Planning and Designing of Commercial Complex

Project Work 1989

P-35

Submitted by

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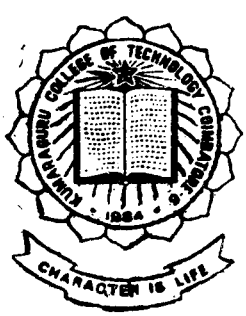

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This is the bonafide record of the project titled "PLANNING AND DESIGNING OF COMMERCIAL COMPLEX" done by

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In partial fulfilment of the requirement of the degree of Bachelor of Engineering in Civil Engineering Branch of Bharathiar University, Coimbatore - 641 046.

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ACKNOWLEDGEMENT

We are deeply indebted to our guide **Prof.P.Jeyapaul, MSc (Engg.).,** whose invaluable guidance and encouragement enabled us to complete this project suceessfully.

We express our gratitude to the staff members of Civil Engineering department who have helped us in this task.

We also take this opportunity to thank our beloved Principal **Prof. Rm. Lakshmanan, B.E, MSc (Engg.).,** for providing us with necessary facilities in the college for the successful completion of this project.

Last but not the least we express our grateful thanks to **Mr.V.Karuppannan, B.E, M.T.P.,** Deputy Director of Town Planning for his valuable suggestions and help to take up this project.

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SYNOPSIS

The object of this project work is to develop independent and creative thinking correlating fundamental and theoritical knowledge that we have occured during our course of study to the practical application in the field.

This project work deals with the planning, structural analysis and design of a commercial complex.

Load from slab to beam is obtained using yield line theory. The analysis of frames, has been carried out by moment distribution method using substitute frame.

Limit state design is adopted for the structural design of all components of the complex, as per IS code regualations and Design Aids for IS 456-1978 is followed.

INTRODUCTION

A commercial complex would be an excellent proposition to the residents of a locality as it would save them the time and effort otherwise spent in travelling. Further-more it would provide them with a wide range of quality goods to choose from. These factors prompted us to plan to design a Commercial Complex which can be set up in a city.

Structural design is a science of designing with economy and elegance, a durable structure which can safely carry the forces and can serve the desired function satisfactorily during its expected service life span.

Selection of site

This commercial complex is proposed to be constructed in the heart of Coimbatore city. The site is situated beside the Big Bazar street and adjacent to Guru Hotel. The site area comes to about one acre. We got the site plan and details from the town planning officer, Coimbatore corporation.

PLANNING OF COMMERCIAL COMPLEX

This four storeyed commercial complex is planned for the occupation of shops, hotel, bank, lodge etc., All the facilities that would be required are kept in mind while planning.

It has been planned to provide shops, hotel and reception room for lodge in the ground floor. 28 shops are provided in the ground floor and the carpet area of each shop is about 12-16 sq.m. Interior dimension for the dinning hall of the restaurant is 8 m x 16 m with a seating capacity of about 75. The access to the lodge has been provided from the reception hall by means of a staircase and a lift.

In the first floor, 32 shops and a bank are provided. In both ground floor and first floor, urinal and toilet facilities are provided seperately for ladies and gents.

In the second and third floors spacious rooms are provided for lodging. The space for rooms are fixed so as to provide more comfortable accomodation. Carpet area of each room is about 24 sq.m. Each room is attached with bath and W.C. Balcony is also provided for each room. The total capacity of the lodge is 100.

An overhead water tank of capacity of 90,000 litres is provided to facilitate water supply to the entire complex.

A spacious lawn is also planned to provide necessary ventillation and to add nice look.

Desired parking place for cars, two wheelers etc., are planned for the easy movement of the vehicles inside the premisses.

Structural Analysis

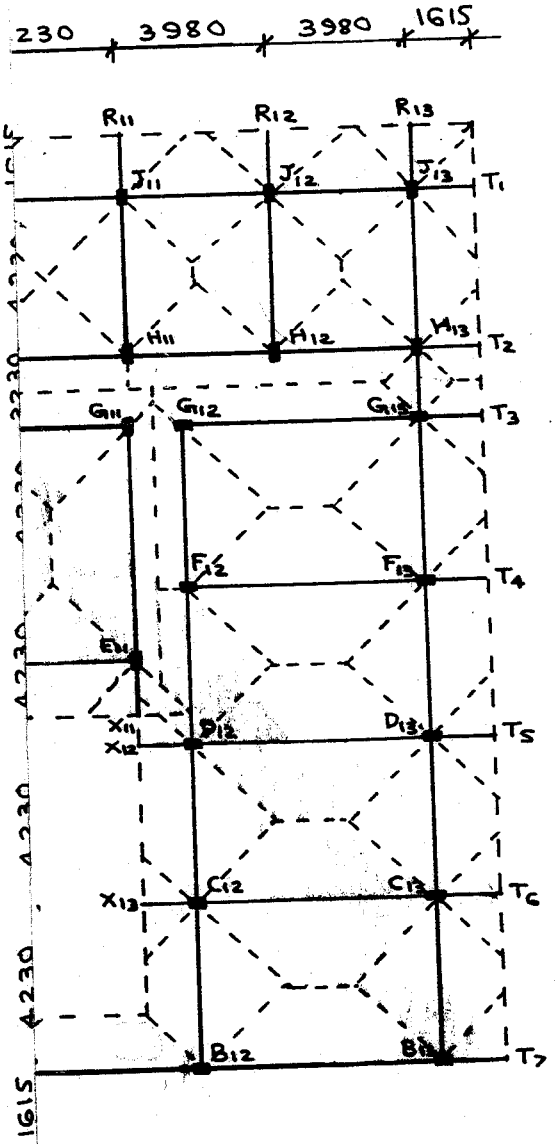
In multistoreyed buildings, concrete members are built monolithically and rigidly which form frames. To design such frames, bending moments and shear force are required.

Loads are transmitted from slab to soil through beams, columns and footings. So to arrive at loads from slab to beams yield line pattern are followed.

To carry out analysis of frames, dimensions of columns and beams are required, so that moment of inertia, stiffness and distribution factor can be computed. Here dimensions of beams and columns are assumed keeping in mind, span of beam, height of column, superimposed load.

To analyse frames, substitute frames are considered, and method of moment distribution is followed. In analysis, dead load is kept constant and live load arrangement is altered bay to bay and maximum moments are taken to design the member.

Limit state method is adopted for design of concrete member. The structures are designed to provide adequate strength, serviceability and durability. Partial safety factors are applied to the loads and moments to provide required safety and serviceability of the structure.



ALL DIMENSION IN mm

ANALYSIS OF MAIN FRAME

FRAME : $P_3\ C_1\ C_2\ X_1$

LOADS :

Floor finishes = 550 N/sq.m.

Unknown partition load = 1500 N/sq.m.

Live load = 5000 N/sq.m.

Beam $C_1\ C_2$; size 230 x 600 mm

Area of shaded portion = 2x1/2x(4+8.23)x4.23/2

= 25.866 sq.mm.

Dead load from slab = 0.15x25.866x25000

= 96998 N.

Load from web = 0.23x(0.6-0.15)x8.23x25000

= 21295 N.

Load from floor finishes= 550x25.866 = 14226 N.

Partition load = 1500x25.866 = 38799 N.

Load on beam from wall = 0.23x3.6x8.23x19200

= 130837 N.

Total dead load = 302155 N.

Uniformly distributed dead load = $\frac{302155}{8.23}$ = 36714 N/m.

Live load = 5000x25.866

= 129330 N.

Uniformly distributed dead load = 302155/8.23 = 36714 N/m

Live load = 5000 x 25.866

= 129330 N

Uniformly distributed live load = 129330/8.23 = 15715 N/m

BEAM P C
5 1

Size : 230 x 600 mm

Area of shaded portion = 2x1/2x1.615x1.615

= 2.608 sq.m

Dead load from slab = 0.15x2.608x25,000

= 9780 N

Load from web = 0.23x(0.6-0.15)x1.618x25000

= 4179 N

Load from floor

finishes = 550x2.608

= 1435 N

Load on beam from wall = 0.115x3.6x1.615x19200

= 12837 N

Total dead load = 28231 N

Uniformly distributed dead load = 28231/1.615

= 17481 N/m

Live load from slab = 5000x2.608

= 13040 N

Uniformly distributed live load = 13040/1.615 = 8075 N/m

BEAM X_1 C_2

Uniformly distributed dead load = 17481 N/m

Unifomly distributed live load = 8075 N/m

DISTRIBUTION FACTOR:

Joint	Member	Stiffness factor	Distribution factor
E_2	$E_2 F_2$	5.182×10^{-4}	0.337
	$E_2 G_2$	5.182×10^{-4}	0.337
	$E_2 E_3$	5.030×10^{-4}	0.326
E_3	$E_3 E_2$	5.030×10^{-4}	0.326
	$E_3 H_3$	5.182×10^{-4}	0.337
	$E_3 I_3$	5.182×10^{-4}	0.337

ANALYSIS FOR DEAD LOADS:

Joint	C1			C2					F	G	H	I
Member	C1 P5	C1 F	C1 G	C1 C2	C2 G	C2 H	C2 I	C2X1	FC1	GC1	HC2	IC2
D.F	-	0.337	0.337	0.326	0.326	0.337	0.337	-	-	-	-	-
F.E.M	+22.797	-	-	-207.230	+207.230	-	-	-22.797	-	-	-	-
Balancing	-	+62.154	+62.154	60.125	-60.125	-62.154	-	-	-	-	-	-
Carry over	-	-	-	-30.063	+30.063	-	-	-	+31.077	31.077	-31.077	-31.077
Bal	-	+10.131	+10.131	+9.801	-9.801	-10.131	-10.131	-	-	-	-	-
Carry over	-	-	-	-4.901	+4.901	-	-	-	+5.066	+5.066	-5.066	-5.066
Bal	-	+1.652	+1.652	+1.598	-1.598	-1.652	-1.652	-	-	-	-	-
Carry over	-	-	-	-0.799	+0.799	-	-	-	+0.826	+0.826	-0.826	-0.826
Bal	-	+0.269	+0.269	+0.260	-0.260	-0.269	-0.269	-	-	-	-	-
Carry over	-	-	-	-0.130	+0.130	-	-	-	0.135	0.135	-0.135	-0.135
Bal	-	+0.044	+0.044	+0.042	-0.042	-0.044	-0.044	-	-	-	-	-
Carry over	-	-	-	-0.021	+0.021	-	-	-	+0.022	+0.022	-0.022	-0.022
Bal	-	+0.007	+0.007	+0.007	-0.007	-0.007	-0.007	-	-	-	-	-
Carry over	-	-	-	-0.004	+0.004	-	-	-	0.004	0.004	-0.004	-0.004
Final moments	22.797	74.257	74.257	-171.311	+171.311	-74.257	-74.257	-22.797	+37.13	+37.13	-37.13	-37.13

ANALYSIS FOR LIVELOADS

CASE I LIVE LOADS ON C C
1 2

Joint		C1				C2			F	G	H	I
Member	C1 P5	C1 F	C1 G	C1 C2	C2 G	C2 H	C2 I	C2X1	FC1	GC1	HC2	IC2
D.F	-	0.337	0.337	0.326	0.326	0.337	0.337	-	-	-	-	-
F.E.M	-	-	-	-88.702	+88.702	-	-	-	-	-	-	-
Bal	-	+29.893	+29.893	+28.917	-28.917	-29.893	29.893	-	-	-	-	-
Carry over	-	-	-	-14.460	+14.460	-	-	-	+14.947	+14.947	-14.947	-14.947
Bal	-	+4.873	+4.873	+4.714	-4.714	-4.873	-4.873	-	-	-	-	-
Carry over	-	-	-	-2.357	+2.357	-	-	-	+2.436	+2.436	-2.436	-2.436
Bal	-	+0.794	+0.794	+0.768	-0.768	-0.794	-0.794	-	-	-	-	-
Carry over	-	-	-	-0.384	+0.384	-	-	-	+0.397	+0.397	-0.397	-0.397
Bal	-	+0.129	+0.129	+0.125	-0.125	-0.129	-0.129	-	-	-	-	-
Carry over	-	-	-	-0.063	+0.063	-	-	-	+0.065	+0.065	-0.065	-0.065
Bal	-	+0.021	+0.021	0.020	-0.020	-0.021	-0.021	-	-	-	-	-
Carry over	-	-	-	-0.010	+0.010	-	-	-	+0.011	+0.011	-0.011	-0.011
Final moments	-	+35.715	+35.715	-71.430	+71.430	-35.715	-35.715	-	+17.853	+17.853	-17.853	-17.853

Case ii
Live load on P5 C1 & C1 X1

Joint		C1				C2			F	G	H	I
Member	C1 P5	C1 F	C1 G	C1 C2	C2 G	C2 H	C2 I	C2X1	FC1	GC1	HC2	IC2
D.F	-	0.337	0.337	0.326	0.326	0.337	0.337	-	-	-	-	-
F.E.M	+10.531	-	-	-	-	-	-	-10.531	-	-	-	-
Bal	-	-3.549	-3.549	-3.433	+3.433	+3.549	+3.549	-	-	-	-	-
Carry over	-	-	-	+1.717	-1.717	-	-	-	+1.775	-1.775	+1.775	+1.775
Bal	-	-0.579	-0.579	-0.560	+0.560	+0.579	+0.579	-	-	-	-	-
Carry over	-	-	-	+0.280	-0.280	-	-	-	-0.289	-0.289	+0.289	+0.289
Bal	-	-0.094	-0.094	-0.091	+0.091	+0.094	+0.094	-	-	-	-	-
Carry over	-	-	-	+0.046	-0.046	-	-	-	-0.047	-0.047	+0.047	+0.047
Bal	-	-0.016	-0.016	-0.015	+0.015	+0.016	+0.016	-	-	-	-	-
Carry over	-	-	-	+0.008	-0.008	-	-	-	-0.008	-0.008	+0.008	+0.008
Bal	-	-0.003	-0.003	-0.003	+0.003	+0.003	+0.003	-	-	-	-	-
Carry over	-	-	-	+0.001	+0.001	-	-	-	-0.001	-0.001	+0.001	+0.001
Final moments	+10.531	-4.241	-4.241	-2.05	+2.05	+4.241	+4.241	-10.531	-2.12	-2.12	+2.12	+2.12

Case III live load on P5 C1 & C1 & C2

Joint		C1				C2			F	G	H	I
Member	C1 P5	C1 F	C1 G	C1 C2	C2 G	C2 H	C2 I	C2X1	FC1	GC1	HC2	IC2
D.F	-	0.337	0.337	0.326	0.326	0.337	0.337	-	-	-	-	-
F.E.M	+10.531	-	-	-88.702	+88.702	-	-	-	-	-	-	-
Bal	-	+26.344	+26.344	+25.84	-28.917	-29.893	-29.893	-	-	-	-	-
Carry over	-	-	-	-14.560	+12.742	-	-	-	+13.172	+13.172	-14.947	-14.947
Bal	-	+4.873	+4.873	+4.714	-4.154	-4.294	-4.294	-	-	-	-	-
Carry over	-	-	-	-2.077	+2.357	-	-	-	+2.436	+2.436	-2.147	-2.147
Bal	-	+0.699	+0.699	+0.677	-0.768	-0.794	-0.794	-	-	-	-	-
Carry over	-	-	-	-0.384	+0.339	-	-	-	+0.350	+0.350	-0.397	-0.397
Bal	-	+0.129	+0.129	+0.125	-0.110	-0.114	-0.114	-	-	-	-	-
Carry over	-	-	-	-0.055	+0.063	-	-	-	+0.064	+0.064	-0.057	0.057
Bal	-	+0.019	+0.019	+0.018	-0.020	-0.021	-0.021	-	-	-	-	-
Carry over	-	-	-	-0.010	+0.009	-	-	-	+0.009	+0.009	-0.011	-0.011
Bal	-	+0.003	+0.003	+0.003	-0.003	-0.003	-0.003	-	-	-	-	-
Carry over	-	-	-	-0.002	+0.002	-	-	-	+0.002	+0.002	-0.002	-0.002
Final moments	+10.531	+32.067	+32.067	-74.668	+70.240	-35.119	-35.119	-	+16.033	+16.033	-17.560	-17.560

FINAL MAXIMUM MOMENTS

Member	C1 E1	C1 F	C1 G	C1 C2	C2 C1	C2 H	C2 I	C2 X1	F C1	C1 C1	H C2	I C2
Dead load moment	+22.797	+74.257	+74.257	-171.311	+171.311	-74.257	-74.257	-22.797	+37.130	+37.130	-37.13	-37.13
live load moment case I	-	+35.715	+35.715	-71.430	+71.430	-35.715	-35.715	-	+17.858	+17.858	-17.858	-17.858
case II	+10.531	-4.241	-4.241	-2.050	+2.05	+4.241	+4.241	-10.531	-2.120	-2.120	+2.120	+2.120
case III	+10.531	+32.067	+32.067	-74.668	+70.24	+35.119	+35.119	-	+16.033	+16.033	-17.560	17.560
Final maximum moments	+33.328	+109.972	+109.972	-245.979	+245.979	-109.972	-109.972	-33.328	+54.987	+54.987	-54.987	-54.987

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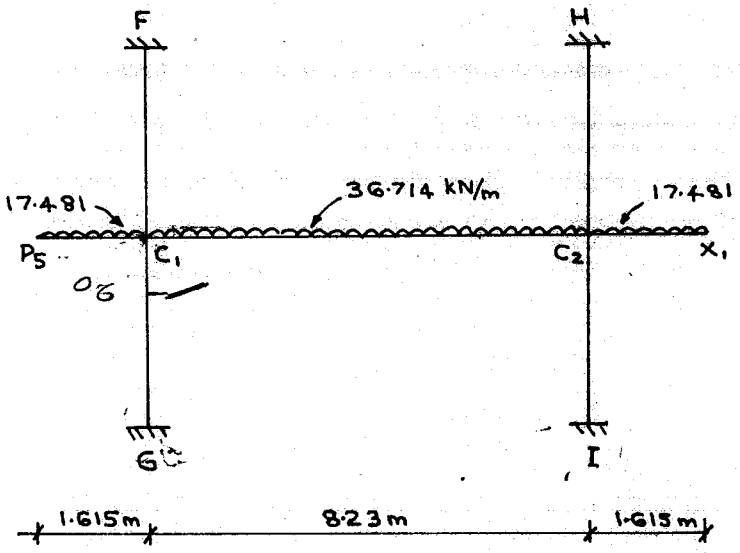
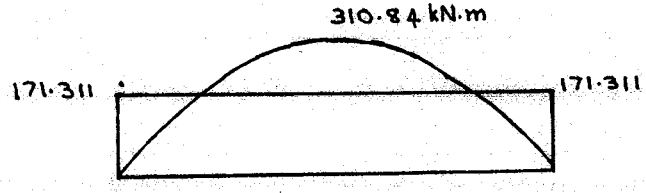
DEAD LOAD MOMENTS





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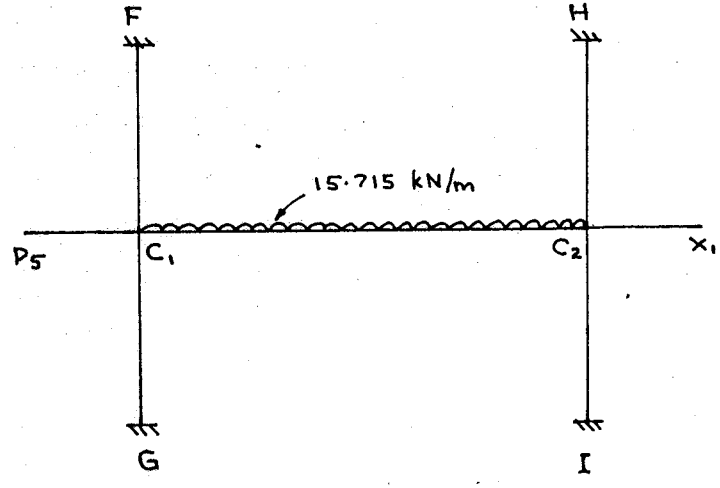
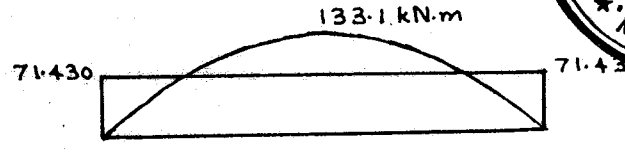
LIVE LOAD MOMENTS

CASE-(i)





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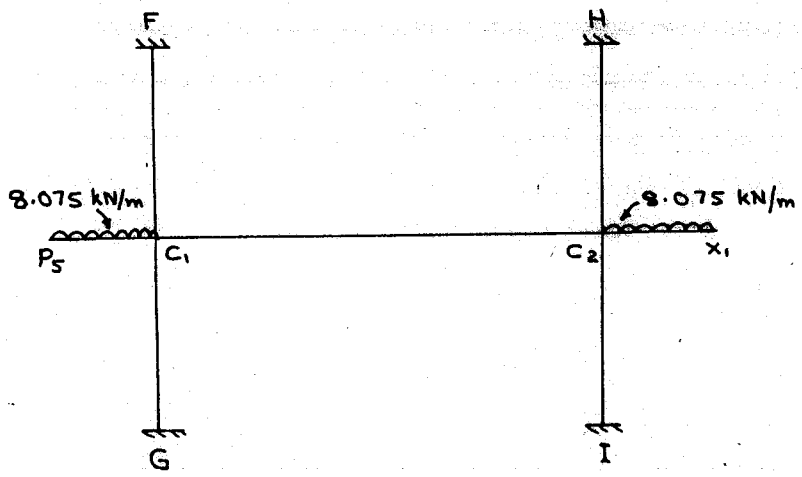
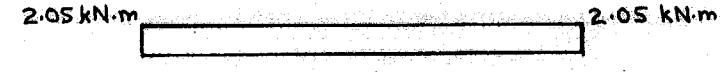
CASE -(ii)





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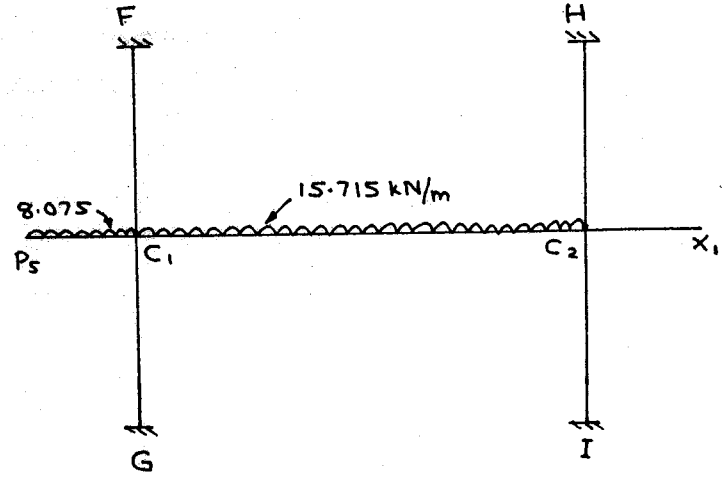
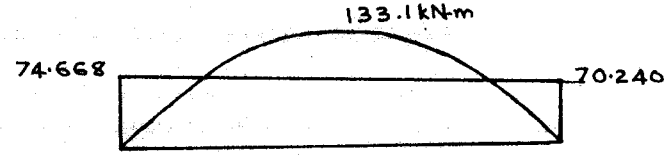
CASE -(iii)





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ANALYSIS OF SECONDARY FRAME

FRAME P10 J1 J2 J3 J4

BEAM P10 J1 Size 230 x 600 mm

Area of shaded portion = 2.608 sq.m.

Load from slab = 0.150 x 2.068 x 25000

= 9780 N.

Load from web = 0.23 x 0.45 x 1.615 x 25000

= 4179 N.

Load from floor finishes = 550 x 2.608

= 1435 N.

Load on beam from wall = 0.115 x 3.6 x 1.615 x 19200

= 12837 N.

Total dead load = 28231 N.

Uniformly distributed dead load $= \frac{28231}{1.615} = 17481$ W/m.

LIve load from slab = 5000 x 2.608

= 13040 N.

Uniformly distributed live load $= \frac{13040}{1.615} = 8075$ N/m.

BEAM J1 J2 Size 230 x 600 mm.

Area of shaded portion = 23.616 sq.m.

Load from slab = 0.15x23.616x25000

= 88560 N.

Load from web = 0.23x0.45x8.23x25000

= 21295 N.

Load from floor finishes = 550x23.616

= 12989 N.

Partition loads = 1500x23.616

= 35424 N.

Load on beam from wall = 0.23x3.6x8.23x19200

= 130837 N.

Total dead load = 289105 N.

Uniformly distributed dead load $= \frac{289105}{8.23} = 35128$ N/m.

Live load from slab = 5000 x 23.616

= 118080 N.

Uniformly distributed live load $= \frac{118080}{8.23} = 14348$ N/m.

BEAM J2 J3 Size 230 x 300 mm

Area of shaded portion = 1.496 sq.m.

Load from slab = 0.15 x 1.496 x 25000

= 5610 N.

Load from floor finishes = 550 x 1.496

= 823 N.

Load from web = 0.15x0.23x1.73x25000

= 1492 N.

Load on beam from wall = 0.115x3.6x1.73x19200

= 13751 N.

Total dead load = 21676 N.

Uniformly distributed dead load = $\frac{21676}{1.73}$

= 12529 N/m.

Live load from slab = 5000 x 1.496

= 7480 N.

Uniformly distributed live load = $\frac{7480}{1.73}$ = 4324 N/m.

BEAM J3 J4 Size 230 x 300 mm.

Area of shaded portion = 8.696 sq.m.

Load from slab = 0.15 x 8.696 x 25000

= 32610 N.

Load from web = 0.23x0.15x4.23x25000

= 3648 N.

Load from floor finishes = 550 x 8.696

= 4783 N.

Partition loads = 1500x8.696 = 13044 N.

Load on beam from wall = 0.23x3.6x4.23x19200

= 67247 N.

Total dead load = 121332 N.

Uniformly distributed dead load = $\frac{121332}{4.23}$ = 28684 N/m.

Live load from slab = 5000 x 8.696

= 43480 N.

Uniformly distributed live load = $\frac{43480}{4.23}$ = 10279 N/m.

Joint	Member	Stiffness factor	Distribution factor
J1	J1 P	5.182×10^{-4}	0.336
	J1 Q	5.182×10^{-4}	0.336
	J1 J2	5.030×10^{-4}	0.328
J2	J2 J1	5.030×10^{-4}	0.274
	J2 R	5.182×10^{-4}	0.281
	J2 S	5.182×10^{-4}	0.281
	J2 J3	2.991×10^{-4}	0.164
J3	J3 J2	2.991×10^{-4}	0.482
	J3 T	1.295×10^{-4}	0.209
	J3 U	1.295×10^{-4}	0.209
	J3 J4	0.6117×10^{-4}	0.1

DESIGN OF INTERMEDIATE FLOOR SLAB

SLAB : S1

Size : 8.23mx4.23m Ly/Lx = 8.23/4.23 = 1.95<2

The slab is designed as a two - way slab

i) Effective depth:

Overall depth of slab = 150mm

Assuming 10mm dia bars,15mm clear cover,

Effective depth along shorter span = 150-15-10/2

= 130mm

Effective depth along longer span = 130-10

= 120mm

ii) Loads:

Floor finishes	= 550 N/sq.m
Self weight of slab	= 0.15x25000=3750 N/sq
Unknown partition load	= 1500 N/sq.m
Live load	= 5000 N/ sq.m
Total load (w)	= 10800 N/sq.m

iii)Bending moment:

From code is :456-1978 , Appendix C, Table no 22

1) Bending moments in shorter span = $Mx=Xx\ Wlx^2$

2) Bending moments in longer span = $My=Xx\ Wlx^2$

Where

w is the total design load per unit area

Lx,Ly are the lenghts of shorter span and longer span respectively

Xx, Xy are coefficients as given in table : 22

Case no	Type of panel and moments considered	short span coefficients Xx	long span coefficients Xy
1.	Interior panles : Negative moments at Continous edge	0.064	0.032
	Positive moment at mid span	0.0482	0.024

1) Shorter span (-Ve) Bending moments at continuous edge

$= Xx\ Wlx^2$

$= 0.064 \times 10800 \times (4.23)^2$

$= 12367.57\ N\ .m$

2) Shorter span (+Ve) Bending momets at mod span

$= Xx\ Wlx^2$

$= 0.0482 \times 10800 \times 4.23^2$

$= 9314.33\ N\ m$

3) Longer span (-Ve) Bending moments at continous edge

$= Xy.wlx^2$

$= 0.032 \times 10800 \times 4.23^2$

$= 6183.8\ N\ m$

4) Longer span (+Ve) Bending moments at mid span

$= Xy.w.lx^2$

$= 0.024 \times 10800 \times 4.23^2$

$= 4637.84N\ m$

Requirement of steel (shorter span)

Steel requirement at the mid span (+Ve) moment

Limiting Moment for the section

$Mu,lim = 0.36\ Xumax/d\ (1-0.42x\ Xumax/d)bd^2\ fck$

for Fy = 415 N/sq.m, Xumax/d= 0.48

$Mu,lim = 0.36x0.48(1-0.42x0.48)x1000x(130)^2x15$

$= 34.97x10^6\ N\ -mm$

Moments at mid-span Mu = 1.5x9314.33=13971.9N-m

Mu is less than Mu,lim:

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

$Mu/bd^2 = 0.87\ fy\ pt/100\ (1-pt/100.fy/fck)$

$$\frac{13971.9x1000}{1000x130^2} = 0.87x415xpt/100(1-pt/100x415/15)$$

$$0.827 = 3.611\ Pt - 0.9992Pt^2$$

$$Pt^2 - 3.614\ Pt + 08274 = 0$$

Solving

$Pt = 0.246\ \%$

$$Ast = \frac{0.246}{100}x130x1000$$

$= 319.8\ sq.mm$

Provide 10 mm dia HYSD bars at 220 mm c/c

steel requirement at continous edge (-Ve) moment

$Mu = 1.5x12367.57x10^3 = 18.55x10^6\ N\ -mm$

$$\frac{18.551x10^6}{1000x130^2} = 3.611\ Pt - 0.999\ 2\ Pt^2$$

$Pt - 3.614Pt + 1.099 = 0$

solving

$$Pt = 0.3352\ \%$$

$$\text{At Support Ast} = \frac{0.3352}{100} \times 130 \times 1000 = 435.76 \text{ sq.mm}$$

Alternate bars provided at mid span are cranked at the support additional steel required $= 435.76 - \frac{n \times 10^2}{4} \times \frac{1000}{440}$

$$= 257.3 \text{ sq.mm}$$

Provide extra 12 mm dia HYSD bars at 440 mm c/c

Requirement of steel in longer span

steel requirement at mid span (+Ve) moment

$$Mu = 1.5 \times 4637.84 \times 10^3 = 6.956 \times 10^6 \text{ N - mm}$$

$$\text{therefore} \quad \frac{6.956 \times 10^6}{1000 \times 120^2} = 3.61\ Pt - 0.9992\ Pt^2$$

$$Pt^2 - 3.614\ Pt + 0.483 = 0$$

Solving

$$Pt = 0.139\ \%$$

therefore Ast $= 0.139/100 \times 1000 \times 120 = 166.8$ Sq. mm

But minimum reinforcement = 0.12 % of cross sectiona area

$$= 0.12/100 \times 1000 \times 150$$

$$= 180 \text{ sq.mm}$$

Provide 10 mm dia HYDS bars at 300 mm c/c

Steel requirement at continuous edge (-Ve) moment

$$Mu = 1.5 \times 6183.8 = 9275.7 \times 10^{3} \text{ N - mm}$$

$$\frac{9275.7 \times 10^{3}}{1000 \times 120^{2}} = 3.611\ Pt - 0.9992\ Pt^{2}$$

$$Pt^{2} - 3.614\ Pt + 0.644 = 0$$

Solving

$$Pt = 0.188\ \%$$

Ast = 0.188/100x130x1000= 225.6mm sq.

Alternate bars provided at mid span are cranked at support additional steel reniforcement

$$= 225.6 - \frac{U \times 10^{2}}{4} \times \frac{1000}{600}$$

$$= 94.7 \text{ sq mm}$$

Provide extra 10 mm HYSD bars at 600 mm C/C

Reinforcement at edge strips:

As per code IS 456-1978

Minimum reniforcement required is 0.12 % of total cross sectional area.

Steel are required for long span

= 0.12/100x1028.8x150=185 sq mm

Provide 4 no.s of 8 mm G 8 mm q HYSD bars steel area required for shorter span = 0.12/100x528.8x150= 95 sq.mm

Provide 2 no.s of 8 mm G HYSD bars

Since the slab is continuous over all edges, torsion

reniforcement is not necessasary

Check for bear:

Maximum sbear force V = 0.6 W

= 0.06x10800=6480 N

$$= \frac{Vu}{bd} = \frac{6480 \times 1.5}{1000 \times 130} = 0.075 \text{ Mpa}$$

But Tc min = 0.35Mpa

Therefore the slab is safe in sbear.

SLAB : S_2

1) Size : 4.23 x 4.23 m lx/ly = 1

Two- way slab with one edge discontinuous.

2) Load : 10800 N sq.mm

3) Moments :

Factored bending moments at mid span (+Ve)

$= 8.117 \times 10^6$ N-mm.

Factored bending moments at

continuous edge (-Ve)

$= 10.725 \times 10^6$ N-mm

4) Reniforcement in middle strip

Provide 10 mm Q HYSD bars @ 300 mm c/c at mid span

Provide extra 10 mm Q HYSD bars @ 600 mm c/c at continuous edge.

Reniforcement in edge strip

provide 2 no.s of 8 mm Q HYSD bars for each span .

Torsion Reniforcement

Provide 2 layers of 8 mm Q HYSD bars @ 350 mm c/c , at the corner contained by discontinuous edge

SLAB : S
3

1) Size : 4.23 x 4.23 m

Two-way slab with all edge continuous.

2) Load : 10800 N/ m sq.

3) Moments :

Factored bending moment at mid span(+Ve) $= 6.956 \times 10^6$ N-mm

Factored bending moment at continuous edge (-Ve) $= 9.275 \times 10^6$ N-mm

4) Reniforcement in middle strip

Provide 10 mm Q HYSD bars @ 300 mm c/c at mid span

Provide extra 8 mm Q HYSD bars @ 600 mm c/c at continuous edge.

Reniforcement in edge strip

Provide = no.sof 8 mm Q HYSD bars for each span

SLAB: S
4

1) Size : 8.23 x 2.23 m

One-way slab with all edge continuous.

2) Load : 10800 N/ m sq.

3) Moments:

Factored bending moment at mid span (+Ve) = 4.911×10^6 N-mm

Factored bending moment at support (-Ve) = 7.749×10^6 N-mm.

4) Reniforcement in mid span

Provide 10 mm Q HYSD bars @ 380 mm c/c

Reniforcement at support

Provide extra 10 mm Q bars @ 760 mm c/c

Distribution reniforcement

Provide 8 mm Q HYSD bars @ 270 mm c/c

SLAB : S8

1) Size 4.23 x 1.615 m

Two-way slab with one long edge dis continuous

2) Load : 9300 N/ m sq.

3) Moments :

Factored bending moment at mid span for shorter span (+Ve)

$= 2.365 \times 10^6$ N-mm

Factored bending moment at continuous edge for shorter span (-Ve)

$= 3.092 \times 10^6$ N-mm.

Factored bending moment at mid span for longer span (+ve)

$= 1.02 \times 10^6$ N-mm.

Factored bending moment at continuous edge for longer span (+ve)

$= 1.346 \times 10^6$ N-mm.

4) Reinforcement for shorter span

Provide 10mm Dia HYSD @ 300 mm c/c.

Reinforcement for longer span

Provide 10mm Dia HYSD @ 300 mm c/c.

Torsion reinforcement

Provide 2 layers of 8mm Dia bars @ 300 mm c/c, @ corners where slab is discontinuous.

DESIGN OF TOP FLOOR SLAB

SLAB : S_2

Size : 4.23 m x 4.23 m ly/lx = 1 < 2
One edge discontinuous

The slab is designed as two-way slab

i) Effective depth

Overall depth = 150 mm

Assuming 10 mm dia bars, 15 mm clear cover,

Effective depth = 150 - 15 - 10/2 = 130mm.

ii) Loads

Floor finishes = 1000 N/sq.m.

Self weight of slab = 0.15 x 25000 = 3750 N/sq.m

Live load = 2000 N/sq.m.

Total load, w = 6750 N/sq.m.

iii) Bending moment

From code IS:456-1978, Appendix C, Table No.22

1. Bending moment in shorter span = Mx = $xwlx^2$

2. Bending moment in longer span = My = $ywlx^2$

Case No.	Type of panel and moments considered	short span coeff. x	long span coeff. y
2.	One short edge discontinuous : -ve moment at continuous edge	0.037	0.037
	+ve moment at mid span	0.028	0.028

1) Shorter span (-ve) Bending moment at continuous edge

$$= 0.037 \times 6750 \times 4.23^2$$

$$= 4468.8 \text{ N-m.}$$

2) Shorter span (+ve) Bending moment at mid span

$$= 0.028 \times 6750 \times 4.23^2$$

$$= 3381.8 \text{ N-m.}$$

3) Longer span (-ve) Bending moment at continuous edge

$$= 0.037 \times 6750 \times 4.23^2$$

$$= 4468.8 \text{ N-m.}$$

4) Longer span (+ve) Bending moment at mid span

$$= 0.028 \times 6750 \times 4.23^2$$

$$= 3381.8 \text{ N-m.}$$

Requirement of steel

At mid span : $Mu = 1.5 \times 3381.8 = 5072.63 \times 10^3$ N-mm.

Mu limit $= 0.36 \times 0.48 (1 - 0.42 \times 0.48) \times 1000 \times 130 \times 15^2$

$$= 34.97 \times 10^6 \text{ N-mm.}$$

Mu is less than Mu limit

$$\frac{Mu}{bd^2} = 0.87\, fy \frac{Pt}{100} \left(1 - \frac{Pt}{100} \times \frac{fy}{fck}\right)$$

$$\frac{5.072 \times 10^6}{1000 \times 130 \times 130} = 3.611\ Pt - 0.9992\ Pt^2$$

$$Pt^2 - 3.614\ Pt + 0.3004 = 0$$

Solving

$$Pt = 0.085\%$$

$$Ast = \frac{0.085}{100} \times 1000 \times 130 = 110.66 \text{ sq.mm.}$$

Since this area is less than the minimum required, provide

Ast = 180 sq.mm.

Provide 10mm dia @ 300mm c/c.

At continuous edge : $Mu = 1.5 \times 4468.8 \times 10^3 = 6.7032 \times 10^6$ N-mm

$$\frac{6.7032 \times 10^6}{1000 \times 130 \times 130} = 3.611\ Pt - 0.9992\ Pt^2$$

$$Pt^2 - 3.614\ Pt + 0.397 = 0.$$

Solving

$$Pt = 0.113\%$$

At support Ast = (0.113/100) x 130 x 1000

= 147.4 sq.mm.

Since this is less than the minimum required reinforcement, provide Ast = 180 sq.mm.

Alternate bars from mid span are cranked at the support.

$$\text{Additional steel required} = 180 - \frac{n\ 10^2}{4} \times \frac{1000}{600}$$

= 49.1 sq.mm.

Provide extra 8mm dia bars @ 600mm c/c.

Edge strip reinforcement :

$$\text{Steel area required} = \frac{0.12}{100} \times 528.8 \times 150 = 180 \text{ sq.mm.}$$

Provide 2 numbers of 8mm dia HYSD bars in each face.

Torsion reinforcement :

As per code IS:456-1978, the area of torsion reinforcement to be provided is three quarters of area required for maximum mid span moment in the slab.

Area of torsion reinforcement in each layer = (3/4)x194.04

= 145.53 sq.mm.

Torsion reinforcement equal to half of the above value is provided at a corner contained by edges over only one of which the slab is continuous.

Hence torsion reinforcement at corners consists of only two layers.

Distance over which torsion reinforcement is provided

= (1/5)xshorter span

= (1/5)x4.23
= 0.846 m.

Provide 1m long, 8mm dia bars @ 300mm c/c at 2 corners.

SLAB : S
3

i) Size : 4.23 x 4.23 m.

Two way slab with all edges continuous.

ii) Loads : 6750 N/sq.m.

iii) Moments :

Factored bending moment at mid span (+ve)

$$= 4.348 \times 10^{6} \text{ N-mm.}$$

Factored bending moment at continuous edge (-ve)

$$= 5.797 \times 10^{6} \text{ N-mm.}$$

iv) Reinforcement in middle strip

Provide 10 mm dia HYSD bars @ 300 mm c/c at mid span

Provide extra 8 mm dia HYSD bars @ 600 mm c/c at continuous edge.

Reinforcement in edge strip

SLAB : S_4

i) Size : 8.23x2.23m.

One-way slab with all edges continuous

ii) Load : 6750 N/sq.m

iii) Moments

Factored bending moment at mid span (+ve)

$$= 2.720 \times 10^{6} \text{ N-mm}$$

Factored bending moment at support (-ve)

$$= 4.610 \times 10^{6} \text{ N-mm}$$

iv) Reinforcement in middle strip

Provide 10 mm dia HYSD bars @ 300 mm c/c at mid span

Provide extra 8 mm dia HYSD bars @ 600 mm c/c at continuous edge

Reinforcement in edge strip

Provide 2 no : of 8 mm dia HYSD bars

Provide extra 8 mm dia HYSD bars @ 270 mm c/c

SLAB : S_5

i) Size : 4.23 x 1.73 m.

One way slab continuous over all edges.

ii) Loads : 6750 N/sq.m.

iii) Moments :

Factored bending moment at mid span (+ve)

$= 1.636 \times 10^6$ N-mm.

Factored bending moment at continuous edge (-ve)

$= 2.775 \times 10^6$ N-mm.

iv) Reinforcement in mid span

Provide 10 mm dia HYSD bars @ 300 mm c/c

Reinforcement in support

Provide extra 8 mm dia HYSD bars @ 600 mm c/c

Distirbution Reinforcement.

Provide extra 8 mm dia HYSD bars @ 270 mm c/c

SLAB : S_6

i) Size : 6.23 x 4.23 m.

Two way slab with all edge continuous.

ii) Loads : 6750 N/sq.m.

iii) Moments :

Factored bending moment at mid span for shorter span (+ve)

$$= 11.740 \times 10^{6} \text{ N-mm.}$$

Factored bending moment at continuous edge for shorter span (-ve)

$$= 15.218 \times 10^{6} \text{ N-mm.}$$

Factored bending moment at mid span for longer span (+ve)

$$= 6.957 \times 10^{6} \text{ N-mm.}$$

Factored bending moment at continuous edge for longer span (-ve)

$$= 9.275 \times 10^{6} \text{ N-mm.}$$

iv) Reinforcement in middle strip for shorter span

Provide 10 mm dia HYSD bars @ 300 mm c/c at mid span

Provide extra 10 mm dia HYSD bars @ 600 mm c/c at continuous edge

Reinforcement in middle strip for longer span

Provide 10 mm dia HYSD bars @ 300 mm c/c at mid span.

Provide extra 10 mm dia HYSD bars @ 600 mm c/c at continuous edge

SLAB : $S_{7,8}$

i) Size : 4.23 x 1.615 m.

Two way slab with one long and other short edge discontinuous .

ii) Reinforcement for shorter span

Provide 10 mm dia HYSD bars @ 300 mm c/c at mid span

Alternate bars are cranked at continuous edge

same reinforcement is provided for longer span

Torsion Reinforcement.

Provide 2 layers 8 mm dia HYSD bars @ 300 mm c/c where slab is discontinuous.

SLAB : S_1

i) Size : 8.23 x 4.23 m.

Two way slab with all edge continuous.

ii) Loads : 6750 N/sq.m.

iii) Moments :

Factored bending moment at mid span for shorter span (+ve)

$$= 8.732 \times 10^{6} \text{ N-mm.}$$

Factored bending moment at continuous edge for shorter span (-ve)

$$= 11.594 \times 10^{6} \text{ N-mm.}$$

Factored bending moment at mid span for longer span (+ve)

$$= 4.344 \times 10^{6} \text{ N-mm.}$$

Factored bending moment at continuous edge for longer span (-ve)

$$= 5.797 \times 10^{6} \text{ N-mm.}$$

iv) Reinforcement in middle strip for shorter span

Provide 10 mm dia HYSD bars @ 300 mm c/c at mid span

Provide extra 10 mm dia HYSD bars @ 600 mm c/c at continuous edge

Reinforcement in middle strip for longer span

Provide 10 mm dia HYSD bars @ 300 mm c/c at mid span.

Provide extra 10 mm dia HYSD bars @ 600 mm c/c at continuous edge

Edge ship reinforcement

Provide 4 no.s of 8 mm dia HYSD bars, along longer span

Provide 2 no.s of 4 mm dia HYSD bars, along shorter span

DESIGN OF INTERMIDATE FLOOR BEAM

1. Beam $P_5 C_1 C_2 X_1$

Beam $C_1 C_2$ size = 230 mm x 600 mmm

At center of $C_1 C_2$ the beam is designed as a

T-beam, from code IS: 456-1983 width of flange

bf = 8.23/6+0.23+6x0.15= 2.501m.

Bending moment at center = $Wl^2/8$

$$= \frac{(36.714+15.715)*8.23^2}{8}$$

= 443.896 kn m

Bending moment at support = -171.31-71.43 = -242.74 Kn m

Net Bending moment at center of span

= 443.896-242.74

= 201.16 kN-m

= 201.16×10^6 N-mm.

Mdu = $1.5 \times 201.16 \times 10^6$ = 301.734×10^6 N mm

Mu1 = 0.36 Df (d-0.42 Df) Fck Bf

= 0.360x150(560-0.42x150)x15x2501

= 1070×10^6 N mm

Mdu < Mu1, Xu < Df

Where Df is the depth of flange

$$Mou = 0.6 \times Xu(d - 0.42\,Xu)\,Fck\,Bf$$

$$301.73 \times 10^{6} = 0.36 \times Xu(560 - 0.42 \times Xu) \times 15 \times 250$$

$$Xu^{2} - 1333.33\,Xu + 53195 = 0$$

Solving

$$Xu = 4.16 \text{ mm}$$

$$Ast = \frac{0.36 Xu\ fck\ bf}{0.87\ fy}$$

$$= \frac{0.36 \times 41.16 \times 15 \times 250}{0.87 \times 415}$$

$$= 1539.8 \text{ sq.mm.}$$

Provide 5 no.s of 20 mm dia : bars at the mid span

At support of C_1 C_2 the beam is designed as rectangular beam .

Limiting moment Mu1 limit

$$= 0.36 \times Xu\ max\ (1 - 0.42 \times \frac{Xu,max}{d}) \times bd^{2} fck$$

$$= 0.36 \times 0.48(1 - 0.42 \times 0.48) 230 \times (560)^{2} \times 15$$

$$= 144.27 \times 10^{6} \text{ N mm}$$

Ultimate moment Mu = 1.5x245.979

$$= 368.97 \times 10^{6} \text{ N mm}$$

Since Mu > Mu limit :, the section is doubly-reinforced.

From code is 456-1978, Appendix E, E-1,2

$$Mu - Mu_{limit} = fsc\ Asc_1(d - d)$$

Where

fsc is the design stress in compression reinforcement

corresponding to a strain of $\frac{0.0035(x\ max:-d')}{Xu\ max}$

Asc is the area of cpmpression reinforcement.

D'is the depth of compresion reinforcement from

compression face

For fy = 415 N sq.mm , Xumax/d= 0.48 Xu max = 0.48 d

$$\frac{0.0035(0.484-d')}{0.48\ d} = \frac{0.0035(0.48x560-40)}{0.48x560}$$

$$= 2.98x10^{-3}$$

For fsc = 415 N/sq.mm , strain = 0.0035

Therefore for strain $= 2.98x10^{-3}$, fsc $= 415x2.98x10^{-3}$
$= 353.343$ N/sq.m.

$(368.97-149.27)x10^{6} = 353.343x\ Asc\ (560-40)$

$Asc = 1195.722$ sq.mm.

Area of tension Reinforcement

Area of tension Reinforcement for a singly Reinforced section

for Mu limit , Ast1 = 0.712x230x560 = 918 sq.mm.

$$Ast1 = \frac{Ascfsc}{0.87fy}$$

$$= \frac{1195.72x353.343}{0.87x415} = 1168 \text{ sq.mm.}$$

2 no.s of 20 mm dia bars @ mid span , are cranked at support.

Moreover 2 no. of 16 mm dia bars also provided to carry shear reinforcement.

Provided Ast = 2x n x(20x20/4)+2x n x 16 x16/4

= 1030 sq.mm

Ast = 1168 + 918−1030

= 1056 sq.mm

Provide extra 2 no, s of 16 mm dia bars

Asc = 1195.722 sq.mm

But 3 bars from mid span are extended to the support

Provide Asc = 3 x n x 20x20/4

= 942.5 sq. mm

Therefore Asc = 1195.722−942.5

= 253.24 sq.mm

Provide 1 extra bar of 20 mm dia

BEAM P C
5 1

This is designed as rectangular beam

Mu = 1.5x33.328 = 49.992 Kn m

$= 49.992 \times 10^{6}$ N mm

Mu limit = $0.36 \times 0.48(1-0.42 \times 0.48) \times 230 \times (560)^{2} \times 15$

$= 149.27 \times 10^{6}$ N mm

Mu, lim > Mu

Section is designed as singly reinforced

$$Mu/bd^2 = 0.87\ fy(Pt/100)\ (1-Pt/100xfy/fck)$$

$$\frac{49.992x10^6}{230x(560)^2} = 0.87x415(Pt/100)(1-Pt/100x415/15)$$

$$= 3.611\ Pt - 0.9992\ Pt^2$$

$$Pt^2-3.614Pt+0.694=0$$

Pt = 0.203%

Ast = 0.203/100x230x560

= 261.9 sq.mm

As per code is 456:1983

Minimum Reinforcement, As/bd = 0.85/fy

Therefore As = $\frac{0.85x230x560}{415}$ = 2631.8 sq.mm

Provide 2 no of 16 mm dia bars

Shear Reinforcement.

Shear force Vu = (36.714+15.7150x8.23)0.6

= 258.9 K n

Nominal shear stress Tv = Vu/bd

$$= \frac{258.9x10^3}{230x560}$$

= 2.01 N/sq.mm

$$\frac{Ast}{bd}\times 100 = \frac{4 \times n \times 20 \times 20/4}{230 \times 560}\times 100$$

$$= 0.976\ \%$$

Permissable shear stress tc = 0.6 N/sq.mm

As tv > tc shear reinforcement is necessasary

Shear to be carried by reinforcement

$$= Vu - tc\ bd$$

$$= 215.75\times 1000 - 0.59\times 230\times 560$$

$$= 139.8 \times 1000$$

$$= 139800\ N$$

But same shear is resisted by bent up bars

Area provided bent up bars = sv Asv Six X

$$= 230 \times 2\times n \times 20\times 20/4\times \text{Sin } 45$$

$$= 102.19\times 1000$$

$$= 102190\ N$$

Shear force to be carried by

shear reinforcement $= Vu - tcbd - 102.19\times 10^{3}$

$$= 215.75\times 1000 - 0.59\times 230\times 560 - 102.19\times 10^{3}$$

$$= 37570\ N$$

Providing 2 legged 8 mm dia bars

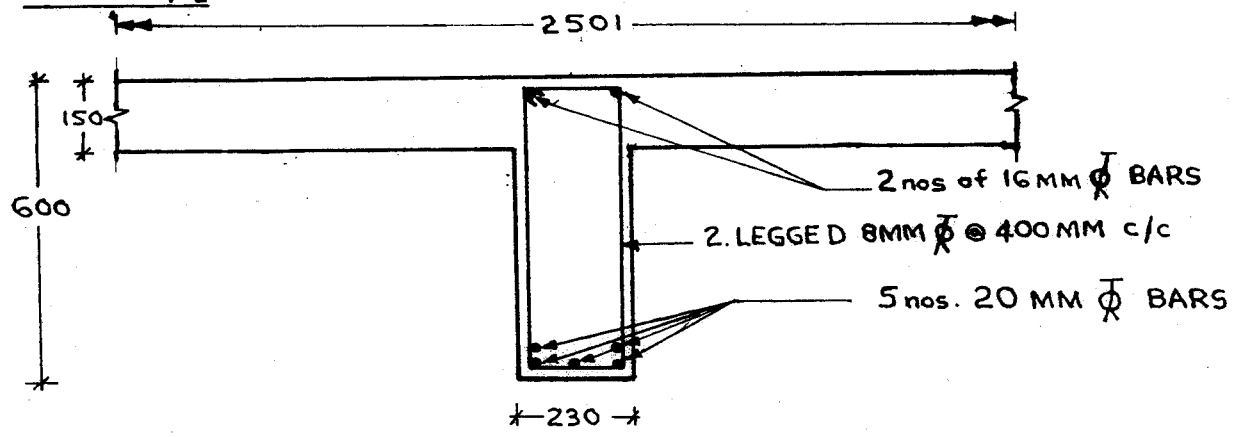
$$\text{Spacing Sv} = \frac{0.87 fy\ Asv.d}{Vus}$$

$$= \frac{0.87\times 415\times 2\times n \times 8^{2}/4\times 560}{27570}$$

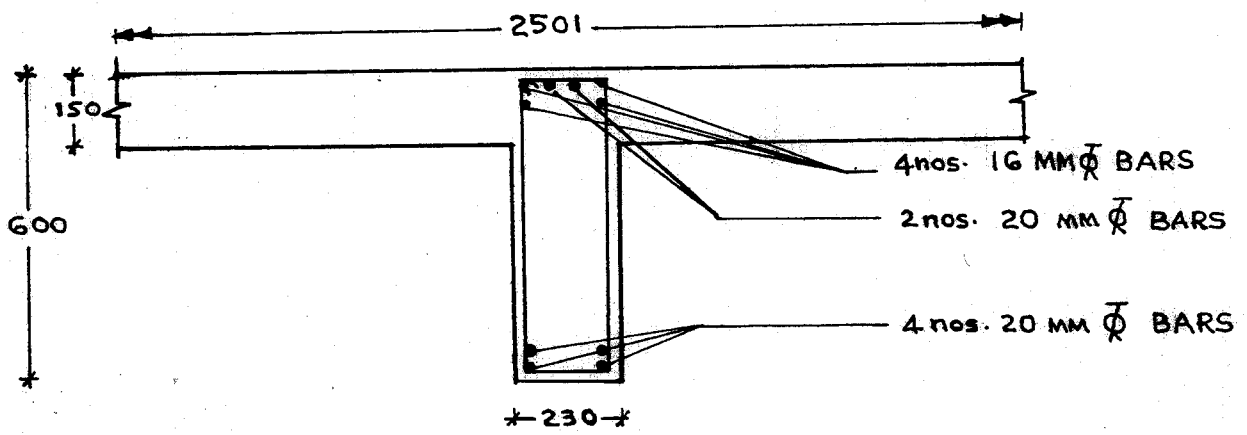
$$= 541\ \text{mm c/c}$$

Provide 2 legged 8 mm dia @ 400 mm c/c

BEAM : C_1C_2

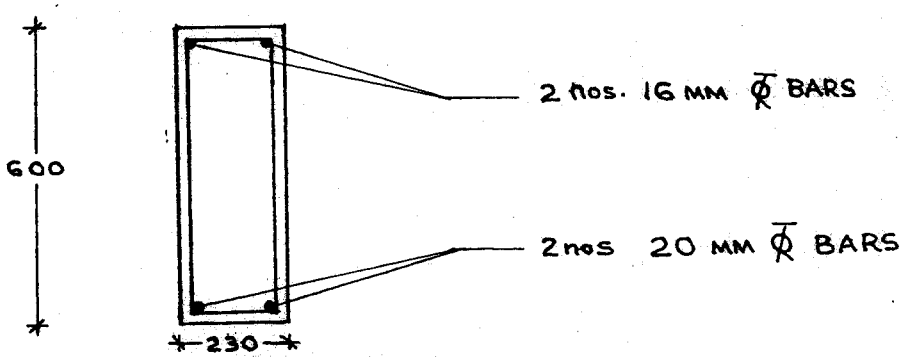


SECTION AT MID SPAN



SECTION AT SUPPORT

BEAM : P_5C_1



SECTION AT SUPPORT

SCALE - 1:150

DESIGN OF COLUMNS

For the design columns are divided into second floor columns and ground floor columns. Columns are designed for axial load and bending moments. Load on a column includes the load from slab and self weight of beams, walls and columns.

Load on a column is calculated using the formula

$$W = \left(TL - \frac{n}{10} LL\right) + n \left(TL' - \frac{n}{10} LL'\right) + n \times Sw$$

Where W ---> total load on the column

TL ---> total load on top column excluding self weight of columns

LL ---> Live load on top column

TL' --> total load on each intermediate column excluding self weight of column

LL' --> Live load on each intermediate column

n ---> Number of storeys above the floor considered

Sw ---> self weight of each column

Second floor columns are designed and the same reinforcement is provided for third floor columns. Similarly the ground floor columns and first floor columns are provided with the same reinforcement.

Design of a typical column is shown in detail. The reinforcement details of all other columns are tabulated and attached.

Design of type III of Second Floor Column

Size of column = 230 mm x 460 mm

Concrete mix = M 20

Grade of steel = Fe 415

P = 590.139 KN

Mx = 111.640 KN m

My = 14.334 KN.m

Design aids for Reinforcement concrete to is : 456-1978 is used for the design

Factored load Pu = 1.5x590.139

= 885.2 Kn

Factored moment acting parallel to the longer dimension Mux = 1.5 x111.64

= 167.46 KN.m

Factored moment acting parallel to the shorter dimension ,Mur =1.5x14.334

= 21.501 KN m

Moment due to minimum excentricity are less than the above vales

Reinforcement is distributed equally on four sides

Assume reinforcement percentage, P = 4

$$\frac{P}{fck} = \frac{4}{20} = 0.2$$

Let Mux be the unimaxial moment capacity of the section about xx - axis

Assuming 25 mm diameter bars with 40 mm clear cover

effective cover d' = 52.5 mm

$$\frac{d'}{D} = \frac{52.5}{460} = 0.114$$

Therefore chart d'/D = 0.1 will be used

$$\frac{Pu}{fck\ b\ D} = \frac{885.2 \times 10^3}{20 \times 230 \times 460} = 0.418$$

Referring to chart 44,

$$\frac{Mu}{fck\ bD^2} = 0.22$$

Therefore $Mux_1 = 0.22 \times 20 \times 230 \times (460)^2/10^6$

= 214.139 K N m

Let Mux be the uniaxial moment capacity of the section about YY axis

$$\frac{d'}{D} = \frac{52.5}{230} = 0.228$$

Chart for d'/D = 0.2 wil be used

Referring to chart 46

$$\frac{Mu}{fck\ bD^2} = 0.16$$

Therefore $Muy_1 = 0.16 \times 20 \times 460 \times 230^2/10^6$

= 77.869 KNm

Calculation of Puz

Reffering to chart 63 corresponding

P = 4 , fy = 415 and fck = 20

$$\frac{Puz}{Ag} = 21 \text{ MPa}$$

Therefore Puz = 21 Ag

$= 21 \times 230 \times 460/10^3$

= 2221.8 Kn

$$\frac{Pu}{Puz} = \frac{885.2}{2221.8} = 0.398$$

$$\frac{Mux}{Mux1} = \frac{167.46}{214.139} = 0.782$$

$$\frac{Muy}{Muy} = \frac{21.501}{77.804} = 0.276$$

Referring to chart 64 corresponding to the above values of Muy/Muy1 and Pu/Puz

Permissible value of $\frac{Mux}{Mux1} = 0.855 > 0.782$

Hence the section is o.k

P = 4 %

Area of steel = 4/100x230x460

= 4232 sq.mm

Provide 12 Nos 22 mm dia HYSD bars distributed equally on four sides

Tranverse Reinforcement

As per is code 456-1978 clause 25.5.3.2,

i) Diameter of polygonal liuks or lateral ties shall not be less than one-fourth of the diameter at the largest longitudinal bar , and in no case less than 5 mm

Diameter of polygonal liuks

i) One fourth of the diameter = 1/4x25= 6.25 mm of largest longitudinal bar

Provide 8 mm dia lateral ties.

ii) Pitch : Shall not be more than least of the following distances:

i) Least lateral dimension of compression member

ii) Sixteen times the smallest diameter of the longitudinal reinforcement.

iii) Forty-eight times the diameter of the transverse reinforcement.

Pitch :

i) 230 mm

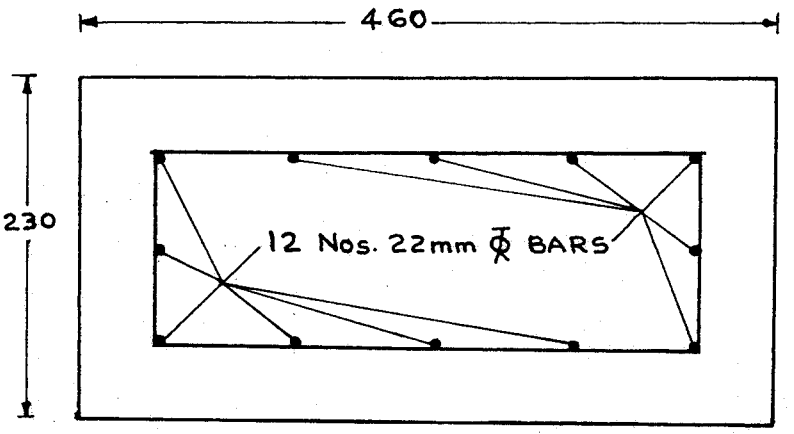
ii) 16x25 = 400 mm

iii) 48 x8 = 384 mm

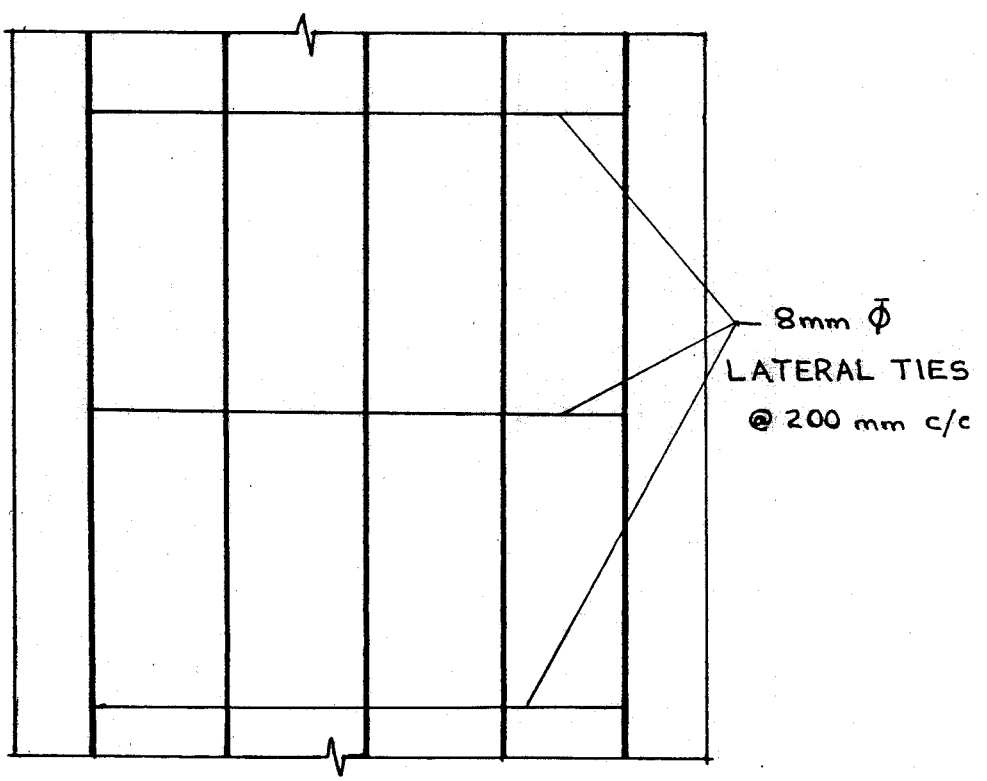
Therefore provide pitch = 230 mm

Therefore 8 mm dia m.s bars @ 230 mm c/c

SECOND FLOOR COLUMN - TYPE III



CROSS SECTION



SECTIONAL ELEVATION

Design of Column Footing

Footing for type - I column

Axial load (P) = 1333.54 KN

Bending moment (M) = 111.64 kN-m

Safe bearing capacity (Qs) = 200 KN/ sq.m = 0.2 MPa

Assume M15 grade concrete and Fe 415 grade steel

Size of Footing

Maximum pressure on soil is limited to the allowable bearing capacity of soil

$$\frac{P}{A} \quad \frac{M}{Z} < Qs$$

Where P is the total axial load inclusive of self weight of footing

A is the area of footing

M is the Bending moment to which the footing is subjected

Z is the Section modules of the section.

Qs is the allowable bearing pressure of soil

Assume 10 % of the column load as self weight of footing let 'a' the projection of the footing beyond the column face, in either direction.

Therefore

$$\frac{1.1 \times 1333.54 \times 10^{3}}{(230+2a)\,(460+2a)} + \frac{111.64 \times 10 \times 6^{6}}{(230+2a)(460+2a)^{2}}$$

$$= 0.2$$

By trial and error

A = 1280 mm

Provide a = 1285 mm

Therefore lenght of footing (L) = 460 + 2x1285 = 3030 mm

Breadth of footing (B) = 230 + 2x1285 = 2800 mm

Pressure and depth

$$Qmax = \frac{1.1 \times 1333.54 \times 10^{3}}{2800 \times 3030} + \frac{111.64 \times 10^{6} \times 6}{2800 \times 3030^{2}}$$

= 0.1989 MPa < Qs (0.2MPa)

Hence the provide section is safe

Qmin = 0.1729 - 0.026

= 0.147 MPa

$$Q1 = \frac{1333.54 \times 10^{3}}{2800 \times 3030} + \frac{111.64 \times 10^{6} \times 6}{2800 \times 3030^{2}}$$

= 0.1832 MPa

$$Q3 = \frac{1333.54 \times 10^{3}}{2800 \times 3030} + \frac{111.64 \times 10^{6} \times 6}{2800 \times 3030^{2}} \times 230$$

= 0.1611 MPa

$$Q5 = \frac{1333.54 \times 10^{3}}{2800 \times 3030} + \frac{111.64 \times 10^{6} \times 6}{2800 \times 3030^{2}} \times 310$$

= 0.1625 MPa

Projection of footing beyond

the face of each column = a= 1285 mm

Maximum Bending moment $M = f B.a^2 (2 q1+q3)$

Where f ---> Partial safety factor

B is the breadth of footing

a is the projection of footing beyond the edge of

Column.

$$= 1.5 \times \frac{2800 \times 1285^2}{6} (2 \times 0.1832 + 0.1611)$$

$$= 60.97 \times 10^7 \text{ N - mm}$$

Moment capacity of a trapezoidal section

$$Mr = ((K1-K2)b1+b2\ K2)d1^2\ fck$$

Where b1 is the widht of the section

at compression face = 230

b2 is the width of the

section at the level of

tension steel = 2800 mm

K2 K1 are constants for a

given reinforcement material for

415 grade steel , K1 = 0.138

K2 = 0.025

$$Mr = (0.138-0.025)230+2800\times0.025)\, d1^2\times15$$

$$= 1440\, d1^2$$

Equating moment capacity to the external

Bending moment

$$1440\, d1^2 = 60.97 \times 10^7$$

$$d1 = 650.7 \text{ mm}$$

$$q0 = P/Bl$$

$$= \frac{1333.54\times10^3}{2800\times3030}$$

$$= 0.1572 \text{ MPa}$$

Punching Shear $Vp = 0.1572\ (2800\times3030-230\times460)$

$$= 1317.05\times10^3 \text{ N}$$

Therefore $d2 = \frac{1317.05\times10^3}{2(230-460)}$

$$= 955 \text{ mm}$$

Provide depth $d = 960$ mm

A pedestel is provided

$$do = dc+60 = 1020 \text{ mm}$$

Direction of footing beyond face of pedestel = a1

$$a1 = a-80$$

$$= 1285-80 = 1205 \text{ mm}$$

Bending moment about the face pf pedestal

$$M1 = \frac{rf\ B\ a1^2}{6}(2q1+q5)$$

$$= \frac{1.5 \times 2800 \times 1205^2}{6}(2 \times 0.1832 \times 0.1625)$$

$$= 53.76 \times 10^7 \text{ N mm}$$

Moment of resistance of trapezoridal section

beyond tha face of the pedestal ,

$$Mr1 = (K1-K2)b1+b2\ K2)\ d1^2\ fck$$

$$= (0.138-0.025)390+2800 \times 0.025)d3^2 \times 15$$

$$= 1711d3^2$$

Therefore $1711\ d3^2 = 53.76 \times 10^7$

$$d3 = 560.5 \text{ mm}$$

punching shear $Vp = 0.1572\ (2800 \times 3030-390 \times 620)$

$$= 1295.67 \times 10^3 \text{ N}$$

$$d4 = \frac{1295.67 \times 10}{2(390+620)}$$

$$= 641.4 \text{ mm}$$

Provided dp = 660 mm

Depth of pedestal = d-dp = 960-660

Check for shear

Critical section shear is at a distance of d/2 from the pheriphering of column where d is the effective depth.

$$V = 1.5 \times 0.1572(2800 \times 3030 - (230+960)(460+960)$$

$$= 1602 \times 10^3 \text{ N}$$

$$b0 = 2(230+960)+2(460+960)$$

$$= 52220 \text{ mm}$$

$$d3 = 230 + \frac{dp-230}{a1}(a-d/2)$$

$$= 230 + \frac{(660-230)}{1205}(1285-960/2)$$

$$= 517.3 \text{ mm}$$

Nominal shear stress (tv) $\frac{V}{bod} = \frac{1602 \times 10^3}{5220 \times 517.3}$

$$= 0.593 \text{ MPa}$$

$$Ks = 0.5 + B/L$$

$$= 0.5 + \frac{2800}{3030} = 1.4271$$

$$Ks = 1$$

Permissible shear stress (tc) = 0.25 fck
= 0.25 15 = 0.968 MPa

Ks tc = 0.968 MPa > tv

Hence the design is safe

Long span steel

$$Ast = \frac{1.15m}{jd\ fy}$$

Where Ast is the area of steel j is the lever arm d is the effective depth

$$\text{Therefore } Ast = \frac{1.15\ M}{(1-0.42k3)xdxfy}$$

(for fe 415 grade steel K3 = 0.48)

$$= \frac{M}{288.11\ d}$$

$$(Ast)x = \frac{60.97x10^{7}}{288.11x960}$$

= 2205 sq.mm

Provide 20 no of 12 mm dia HYSD bars

Ast provided = 2262 sq.mm > 2205 sq.mm

Spacing = 142 mm c/c

Short span steel = (Ast)X x d/dy

$$= 2400x \frac{960}{948}$$

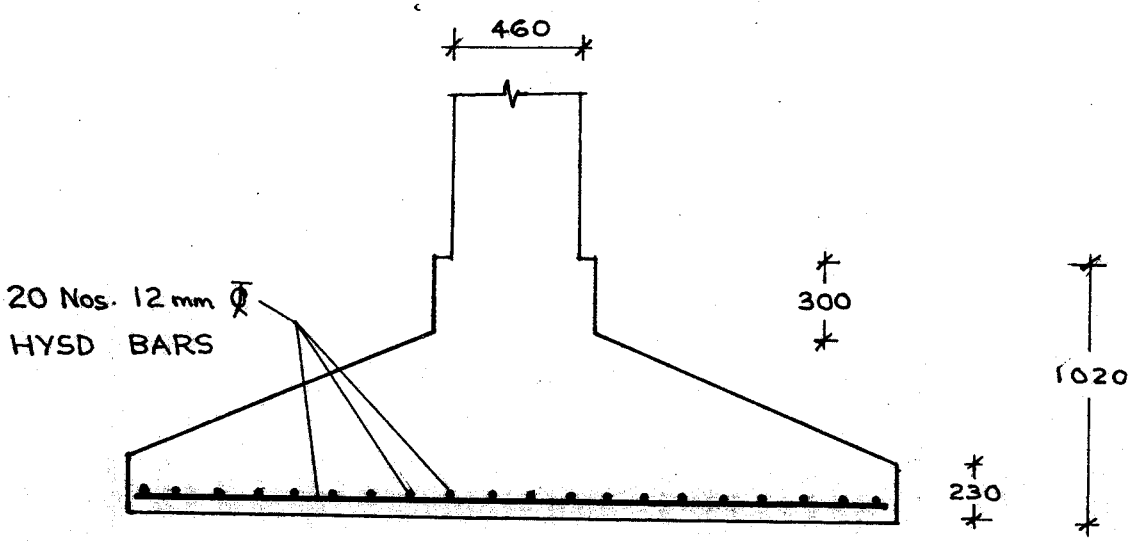
= 2233 sq.mm

Provided 20 no,s of 12 mm dia HYSD bars

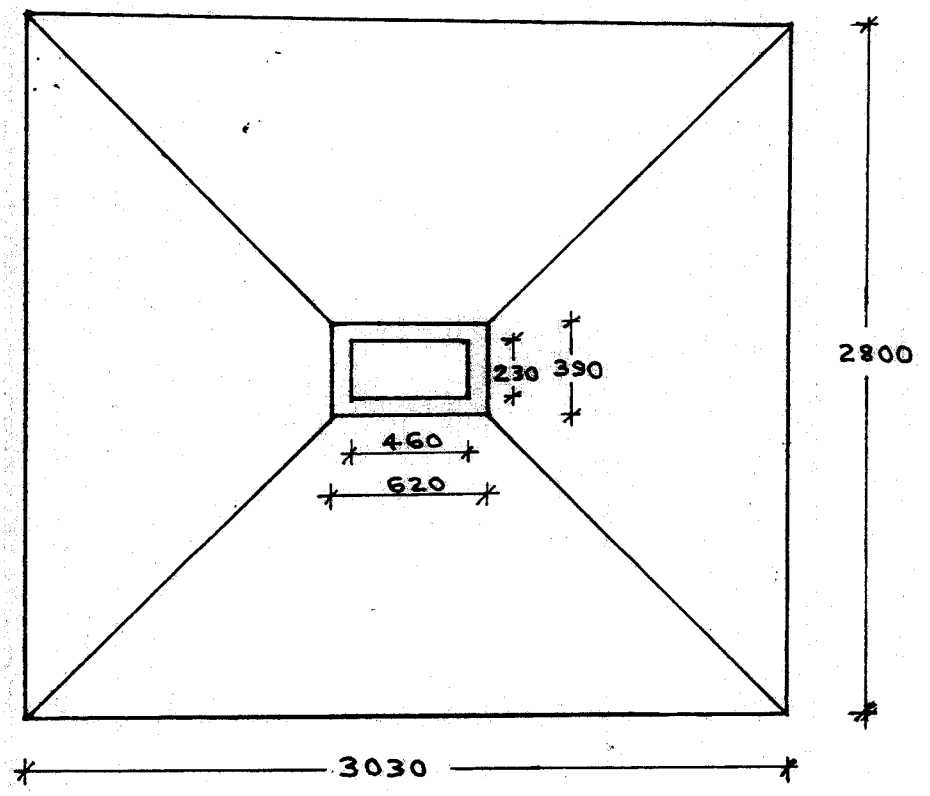
Ast Provided = 2262 sq.mm

(Spacing = 154 mm c/c)

FOOTING FOR TYPE-I COLUMN



SECTIONAL ELEVATION



PLAN

ESIGN COLUMN FOOTING

YPE	P (K N)	L (mm)	B (mm)	Dp (mm)	D(mm)	(Ast)x	(Ast)Y
1	1333.540	3030	2800	300	1020	20 nos 12 mm dia HYSD bars	20 nos of 12 mm dia HYSD
2	1115.140	2760	2530	260	860	16 nos 12 mm dia HYSD bars	20 nos of 12 mm dia HYSD
3	971.690	2560	2330	230	760	14 nos 12 mm dia HYSD bars	14 nos of 12 mm dia HYSD
4	802.630	2360	2130	180	640	13 nos 12 mm dia HYSD bars	13 nos of 12 mm dia HYSD
5	601.919	2060	1830	160	510	10 nos 12 mm dia HYSD bars	10 nos of 12 mm dia HYSD

P is the axial load on footing

L is the length of footing

B is the breadth

Dp is the depth of pedestal

D is the overall depth of footing at the face of column

(Ast) x is the Reinforcement along longer span

(Ast)y is the Reinforcement along shorter span

STAIRCASE

DESIGN OF STAIR CASE NO 1

Assume 150 mm rise for each step no of step risers = 3600/150

= 24

Providing two flights no of risers in each flight = 12

No of treads in eachb flight = 11

Provide going to the stair as 3 m

Tread of each step = 3000/11

= 272.7mm

Spacing for landing = 1.5 m

Loading

Assume thickness of waist slab as 175 mm

Dead weight waist on horizontal area

$$= \frac{0.175 \times 2500}{\frac{272.7}{2} / \frac{272.7}{2} + 150}$$

= 4993 N/sq.m

Dead weight of steps = 1/2x0.150x25000

= 1875 N/sq.m

Weight of floor finish = 550 N/sq.m

Live load = 5000 N/sq.m

Total load = 12418 N/sq.m

Considering 1 m width step load = 12418 N/m

Effective span = 3+1.5 = 4.5m

Design of waist slab

The span is considered simply supported

Max Bending moment $= 12418 \times 4.5^2$

$= 31433$ N .m

$Mu = 1.5 \times 31433 = 4.715 \times 10^4$ N-m

$= 4.715 \times 10^7$ N mm

Using M15 concrete and Fe 415 grade steel,

$$\text{Mu limit} = 0.36 \frac{Xumax}{d} (1-0.42 \times Xumax)\, bd^2 fck$$

$= 0.6 \times 0.48 (1-0.42 \times 0.48) 1000 \times 154^2 \times 15$

$= 4.908 \times 10^7$ N mm > Mu

$$Mu/bd^2 = 0.87\, fy\, Ast/bd\, (1-Ast/bd\, fy/fck)$$

$$\frac{4.715 \times 10^7}{1000 \times 154^2} = 0.87 \times 415 Pt (1- Pt/100 \times 415/15)$$

$$1.988 = 3.611\, Pt\, 0.999\, Pt^2$$

$$Pt^2 - 3.615\, Pt + 1.989 = 0$$

$$Pt = \frac{3.615+2.260}{2} = 0$$

$$Pt = 0.678$$

Ast = 0.678/100x1000x154= 1051 sq.mm in1 m width

using 12mm dia No of bars required in 1.8 m width

= (1051/ nx12x12/4)x1.800

= 16.73 say 17 Nos

Distribution steel = 0.12/100x1000x175 = 210 sq.mm

Provide 10 mm dia HYSD bars @ 360 mm c/c as distribution steel .

Check for shear stress

Vamx = Wl/2 = 12418x4.5/2

= 27941 N

Vu = 1.5x27941 = 41911 N

$$Cv = Vu/bd = \frac{41911}{1000x154} = 0.272 \text{ Mpa}$$

Ast = 17 x n x12x12/4

= 1923 sq.mm

$$Pt = \frac{1923}{1800x154} x100 = 0.693\ \%$$

Tc = 0.522 N/ sq.mm

In solid slabs design shear strength of concrete = Ktc

In 175 mm thick slab k = 1.25

Therefore design shear strength = 1.25x0.522

= 0.653 N sq.mm > 0.272 N/sq.mm

Therefore shear stress are within safe limits

DESIGN OF STAIRCASE NO 2

Rise of each step = 150 mm

Therefore no of risers =3600/150 = 24

Providing 3 flights each consisting of 8 risers

No of treads in each flight= 7

Provide tread as 300 mm

Therefore going = 7x300 = 2100 mm

Widht of flight = (4000 - 2100)/2

= 950 mm.

Design of flight AB and DC

i) Load on flights

Dead load of steps per horizontal m run

= 1/2 x 0.15 x 0.95 x 25000

= 1782 N/m.

Assume thickness of waist slab as 175 mm

Weight of slab per horizontal m run

$$= \frac{0.175 \times 25000 \times 0.95}{0.8944}$$

= 4647 N/m.

Floor finish per horizontal m run

= 550 x 0.95

= 523 N/m

Total dead load per horizontal m run

= 6952 N/m.

Live load per horizontal m run

= 5000 x 0.95

= 4750 N/m.

Total load per horizontal m run

= 11702 N/m.

ii) Load on landings

Weight of slab = 0.175 x 25000 x 0.95

= 4157 N/m

Floor finish = 550 x 0.95 = 523 N/m.

Live load = 5000 x 0.95 = 4750 N/m.

Total load = 9430 N/m.

As per IS: 456-1978 clause 32.2

In the case of stairs with open wells, where spans partly crossing at right angles occur the load on areas common to any such spans may be taken as one half in each direction.

Therefore, Load on landings = 9430/2 = 4715 N/m.

iii) Maximum bending moment

Reaction at support,

$$R_A = \frac{4715 \times (1.065^2/2) + 11702 \times 2.1 \times ((2.1/2) + 1.065)}{2.1 + 1.065}$$

= 17266 N.

Let maximum bending moment occur at section distant x form support A.

Equating shear force at the section to be zero, we get

$$17266 - 11702\,x = 0$$

$$x = 1.475 \text{ m.}$$

$$M_{max} = 17266 \times 1.475 - 11702 \times (1.475^2/2)$$

$$= 12738 \text{ N-m.}$$

$$= 12.738 \times 10^6 \text{ N-mm.}$$

Factored moment, $Mu = 12.738 \times 10^6 \times 1.5 = 19.107 \times 10^6$ N-mm.

$$Mu \text{ limit} = 0.36 \times 0.48 (1 - 0.42 \times 0.48) \times 950 \times 155^2 \times 15$$

$$= 47.233 \times 10^6 \text{ N-mm} > Mu$$

iv) Reinforcement

$$\frac{Mu}{bd^2} = 0.87 fy \frac{Ast}{bd} \left(1 - \frac{Ast}{bd} \times \frac{fy}{fck}\right)$$

$$\frac{19.107 \times 10^6}{950 \times 155^2} = 0.87 \times 415 \frac{Pt}{100} \left(1 - \frac{Pt}{100} \frac{415}{15}\right)$$

$$0.837 = 3.611 Pt - 0.999 Pt^2$$

Solving, Pt = 0.250%

$$Ast = \frac{0.25}{100} \times 950 \times 155 = 368 \text{ sq.mm.}$$

Provide 6 Nos. 10 mm dia HYSD bars

$$\text{Distribution steel} = \frac{0.12}{100} \times 1000 \times 175$$

$$= 210 \text{ sq.mm.}$$

Provide 8 mm dia HYSD bars at 220 mm c/c as distribution steel.

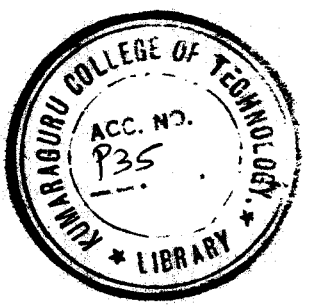
v) Check for shear

Maximum shear force = support reaction at A

= 17266 N

$$Vu = 1.5 \times 17266 = 25899 \text{ N}$$

$$tv = \frac{Vu}{bd} = \frac{25899}{950 \times 155} = 0.176 \text{ MPa.}$$



$$Ast = 6 \times n \times \frac{10^2}{4} = 471 \text{ sq.mm.}$$

$$Pt = \frac{471}{950 \times 155} \times 100 = 0.32\%$$

From table 13 of IS: 456-1978, for

Pt = 0.32% tc = 0.381 MPa

For solid slabs, design shear strength of concrete = Ktc

For 175 mm thick slab, K = 1.25

Therefore, design shear strength = 1.25 x 0.381

= 0.476 MPa

> 0.176 MPa

Therefore, shear stresses are within safe limits.

Design of flight BC

Load on flight = 11702 N/m.

Load on landing = 4715 N/m.

Reaction at support R_B = 1/2 (4715x1.065x2+11702x2.1)

= 17309 N.

Since loading is symmetrical, maximum bending moment occurs at centre.

Therefore,

$$M = 17309 \times 2.115 - 4715 \times 1.065\ (2.115-(1.065/2)) - 11702 \times \frac{1.05^2}{2}$$

$$= 22211\ \text{N-m} = 22.211 \times 10^6\ \text{N-mm.}$$

Factored moment, $Mu = 1.5 \times 22.211 \times 10^6 = 33.317 \times 10^6$ N-mm.

$$\text{Mu limit} = 47.233 \times 10^6\ \text{N-mm} > Mu$$

Reinforcement

$$\frac{Mu}{bd^2} = 0.87 \times fy \frac{Ast}{bd}\left(1 - \frac{Ast}{bd}\frac{fy}{fck}\right)$$

$$\frac{33.317 \times 10^6}{950 \times 155^2} = 0.87 \times 415 \left(\frac{Pt}{100}\right)\left(1 - \frac{Pt}{100} \times \frac{415}{15}\right)$$

$$1.46 = 3.611\ Pt - 0.999\ Pt^2$$

$$Pt^2 - 3.615\ Pt + 1.462 = 0$$

Solving $Pt = 0.464\ \%$

$$Ast = \frac{0.464}{100} \times 950 \times 155 = 684\ \text{sq.mm}$$

Provide 9 Nos 10 mm diameter HYSD bars as main steel

Provide 8 mm dia HYSD bars at 220 mm c/c

as distrubution steel.

CHECK FOR SHEAR

Maximum shear force = 17309 N

$Vu = 1.5 \times 17309 = 25964$ N

$$Tv = \frac{Vu}{bd} = \frac{25964}{950 \times 155} = 0.176 \text{ MPa}$$

$$Pt = \frac{9 \times n^2 \times 10/4 \times 100}{950 \times 155} = 0.48 \%$$

For Pt = 0.48 % Tc = 0.451 MPa

Design shear strength = Ktc

= 1.25x0.451

= 0.564 MPa > 0.176 MPa

Therefore shear stress are with in safe limits

DESIGN OF LINTEL

LINTELS OVER WINDOW OPENINGS

Width of opening = 1.6 m.

Bearing at ends = 150 mm.

Effective depth (d) = 120 mm.

Effective span is least of

i) c/c of support = 1600 + 150 = 1750 mm.

ii) Clear span + effective depth = 1600 + 120 = 1720 mm.

Height of wall above opening = 1.4 m.

Unit weight of brick = 20000 N/cub.m.

Weight of masonry wall = 1.4x1.72x0.23x20000

= 11076.8 N

Bending moment due to self weight of masonry = WL/6

$$= \frac{11076.8 \times 1.72}{6}$$

= 3175.35 N-m.

Dead load of lintel = 0.23 x 0.15 x 25000

= 862.5 N/m.

$$\text{Bending moment due to dead load of lintel} = \frac{wl^2}{8}$$

$$= \frac{862.5 \times 1.72^2}{8}$$

= 318.95 N-m.

Total Bending moment = 3494.31 N-m.

Therefore, ultimate moment Mu = 1.5 x 3494.3

= 5241.5 N-m.

Limiting moment Mu lim = 6853.9 N-m > Mu

Therefore,

$$\frac{Mu}{bd^2} = \frac{5241.5 \times 10^3}{230 \times 120^2} = 1.582 \text{ N/sq.mm.}$$

Percentage of steel Pt = 0.511%

Area of steel required $= \frac{0.511}{100} \times 230 \times 120$

= 141.04 sq.mm.

From IS: code 456-1978

Minimum area of Reinforcement Ast $= \frac{0.85\ bd}{fy}$

$$= \frac{0.85 \times 230 \times 120}{415}$$

= 56.53 sq.mm.

Provide 2 nos of 12 mm dia HYSD bars as holdup bars

DESIGN OF STRIRRUPS

Weight of wall = 11077 N.

Dead load of lintel = 862.5 N/m.

Maximum shear force = W/2+Wl/2 $= \frac{11077}{2} + \frac{862.5 \times 1.72}{2}$

= 6280.3 N.

Ultimate shear force Vu = 1.5 x Vmax

= 1.5 x 6280.3

= 9420.4 N.

Normal shear stress $v = \frac{Vu}{bd} = \frac{9420.4}{230 \times 120}$

$= 0.3413$ N/sq.mm.

Percentage of tensile reinforcement $= 100 \times \frac{As}{bd}$

$= \frac{100 \times 2 \times \quad \times (\quad)^2/4}{230 \times 120}$

$= 0.82\%$

Permissible shear stress $c = 0.5568$ N/sq.mm $> v$

Provide 2 legged, 6 mm dia bars @ 250 mm c/c

LINTEL OVER DOOR OPENING

Width of opening = 1.20 m.

Effective depth (d) = 90 mm.

Effective span in least of,

i) c/c of support = 1200+150 = 1350 mm.

ii) Clear span + effective depth

= 1200 + 90 = 1290 mm.

Height of wall above opening = 1.4 m.

Unit weight of brick = 20,000 N/cub.m.

Weight of masonry wall = 1.42 x 1.29 x 0.23 x 20000

= 8426.3 N.

Bending moment to self weight of masonry = WL/6

$$= \frac{8426.8 \times 1.29}{6}$$

= 1811.7 N-m.

Dead load lintel = 0.23 x 0.12 x 25000

= 690 N/m.

Bending moment due to dead load weight of lintel = $WL^2/8$

$$= \frac{690 \times 1.29^2}{8}$$

= 143.53 N-m.

Therefore, total Bending moment = 1811.7 + 143.53

= 1955.23 N-m.

Ultimate moment Mu = 1.5 x 1955.23 = 2932.85 N-m.

Limiting moment Mu Line = 3855.4 N-m > Mu.

$$\frac{Mu}{bd^2} = \frac{2932.85 \times 10^3}{230 \times 90^2} = 1.574 \text{ N/sq.mm.}$$

Percentage of steel Pt = 0.508%

$$\text{Area of steel required Ast} = \frac{0.508}{100} \times 230 \times 90 = 105.2 \text{ sq.mm}$$

$$\text{Minimum Reinforcement Ast} = \frac{0.85}{fy} \times bd$$

$$= \frac{0.85 \times 230 \times 90}{415} = 42.4 \text{ sq.mm}$$

Provide 2 nos of 12 mm dia HYSD bars, as holdup bars

Design of strips

Weight of wall = 8426.8 N.

Dead load of lintel = 690 N/m.

$$\text{Maximum shear force Vmax} = \frac{8426.8}{2} + 690 \times \frac{1.29}{2}$$

$$= 4658.5 \text{ N.}$$

Ultimate shear force Vu max = 1.5 x 4658.5

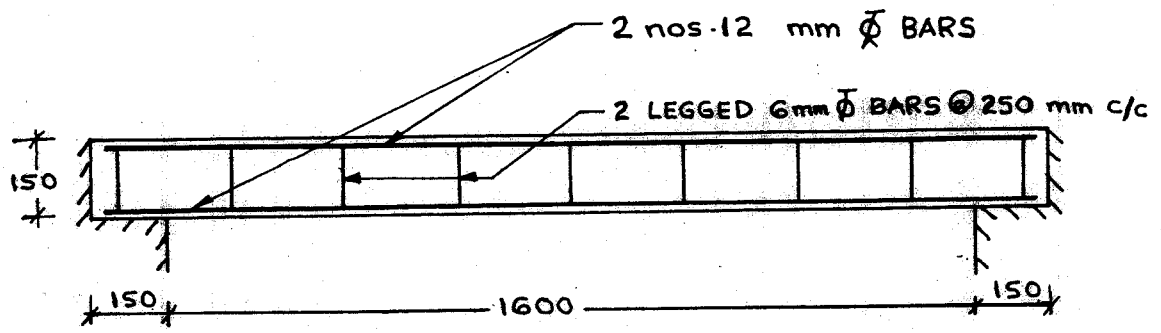
= 6987.7 N.

$$\text{Nominal shear stress } v = \frac{Vu\ max}{bd} = \frac{6987.7}{230 \times 90}$$

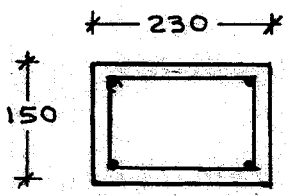
= 0.338 N/sq.mm.

Percentage of tensile Reinforcement Pt

LINTEL OVER WINDOW OPENINGS

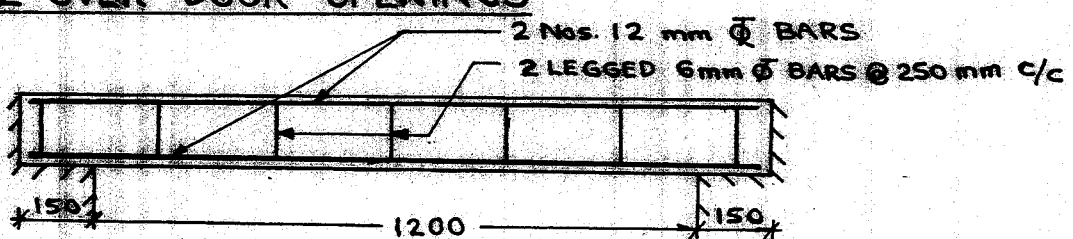


SECTIONAL ELEVATION

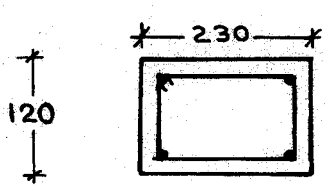


CROSS SECTION

LINTEL OVER DOOR OPENINGS



SECTIONAL ELEVATION



CROSS SECTION

Estimation (Approximate)

Ground Floor

Total plinth area = 1302.223 sq.m

$$= \frac{1302.223}{0.093}$$

= 14002.398 sq foot

Rate for one square foot = Rs 190/-

Total cost of ground floor = 14002.398x190

= 26,60,455/-

First Floor

Total plinth area = 14002.398 square foot

Rate for one square = 200/-

Total cost for the first floor = 14002.398x200

= 28,00,479/-

Second Floor

Total plinth area = 14002.398 square foot

Rate for one square foot = 210/-

Total cost for second floor = 14002.398x210

= 29,40,503/-

Third Floor

Total plinth area = 14002.398 square foot

Rate for one square foot = 220/-

Total cost for the third floor = 14002.398x220

= 30,80,527/-

Total estimation :

1) Cost for gorund floor	= 26,60,455
2) Cost for first floor	= 28,00,479
3) Cost for second floor	= 29,40,503
4) Cost for third floor	= 30,80,527
Estimated total cost Rs	1,14,81,964/-

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