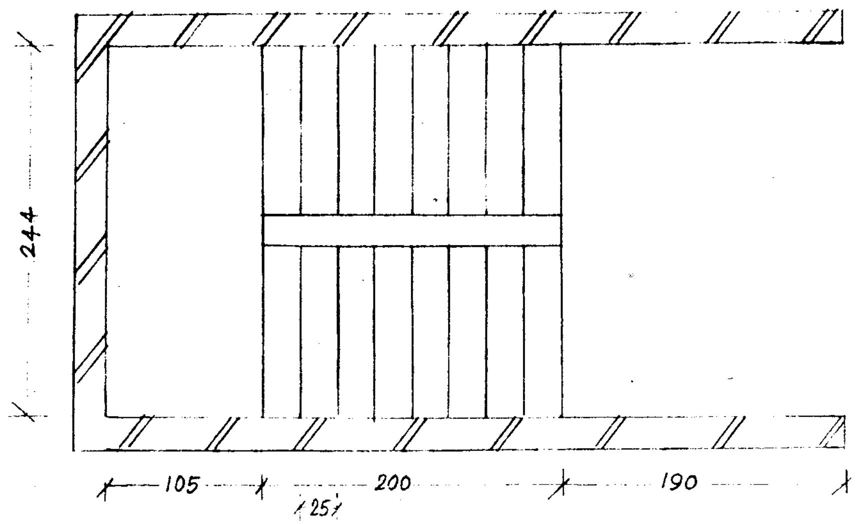
REINFORCEMENT DETAILS

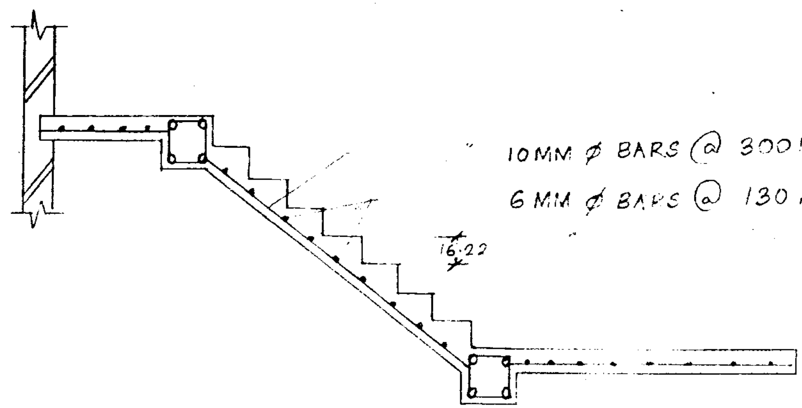
ALL DIMENSIONS ARE IN CM

FOOTING FOR COLUMN - C12 SCALE: 1:40

STAIR CASE

PLAN





10MM ϕ BARS @ 300MM C/C.

6 MM ϕ BARS @ 130 MM C/C.

CROSS SECTION.

ALL DIMENSIONS ARE IN CM SCALE 1:50.

BIBLIOGRAPHY

1. Indian standard code of practice for plain and reinforced concrete IS456 - 1978.

2. Design aids for Reinforced concrete to IS456 - 1978

3. Vazirani, V.N. and Ratwani. N.M. Analysis, Design and Details of concrete structures.

4. Ramamurtham. S - Strength of materials

5. Gopinatha Rao. C.H - Ownership of flat.

6. Reinforced Concrete structures - S. Ramamurtham.

7. Structural Analysis I and II - Vazirani. V.N. and Ratwani N.M

8. Reinforced concrete structures - I.C. Syal A.K. Goel

9. Reinforced concrete structures - P. Dayarathnam.