Design Of Slabs : Default Level

Design Method : Limit State Method
Design Code : IS 456:2000

Design Of Slab

Slab Name : S1
Slab Type : TwoWay Slab
Grade of Concrete :M20
Grade of Steel : Fe415
Dimensions : Lx = 3.200 m ,Ly = 3.125 m ,Thickness= 110 mm
Span Ratio : Longer/Shorter = 1.024
Boundary Condition : Internal Panel

Loading on Slab :(Table 1)

Total DL = (1) + (2) + (3)

Self Weight (1) kN/m^2	Floor Finish (2) kN/m^2	Sunk Load (3) kN/m^2	Total DL (4) kN/m^2	Live Load (5) kN/m^2	Total LL (6) kN/m^2	Total Design Load 1.5 x [(4)+(6)] kN/m^2
2.750	1.000	0.000	3.750	2.000	2.000	8.625

Deflection Check :(Table 2)

As per clause 23.2.1 of IS 456:2000:

(Span/Depth) Ratio	Modification Factor (a)	Basic Factor (b)	Permissible Ratio (a x b)	Status
28.409	1.000	32.000	32.000	OK

Moment And Steel Calculations :

W = Total Design Load = 8.625 kN/m^2
L_x = Shorter span = 3.125 m

Position	Moment Coeff.α_x or α_y	Moment kN-m	A_{st_rqd} per metre mm^2	Steel Detail dia @ spc mm c/c	A_{st_prv} per metre mm^2	Remark
MidShort	0.025	2.102	69.690	#8 @ 200	251.327	Main
MidLong	0.024	2.021	66.964	#8 @ 200	251.327	Other
SuppDown	0.033	2.796	93.242	--	0.000	Extra at Top
SuppTop	0.033	2.796	93.242	--	0.000	Extra at Top
SuppLeft	0.032	2.695	89.794	--	0.000	Extra at Top
SuppRight	0.032	2.695	89.794	--	0.000	Extra at Top

Minimum Steel Check :

As per clause 26.5.2.1 of IS 456:2000:
Minimum area of steel For Main Steel = 0.12 x C/S Area
= 132.000 mm^2

Notes :

Extra steel at Top support is computed considering
the bent-ups,if any,coming from the adjoining span.
It is the maximum of the extra steel required
for each slab at that common support.

S. B. Bhattacharyya
B.E. (Civil)
ESE- 116 /1

Slab Name : S2
Slab Type : TwoWay Slab
Grade of Concrete :M20
Grade of Steel : Fe415
Dimensions : Lx = 4.400 m ,Ly = 3.125 m ,Thickness= 110 mm

Span Ratio : Longer/Shorter = 1.408

Boundary Condition : Internal Panel

Loading on Slab :(Table 1)

Total DL = (1) + (2) + (3)

Self Weight (1) kN/m²	Floor Finish (2) kN/m²	Sunk Load (3) kN/m²	Total DL (4) kN/m²	Live Load (5) kN/m²	Total LL (6) kN/m²	Total Design Load 1.5 x [(4)+(6)] kN/m²
2.750	1.000	0.000	3.750	2.000	2.000	8.625

Deflection Check :(Table 2)

As per clause 23.2.1 of IS 456:2000:

(Span/Depth) Ratio	Modification Factor (a)	Basic Factor (b)	Permissible Ratio (a x b)	Status
28.409	1.000	32.000	32.000	OK

Moment And Steel Calculations :

W = Total Design Load = 8.625 kN/m^2

L_x = Shorter span = 3.125 m

Position	Moment Coeff.α_x or α_y	Moment kN-m	A_{st_rqd} per metre mm²	Steel Detail dia @ spc mm c/c	A_{st_prv} per metre mm²	Remark
MidShort	0.039	3.298	110.455	#8 @ 200	251.327	Main
MidLong	0.024	2.021	66.964	#8 @ 200	251.327	Other
SuppDown	0.051	4.309	145.586	--	0.000	Extra at Top
SuppTop	0.051	4.309	145.586	--	0.000	Extra at Top
SuppLeft	0.032	2.695	89.794	--	0.000	Extra at Top
SuppRight	0.032	2.695	89.794	--	0.000	Extra at Top

Minimum Steel Check :

As per clause 26.5.2.1 of IS 456:2000:

Minimum area of steel For Main Steel = 0.12 x C/S Area

= 132.000 mm^2

Notes :

Extra steel at Top support is computed considering the bent-ups,if any,coming from the adjoining span. It is the maximum of the extra steel required for each slab at that common support.

Slab Name : S3

Slab Type : TwoWay Slab

Grade of Concrete :M20

Grade of Steel : Fe415

Dimensions : Lx = 3.200 m ,Ly = 3.250 m ,Thickness= 110 mm

Span Ratio : Longer/Shorter = 1.016

Boundary Condition : Internal Panel

Loading on Slab :(Table 1)

Total DL = (1) + (2) + (3)

Self Weight (1) kN/m²	Floor Finish (2) kN/m²	Sunk Load (3) kN/m²	Total DL (4) kN/m²	Live Load (5) kN/m²	Total LL (6) kN/m²	Total Design Load 1.5 x [(4)+(6)] kN/m²
2.750	1.000	0.000	3.750	2.000	2.000	8.625

Deflection Check :(Table 2)

As per clause 23.2.1 of IS 456:2000:

(Span/Depth) Ratio	Modification Factor (a)	Basic Factor (b)	Permissible Ratio (a x b)	Status
29.091	1.000	32.000	32.000	OK

Moment And Steel Calculations :

W = Total Design Load = 8.625 kN/m^2

L_x = Shorter span = 3.200 m

Position	Moment Coeff.α_x or α_y	Moment kN-m	A_{st_rqd} per metre mm²	Steel Detail dia @ spc mm c/c	A_{st_prv} per metre mm²	Remark
MidShort	0.025	2.175	72.138	#8 @ 200	251.327	Main

MidLong	0.024	2.120	70.275	#8 @ 200	251.327	Other
SuppDown	0.032	2.826	94.261	--	0.000	Extra at Top
SuppTop	0.032	2.826	94.261	--	0.000	Extra at Top
SuppLeft	0.033	2.895	96.619	--	0.000	Extra at Top
SuppRight	0.033	2.895	96.619	--	0.000	Extra at Top

Minimum Steel Check :

As per clause 26.5.2.1 of IS 456:2000:

Minimum area of steel For Main Steel = 0.12 x C/S Area

= 132.000 mm^2

Notes :

Extra steel at Top support is computed considering
the bent-ups,if any,coming from the adjoining span.
It is the maximum of the extra steel required
for each slab at that common support.

Slab Name : S4

Slab Type : TwoWay Slab

Grade of Concrete :M20

Grade of Steel : Fe415

Dimensions : Lx = 3.275 m ,Ly = 3.250 m ,Thickness= 110 mm

Span Ratio : Longer/Shorter = 1.008

Boundary Condition : Internal Panel

Loading on Slab :(Table 1)

Total DL = (1) + (2) + (3)

Self Weight (1) kN/m²	Floor Finish (2) kN/m²	Sunk Load (3) kN/m²	Total DL (4) kN/m²	Live Load (5) kN/m²	Total LL (6) kN/m²	Total Design Load 1.5 x [(4)+(6)] kN/m²
2.750	1.000	0.000	3.750	2.000	2.000	8.625

Deflection Check :(Table 2)

As per clause 23.2.1 of IS 456:2000:

(Span/Depth) Ratio	Modification Factor (a)	Basic Factor (b)	Permissible Ratio (a x b)	Status
29.545	1.000	32.000	32.000	OK

Moment And Steel Calculations :

W = Total Design Load = 8.625 kN/m^2

L_x = Shorter span = 3.250 m

Position	Moment Coeff.α_x or α_y	Moment kN-m	A_{st_rqd} per metre mm²	Steel Detail dia @ spc mm c/c	A_{st_prv} per metre mm²	Remark
MidShort	0.024	2.214	73.476	#8 @ 200	251.327	Main
MidLong	0.024	2.186	72.529	#8 @ 200	251.327	Other
SuppDown	0.032	2.950	98.503	--	0.000	Extra at Top
SuppTop	0.032	2.950	98.503	--	0.000	Extra at Top
SuppLeft	0.032	2.915	97.304	--	0.000	Extra at Top
SuppRight	0.032	2.915	97.304	--	0.000	Extra at Top

Minimum Steel Check :

As per clause 26.5.2.1 of IS 456:2000:

Minimum area of steel For Main Steel = 0.12 x C/S Area

= 132.000 mm^2

Notes :

Extra steel at Top support is computed considering
the bent-ups,if any,coming from the adjoining span.
It is the maximum of the extra steel required
for each slab at that common support.

Slab Name : S5

Slab Type : TwoWay Slab

Grade of Concrete :M20

Grade of Steel : Fe415

Dimensions : Lx = 1.125 m ,Ly = 1.775 m ,Thickness= 110 mm

Span Ratio : Longer/Shorter = 1.578

Boundary Condition : Internal Panel

Loading on Slab :(Table 1)

Total DL = (1) + (2) + (3)

Self Weight (1) kN/m²	Floor Finish (2) kN/m²	Sunk Load (3) kN/m²	Total DL (4) kN/m²	Live Load (5) kN/m²	Total LL (6) kN/m²	Total Design Load 1.5 x [(4)+(6)] kN/m²
2.750	1.000	0.000	3.750	2.000	2.000	8.625

Deflection Check :(Table 2)

As per clause 23.2.1 of IS 456:2000:

(Span/Depth) Ratio	Modification Factor (a)	Basic Factor (b)	Permissible Ratio (a x b)	Status
10.227	1.000	32.000	32.000	OK

Moment And Steel Calculations :

W = Total Design Load = 8.625 kN/m^2

L_x = Shorter span = 1.125 m

Position	Moment Coeff.α_x or α_y	Moment kN-m	A_{st_rqd} per metre mm²	Steel Detail dia @ spc mm c/c	A_{st_prv} per metre mm²	Remark
MidShort	0.042	0.461	15.082	#8 @ 200	251.327	Main
MidLong	0.024	0.262	8.555	#8 @ 200	251.327	Other
SuppDown	0.032	0.349	11.414	--	0.000	Extra at Top
SuppTop	0.032	0.349	11.414	--	0.000	Extra at Top
SuppLeft	0.055	0.602	19.721	--	0.000	Extra at Top
SuppRight	0.055	0.602	19.721	--	0.000	Extra at Top

Minimum Steel Check :

As per clause 26.5.2.1 of IS 456:2000:

Minimum area of steel For Main Steel = 0.12 x C/S Area

= 132.000 mm^2

Notes :

Extra steel at Top support is computed considering the bent-ups,if any,coming from the adjoining span. It is the maximum of the extra steel required for each slab at that common support.

Slab Name : S6

Slab Type : TwoWay Slab

Grade of Concrete :M20

Grade of Steel : Fe415

Dimensions : Lx = 3.150 m ,Ly = 2.800 m ,Thickness= 110 mm

Span Ratio : Longer/Shorter = 1.125

Boundary Condition : Internal Panel

Loading on Slab :(Table 1)

Total DL = (1) + (2) + (3)

Self Weight (1) kN/m²	Floor Finish (2) kN/m²	Sunk Load (3) kN/m²	Total DL (4) kN/m²	Live Load (5) kN/m²	Total LL (6) kN/m²	Total Design Load 1.5 x [(4)+(6)] kN/m²
2.750	1.000	0.000	3.750	2.000	2.000	8.625

Deflection Check :(Table 2)

As per clause 23.2.1 of IS 456:2000:

(Span/Depth) Ratio	Modification Factor (a)	Basic Factor (b)	Permissible Ratio (a x b)	Status
25.455	1.000	32.000	32.000	OK

Moment And Steel Calculations :

W = Total Design Load = 8.625 kN/m^2

L_x = Shorter span = 2.800 m

Position	Moment Coeff.α_x or α_y	Moment kN-m	A_{st_rqd} per metre mm^2	Steel Detail dia @ spc mm c/c	A_{st_prv} per metre mm^2	Remark
MidShort	0.029	1.961	64.927	#8 @ 200	251.327	Main
MidLong	0.024	1.623	53.582	#8 @ 200	251.327	Other
SuppDown	0.038	2.603	86.663	--	0.000	Extra at Top
SuppTop	0.038	2.603	86.663	--	0.000	Extra at Top
SuppLeft	0.032	2.164	71.765	--	0.000	Extra at Top
SuppRight	0.032	2.164	71.765	--	0.000	Extra at Top

Minimum Steel Check :

As per clause 26.5.2.1 of IS 456:2000:

Minimum area of steel For Main Steel = 0.12 x C/S Area

= 132.000 mm^2

Notes :

Extra steel at Top support is computed considering
the bent-ups,if any,coming from the adjoining span.
It is the maximum of the extra steel required
for each slab at that common support.

Slab Name : S7

Slab Type : TwoWay Slab

Grade of Concrete :M20

Grade of Steel : Fe415

Dimensions : Lx = 3.150 m ,Ly = 3.625 m ,Thickness= 110 mm

Span Ratio : Longer/Shorter = 1.151

Boundary Condition : Internal Panel

Loading on Slab :(Table 1)

Total DL = (1) + (2) + (3)

Self Weight (1) kN/m²	Floor Finish (2) kN/m²	Sunk Load (3) kN/m²	Total DL (4) kN/m²	Live Load (5) kN/m²	Total LL (6) kN/m²	Total Design Load 1.5 x [(4)+(6)] kN/m²
2.750	1.000	0.000	3.750	2.000	2.000	8.625

Deflection Check :(Table 2)

As per clause 23.2.1 of IS 456:2000:

(Span/Depth) Ratio	Modification Factor (a)	Basic Factor (b)	Permissible Ratio (a x b)	Status
28.636	1.000	32.000	32.000	OK

Moment And Steel Calculations :

W = Total Design Load = 8.625 kN/m^2

L_x = Shorter span = 3.150 m

Position	Moment Coeff.α_x or α_y	Moment kN-m	A_{st_rqd} per metre mm^2	Steel Detail dia @ spc mm c/c	A_{st_prv} per metre mm^2	Remark
MidShort	0.030	2.570	85.534	#8 @ 200	251.327	Main
MidLong	0.024	2.054	68.058	#8 @ 200	251.327	Other
SuppDown	0.032	2.739	91.270	--	0.000	Extra at Top
SuppTop	0.032	2.739	91.270	--	0.000	Extra at Top
SuppLeft	0.040	3.427	114.902	--	0.000	Extra at Top
SuppRight	0.040	3.427	114.902	--	0.000	Extra at Top

Minimum Steel Check :

As per clause 26.5.2.1 of IS 456:2000:

Minimum area of steel For Main Steel = 0.12 x C/S Area

= 132.000 mm^2

Notes :

Extra steel at Top support is computed considering
the bent-ups,if any,coming from the adjoining span.
It is the maximum of the extra steel required
for each slab at that common support.

Slab Name : S8
Slab Type : TwoWay Slab
Grade of Concrete :M20
Grade of Steel : Fe415
Dimensions : Lx = 2.825 m ,Ly = 1.900 m ,Thickness= 110 mm
Span Ratio : Longer/Shorter = 1.487
Boundary Condition : Internal Panel

Loading on Slab :(Table 1)

Total DL = (1) + (2) + (3)

Self Weight (1) kN/m²	Floor Finish (2) kN/m²	Sunk Load (3) kN/m²	Total DL (4) kN/m²	Live Load (5) kN/m²	Total LL (6) kN/m²	Total Design Load 1.5 x [(4)+(6)] kN/m²
2.750	1.000	0.000	3.750	2.000	2.000	8.625

Deflection Check :(Table 2)

As per clause 23.2.1 of IS 456:2000:

(Span/Depth) Ratio	Modification Factor (a)	Basic Factor (b)	Permissible Ratio (a x b)	Status
17.273	1.000	32.000	32.000	OK

Moment And Steel Calculations :

W = Total Design Load = 8.625 kN/m^2

L_x = Shorter span = 1.900 m

Position	Moment Coeff.α_x or α_y	Moment kN-m	A_{st_rqd} per metre mm²	Steel Detail dia @ spc mm c/c	A_{st_prv} per metre mm²	Remark
MidShort	0.041	1.268	41.756	#8 @ 200	251.327	Main
MidLong	0.024	0.747	24.496	#8 @ 200	251.327	Other
SuppDown	0.053	1.642	54.223	--	0.000	Extra at Top
SuppTop	0.053	1.642	54.223	--	0.000	Extra at Top
SuppLeft	0.032	0.996	32.728	--	0.000	Extra at Top
SuppRight	0.032	0.996	32.728	--	0.000	Extra at Top

Minimum Steel Check :

As per clause 26.5.2.1 of IS 456:2000:

Minimum area of steel For Main Steel = 0.12 x C/S Area

= 132.000 mm^2

Notes :

Extra steel at Top support is computed considering
the bent-ups,if any,coming from the adjoining span.
It is the maximum of the extra steel required
for each slab at that common support.

Slab Name : S9
Slab Type : TwoWay Slab
Grade of Concrete :M20
Grade of Steel : Fe415
Dimensions : Lx = 2.825 m ,Ly = 3.700 m ,Thickness= 110 mm
Span Ratio : Longer/Shorter = 1.310
Boundary Condition : Internal Panel

Loading on Slab :(Table 1)

Total DL = (1) + (2) + (3)

Self Weight (1) kN/m²	Floor Finish (2) kN/m²	Sunk Load (3) kN/m²	Total DL (4) kN/m²	Live Load (5) kN/m²	Total LL (6) kN/m²	Total Design Load 1.5 x [(4)+(6)] kN/m²
2.750	1.000	0.000	3.750	2.000	2.000	8.625

Deflection Check :(Table 2)

As per clause 23.2.1 of IS 456:2000:

(Span/Depth) Ratio	Modification Factor (a)	Basic Factor (b)	Permissible Ratio (a x b)	Status
25.682	1.000	32.000	32.000	OK

S. B. Bhattacharyya
B.E. (Civil)
ESE- 116 /1

Moment And Steel Calculations :

W = Total Design Load = 8.625 kN/m^2

L_x = Shorter span = 2.825 m

Position	Moment Coeff.α_x or α_y	Moment kN-m	A_{st_rqd} per metre mm²	Steel Detail dia @ spc mm c/c	A_{st_prv} per metre mm²	Remark
MidShort	0.036	2.498	83.085	#8 @ 200	251.327	Main
MidLong	0.024	1.652	54.556	#8 @ 200	251.327	Other
SuppDown	0.032	2.203	73.076	--	0.000	Extra at Top
SuppTop	0.032	2.203	73.076	--	0.000	Extra at Top
SuppLeft	0.047	3.262	109.201	--	0.000	Extra at Top
SuppRight	0.047	3.262	109.201	--	0.000	Extra at Top

Minimum Steel Check :

As per clause 26.5.2.1 of IS 456:2000:

Minimum area of steel For Main Steel = 0.12 x C/S Area

= 132.000 mm^2

Notes :

Extra steel at Top support is computed considering the bent-ups,if any,coming from the adjoining span. It is the maximum of the extra steel required for each slab at that common support.

Slab Name : S10

Slab Type : TwoWay Slab

Grade of Concrete :M20

Grade of Steel : Fe415

Dimensions : Lx = 3.975 m ,Ly = 4.200 m ,Thickness= 110 mm

Span Ratio : Longer/Shorter = 1.057

Boundary Condition : One short edge discontinuous

Loading on Slab :(Table 1)

Total DL = (1) + (2) + (3)

Self Weight (1) kN/m²	Floor Finish (2) kN/m²	Sunk Load (3) kN/m²	Total DL (4) kN/m²	Live Load (5) kN/m²	Total LL (6) kN/m²	Total Design Load 1.5 x [(4)+(6)] kN/m²
2.750	1.000	0.000	3.750	2.000	2.000	8.625

Deflection Check :(Table 2)

As per clause 23.2.1 of IS 456:2000:

(Span/Depth) Ratio	Modification Factor (a)	Basic Factor (b)	Permissible Ratio (a x b)	Status
46.765	1.969	26.000	51.203	OK

Moment And Steel Calculations :

Coefficients (α_x and α_y) are computed as per clause D-1.1 of IS 456:2000:

$M_x = \alpha_x$ x W x L_x x L_x:

$M_y = \alpha_y$ x W x L_x x L_x:

where

M_x,M_y = moments on strips of unit width spanning L_x and L_y respectively,

W = Total Design Load = 8.625 kN/m^2

L_x = Shorter span = 3.975 m

Position	Moment Coeff.α_x or α_y	Moment kN-m	A_{st_rqd} per metre mm²	Steel Detail dia @ spc mm c/c	A_{st_prv} per metre mm²	Remark
MidShort	0.030	4.124	139.117	#8 @ 200	251.327	Main
MidLong	0.028	3.816	128.361	#8 @ 200	251.327	Other
SuppDown	0.037	5.042	171.483	--	0.000	Extra at Top
SuppTop	0.000	0.000	0.000	--	0.000	Extra at Top
SuppLeft	0.040	5.505	188.015	--	0.000	Extra at Top
SuppRight	0.040	5.505	188.015	--	0.000	Extra at Top

Minimum Steel Check :

As per clause 26.5.2.1 of IS 456:2000:

Beam Detail Report

Floor2 - Default Level

Load Combinations used in beam design :

Load Combi No	Load Combination	Load Combi No	Load Combination
1	1.50 DL + 1.50 LL	2	1.20 DL + 1.20 LL + 1.20 EQL X+
3	1.20 DL + 1.20 LL + 1.20 EQL X-	4	1.20 DL + 1.20 LL + 1.20 EQL Y+
5	1.20 DL + 1.20 LL + 1.20 EQL Y-	6	1.50 DL + 1.50 EQL X+
7	1.50 DL + 1.50 EQL X-	8	1.50 DL + 1.50 EQL Y+
9	1.50 DL + 1.50 EQL Y-	10	0.90 DL + 1.50 EQL X+
11	0.90 DL + 1.50 EQL X-	12	0.90 DL + 1.50 EQL Y+
13	0.90 DL + 1.50 EQL Y-	14	1.20 DL + 1.20 LL + 1.20 EQL Z+
15	1.20 DL + 1.20 LL + 1.20 EQL Z-	16	1.50 DL + 1.50 EQL Z+
17	1.50 DL + 1.50 EQL Z-	18	0.90 DL + 1.50 EQL Z+
19	0.90 DL + 1.50 EQL Z-		

Design of Beam Group : BG1

Beam : B1

Material Properties :

fck = 20.000 N/mm²
fy = 415.000 N/mm²

1.Design Data :

1.1)Dimensions:

Beam Type = 'Rectangular Beam'
Width (b) = 230 mm
Total depth (D) = 300 mm
Effective Tension Cover = 33mm
Effective Compression Cover = 33mm
Clear side Cover = 25mm
Effective depth (d) = 267mm
Span = 1.300m
Clear Span (L) = 1.300m

1.2)Loading on the Beam :

1.2.1)Self wt. of beam : -1.725 kN/m
1.2.2)Slab Load
Dead Load :
UDL in kN/m
St.Pt 0.000 m L = 1.300 m Max. Value = -0.705 kN/m
LiveLoad
UDL in kN/m
St.Pt 0.000 m L = 1.300 m Max. Value = -0.376 kN/m

1.2.3)Wall Load
UDL in kN/m
St.Pt 0.000 m L = 1.300 m Max. Value = -3.848 kN/m

2.Computation of Area of steel :

2.1) Flexural Steel at Top and Bottom
2.1.1)Computation of theoretical steel

At length	0.000L	0.100L	0.200L	0.300L	0.400L	0.500L	0.600L	0.700L	0.800L	0.900L	1.000L
B.M. Hog in kN-m	8.641	7.647	6.749	5.947	5.239	4.628	4.111	3.691	3.365	3.136	3.001
Load Combi-No	11	11	11	11	11	11	11	11	11	11	11
B.M. Sag in kN-m	15.154	15.812	16.311	16.650	16.831	16.852	16.715	16.418	15.962	15.347	14.573
Load Combi-No	6	6	6	6	6	6	6	6	6	6	6
Torsion in kN-m	11.557	11.557	11.557	11.557	11.557	11.557	11.557	11.557	11.557	11.557	11.557
Load Combi-No	9	9	9	9	9	9	9	9	9	9	9
Eq.B.M.Hog in kN-m	0.512	7.647	6.749	5.947	5.239	4.628	4.111	3.691	3.365	0.319	1.093
Eq.B.M.Sag in kN-m	30.820	31.478	31.976	32.316	32.497	32.518	32.381	32.084	31.628	31.013	30.239
Top $A_{st\_req.}$ in mm²	278.319	265.690	254.387	244.373	235.617	228.093	221.778	216.655	212.708	209.928	208.305
Bot $A_{st\_req.}$ in mm²	364.628	373.723	380.668	385.423	387.959	388.261	386.327	382.168	375.810	367.290	356.658

Max.Area of steel req. at Top = 278.319 mm² at 0.000L
Max.Area of steel req. at Bottom = 388.261 mm² at 0.500L
2.1.2)Minimum and Maximum Steel Checks :
a)Minimum Steel Check as per IS 456:2000 Clause 26.5.1.1(a)

A_{st} = (0.85 x b x d) / f_y
= 125.780 mm²

b)Maximum Steel Check as per IS 456:2000 Clause 26.5.1.1(b)

A_{st} = 0.04 x b x D
= 2760.000 mm²

2.1.3)Provided Steel
2.1.3.1)Provided Bottom Steel :

Straight bars = 2 - #16
BentUp bars = ---
Total Area of Steel at bottom at 0.500L = 402.124 mm² > $A_{st\_req.}$ (O.K.)

2.1.3.2)Provided Top Steel :

Straight bars = 2 - #10 +1 - #12
Extra bars at Left = ---
Extra bars at Right = ---
Total Area of Steel at Top
At Left Support = 270.177 mm²
At Right Support = 270.177 mm²

2.2)Computation of Shear Reinforcement(Stirrups) :

τ_{auc} = 0.532 N/mm²
S_{vmin} = 200 mm

Note :
Design Shear = (Equivalent S.F.)-(V_{uc})
V_{usT} = (T_u/b_1 + V_u/2.5)(d/d_1)
Where,
T_u = Torsional moment,
V_u = Ultimate shear force
b_1 = Center to center distance between corner bars in the direction of width
d_1 = Center to center distance between corner bars in the direction of depth
(As per Clause 41.4.3 of IS 456:2000)
If Design Shear < V_{usT} then Design Shear = V_{usT}
V_{usT} = Minimum shear for torsional transverse Reinforcement
2.2.1) Computation of Steel

At length	0.000L	0.100L	0.200L	0.300L	0.400L	0.500L	0.600L	0.700L	0.800L	0.900L	1.000L
S.F.in kN	14.274	13.050	11.826	10.602	9.378	8.154	6.930	-5.738	-6.962	-8.186	-9.410
T_u in kN-m	11.557	11.557	11.557	11.557	11.557	11.557	11.557	11.557	11.557	11.557	11.557

Eq. S.F. in kN	94.673	93.449	92.225	91.001	89.777	88.553	87.328	86.137	87.361	88.585	89.809
V_{usT} in kN	106.545	105.923	105.300	104.678	104.055	103.433	102.810	102.204	102.827	103.449	104.072
Vuc in kN	32.657	32.657	32.657	32.657	32.657	32.657	32.657	32.657	32.657	32.657	32.657
Design Shear in kN	106.545	105.923	105.300	104.678	104.055	103.433	102.810	102.204	102.827	103.449	104.072
Spacing req. in mm for 10 Dia	131	131	132	133	134	134	135	136	135	134	134

2.2.2)Provided Shear Steel (Stirrups) :

Nominal Zone :#10 @ 100 mm c/c from 0.000 to 1.300 .

Note : Nominal zone is where No Shear Reinforcement is needed.
Hence provide minimum Shear Reinforcement of 0.75 d
(As per Clause 26.5.1.5 of IS 456:2000)

Design of Beam Group : BG2

Beam : B2

Material Properties :

fck = 20.000 N/mm²

fy = 415.000 N/mm²

1.Design Data :

1.1)Dimensions:

Beam Type = 'Rectangular Beam'
Width (b) = 230 mm
Total depth (D) = 350 mm
Effective Tension Cover = 33mm
Effective Compression Cover = 33mm
Clear side Cover = 25mm
Effective depth (d) = 317mm
Span = 1.300m
Clear Span (L) = 1.300m

1.2)Loading on the Beam :

1.2.1)Self wt. of beam : -2.013 kN/m
1.2.2)Slab Load
Dead Load :
UDL in kN/m
St.Pt 0.000 m L = 1.300 m Max. Value = -0.705 kN/m
LiveLoad
UDL in kN/m
St.Pt 0.000 m L = 1.300 m Max. Value = -0.376 kN/m

1.2.3)Wall Load
UDL in kN/m
St.Pt 0.000 m L = 1.300 m Max. Value = -10.260 kN/m

2.Computation of Area of steel :

2.1) Flexural Steel at Top and Bottom
2.1.1)Computation of theoretical steel

At length	0.000L	0.100L	0.200L	0.300L	0.400L	0.500L	0.600L	0.700L	0.800L	0.900L	1.000L
B.M. Hog in kN-m	55.979	45.111	34.564	24.906	17.704	10.696	3.880	0.000	0.000	0.000	0.000
Load Combi-No	7	7	7	11	11	11	11	1	1	1	1
B.M. Sag in kN-m	22.736	24.218	25.508	27.171	30.777	34.061	37.023	39.664	41.982	43.980	47.888
Load Combi-No	10	10	10	6	6	6	6	6	6	6	1
Torsion in kN-m	15.220	15.220	15.220	15.220	15.220	15.220	15.220	15.220	15.220	15.220	15.220
Load Combi-No	9	9	9	9	9	9	9	9	9	9	9
Eq.B.M.Hog in kN-m	78.557	67.689	57.142	24.906	17.704	10.696	3.880	0.000	0.000	0.000	0.000

Eq.B.M.Sag in kN-m	22.736	24.218	25.508	49.749	53.355	56.639	59.601	62.241	64.560	66.557	70.465
Top $A_{st\_req.}$ in mm²	842.076	736.082	602.611	480.611	396.750	319.828	248.783	0.000	7.896	27.947	67.184
Bot $A_{st\_req.}$ in mm²	454.771	472.371	487.878	508.160	553.298	595.935	635.786	672.543	705.571	725.048	763.162

Max.Area of steel req. at Top = 842.076 mm² at 0.000L
Max.Area of steel req. at Bottom = 763.162 mm² at 1.000L
2.1.2)Minimum and Maximum Steel Checks :
a)Minimum Steel Check as per IS 456:2000 Clause 26.5.1.1(a)

A_{st} = (0.85 x b x d) / f_y

= 149.334 mm²

b)Maximum Steel Check as per IS 456:2000 Clause 26.5.1.1(b)

A_{st} = 0.04 x b x D

= 3220.000 mm²

2.1.3)Provided Steel
2.1.3.1)Provided Bottom Steel :

Straight bars = 4 - #16

BentUp bars = ---

Total Area of Steel at bottom at 1.000L = 804.248 mm² > $A_{st\_req.}$ (O.K.)

2.1.3.2)Provided Top Steel :

Straight bars = 3 - #16

Extra bars at Left = +2 - #10 0.000,0.564 +1 - #12 0.000,0.564

Extra bars at Right = 2 - #10 0.564,0.564

Total Area of Steel at Top

At Left Support = 873.363 mm²

At Right Support = 760.265 mm²

2.2)Computation of Shear Reinforcement(Stirrups) :

τ_{auc} = 0.645 N/mm²

S_{vmin} = 238 mm

Note :
Design Shear = (Equivalent S.F.)-(V_{uc})
V_{usT} = (T_u/b_1 + $V_u/2.5$)(d/d_1)
Where,
T_u = Torsional moment,
V_u = Ultimate shear force
b_1 = Center to center distance between corner bars in the direction of width
d_1 = Center to center distance between corner bars in the direction of depth
(As per Clause 41.4.3 of IS 456:2000)
If Design Shear < V_{usT} then Design Shear = V_{usT}
V_{usT} = Minimum shear for torsional transverse Reinforcement
2.2.1) Computation of Steel

At length	0.000L	0.100L	0.200L	0.300L	0.400L	0.500L	0.600L	0.700L	0.800L	0.900L	1.000L
S.F.in kN	84.840	82.366	79.891	77.417	74.942	72.467	69.993	67.518	65.044	62.569	60.095
T_u in kN-m	15.220	15.220	15.220	15.220	15.220	15.220	15.220	15.220	15.220	15.220	15.220
Eq. S.F. in kN	190.721	188.247	185.772	183.298	180.823	178.349	175.874	173.400	170.925	168.451	165.976
V_{usT} in kN	166.763	165.556	164.349	163.142	161.935	160.728	159.522	158.315	157.108	155.901	154.694
Vuc in kN	48.379	48.379	48.379	48.379	48.379	47.003	47.003	47.003	47.003	47.003	47.003
Design Shear in kN	166.763	165.556	164.349	163.142	161.935	160.728	159.522	158.315	157.108	155.901	154.694
Spacing req. in mm for 10 Dia	101	101	102	103	104	104	105	106	107	108	108

2.2.2)Provided Shear Steel (Stirrups) :

S. B. Bhattacharyya
B.E. (Civil)
ESE- 116 /1

Nominal Zone :#10 @ 100 mm c/c from 0.000 to 1.300 .
Note : Nominal zone is where No Shear Reinforcement is needed.
Hence provide minimum Shear Reinforcement of 0.75 d
(As per Clause 26.5.1.5 of IS 456:2000)

Design of Beam Group : BG3

Beam : B3

Material Properties :

fck = 20.000 N/mm²

fy = 415.000 N/mm²

1.Design Data :

1.1)Dimensions:

Beam Type	= 'Rectangular Beam'
Width (b)	= 230 mm
Total depth (D)	= 475 mm
Effective Tension Cover	= 33mm
Effective Compression Cover	= 33mm
Clear side Cover	= 25mm
Effective depth (d)	= 442mm
Span	= 0.050m
Clear Span (L)	= 0.050m

1.2)Loading on the Beam :

1.2.1)Self wt. of beam : -2.731 kN/m

1.2.3)Wall Load
UDL in kN/m
St.Pt 0.000 m L = 0.050 m Max. Value = -10.260 kN/m

2.Computation of Area of steel :

2.1) Flexural Steel at Top and Bottom
2.1.1)Computation of theoretical steel

At length	0.000L	0.100L	0.200L	0.300L	0.400L	0.500L	0.600L	0.700L	0.800L	0.900L	1.000L
B.M. Hog in kN-m	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Load Combi-No	1	1	1	1	1	1	1	1	1	1	1
B.M. Sag in kN-m	60.228	60.237	60.246	60.255	60.263	60.270	60.278	60.284	60.291	60.297	60.302
Load Combi-No	6	6	6	6	6	6	6	6	6	6	6
Torsion in kN-m	30.907	30.907	30.907	30.907	30.907	30.907	30.907	30.907	30.907	30.907	30.907
Load Combi-No	9	9	9	9	9	9	9	9	9	9	9
Eq.B.M.Hog in kN-m	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Eq.B.M.Sag in kN-m	115.955	115.964	115.973	115.982	115.990	115.998	116.005	116.012	116.018	116.024	116.029
Top A_{st_req}. in mm²	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Bot A_{st_req}. in mm²	887.305	887.399	887.487	887.572	887.652	887.727	887.798	887.865	887.927	887.985	888.039

Max.Area of steel req. at Top = 0.000 mm² at 0.000L
Max.Area of steel req. at Bottom = 888.039 mm² at 1.000L
2.1.2)Minimum and Maximum Steel Checks :
a)Minimum Steel Check as per IS 456:2000 Clause 26.5.1.1(a)

A_{st} = (0.85 x b x d) / f_y

= 208.219 mm²

b)Maximum Steel Check as per IS 456:2000 Clause 26.5.1.1(b)

A_{st} = 0.04 x b x D

= 4370.000 mm²

2.1.3)Provided Steel
2.1.3.1)Provided Bottom Steel :

Straight bars = 3 - #16 +3 - #12

BentUp bars = ---

Total Area of Steel at bottom at 1.000L = 942.478 mm² > $A_{st\_req.}$ (O.K.)

2.1.3.2)Provided Top Steel :

Straight bars = 2 - #10

Extra bars at Left = +2 - #10 0.564,0.564

Extra bars at Right = 2 - #10 0.012,0.806

Total Area of Steel at Top

At Left Support = 314.159 mm²

At Right Support = 314.159 mm²

2.2)Computation of Shear Reinforcement(Stirrups) :

τ_{auc} = 0.606 N/mm²

S_{vmin} = 332 mm

Note :
Design Shear = (Equivalent S.F.)-(V_{uc})
V_{usT} = (T_u/b_1 + $V_u/2.5$)(d/d_1)
Where,
T_u = Torsional moment,
V_u = Ultimate shear force
b_1 = Center to center distance between corner bars in the direction of width
d_1 = Center to center distance between corner bars in the direction of depth
(As per Clause 41.4.3 of IS 456:2000)
If Design Shear < V_{usT} then Design Shear = V_{usT}
V_{usT} = Minimum shear for torsional transverse Reinforcement
2.2.1) Computation of Steel

At length	0.000L	0.100L	0.200L	0.300L	0.400L	0.500L	0.600L	0.700L	0.800L	0.900L	1.000L
S.F.in kN	56.142	56.052	55.962	55.872	55.783	55.693	55.603	55.513	55.423	55.333	55.243
T_u in kN-m	30.907	30.907	30.907	30.907	30.907	30.907	30.907	30.907	30.907	30.907	30.907
Eq. S.F. in kN	271.147	271.057	270.967	270.877	270.787	270.697	270.607	270.517	270.428	270.338	270.248
V_{usT} in kN	265.530	265.489	265.447	265.406	265.365	265.323	265.282	265.241	265.200	265.158	265.117
Vuc in kN	61.579	61.579	61.579	61.579	61.579	61.579	61.579	61.579	61.579	61.579	61.579
Design Shear in kN	265.530	265.489	265.447	265.406	265.365	265.323	265.282	265.241	265.200	265.158	265.117
Spacing req. in mm for 12 Dia	124	124	124	124	124	124	124	124	124	124	124

2.2.2)Provided Shear Steel (Stirrups) :

Nominal Zone :#12 @ 100 mm c/c from 0.000 to 0.100 .

Note : Nominal zone is where No Shear Reinforcement is needed.
Hence provide minimum Shear Reinforcement of 0.75 d
(As per Clause 26.5.1.5 of IS 456:2000)

Design of Beam Group : BG4

Beam : B4

Material Properties :

fck = 20.000 N/mm²

fy = 415.000 N/mm²

1.Design Data :

1.1)Dimensions:

Beam Type = 'Rectangular Beam'

Width (b) = 230 mm

Total depth (D) = 300 mm

Effective Tension Cover = 33mm

Effective Compression Cover = 33mm

Clear side Cover = 25mm

Effective depth (d) = 267mm

S. B. Bhattacharyya
B.E. (Civil)
ESE- 116 /1

Span = 3.225m

Clear Span (L) = 3.225m

1.2)Loading on the Beam :

1.2.1)Self wt. of beam : -1.725 kN/m

1.2.2)Slab Load

Dead Load :

Trapezoidal Load in kN/m

St.Pt 1.375 m L = 3.225 m Max. Value = -5.156 kN/m

LiveLoad

Trapezoidal Load in kN/m

St.Pt 1.375 m L = 3.225 m Max. Value = -2.750 kN/m

1.2.3)Wall Load

UDL in kN/m

St.Pt 0.000 m L = 3.225 m Max. Value = -10.260 kN/m

2.Computation of Area of steel :

2.1) Flexural Steel at Top and Bottom

2.1.1)Computation of theoretical steel

At length	0.000L	0.100L	0.200L	0.300L	0.400L	0.500L	0.600L	0.700L	0.800L	0.900L	1.000L
B.M. Hog in kN-m	0.000	0.000	0.000	0.000	3.205	17.318	33.035	50.884	76.465	104.294	134.181
Load Combi-No	1	1	1	1	10	10	10	6	6	6	6
B.M. Sag in kN-m	57.793	54.607	52.752	56.921	58.654	57.776	54.224	48.626	46.863	43.751	39.405
Load Combi-No	6	1	7	7	7	7	7	11	11	11	11
Torsion in kN-m	6.244	6.244	6.244	6.244	6.244	6.244	6.244	6.244	6.244	6.244	6.244
Load Combi-No	8	8	8	8	8	8	8	8	8	8	8
Eq.B.M.Hog in kN-m	0.000	0.000	0.000	0.000	3.205	17.318	33.035	59.347	84.929	112.757	142.645
Eq.B.M.Sag in kN-m	66.256	63.070	61.215	65.384	67.118	66.239	62.688	48.626	46.863	43.751	39.405
Top $A_{st\_req.}$ in mm²	257.809	218.725	195.962	247.113	268.380	297.313	522.853	754.770	1057.562	1386.951	1740.706
Bot $A_{st\_req.}$ in mm²	836.549	798.841	776.880	826.229	846.748	836.351	794.314	728.044	707.177	828.310	1194.981

Max.Area of steel req. at Top = 1740.706 mm² at 1.000L

Max.Area of steel req. at Bottom = 1194.981 mm² at 1.000L

2.1.2)Minimum and Maximum Steel Checks :

a)Minimum Steel Check as per IS 456:2000 Clause 26.5.1.1(a)

A_{st} = (0.85 x b x d) / f_y

= 125.780 mm²

b)Maximum Steel Check as per IS 456:2000 Clause 26.5.1.1(b)

A_{st} = 0.04 x b x D

= 2760.000 mm²

2.1.3)Provided Steel

2.1.3.1)Provided Bottom Steel :

Straight bars = 6 - #16

BentUp bars = ---

Total Area of Steel at bottom at 1.000L = 1206.372 mm² > $A_{st\_req.}$ (O.K.)

2.1.3.2)Provided Top Steel :

Straight bars = 3 - #20 +2 - #12

Extra bars at Left = +2 - #10 0.012,0.806

Extra bars at Right = 3 - #16 0.806,0.000

Total Area of Steel at Top

At Left Support = 1325.752 mm²

At Right Support = 1771.858 mm²

2.2)Computation of Shear Reinforcement(Stirrups) :

τ_{auc} = 0.874 N/mm²

Terms Used In Calculation :

In biaxial column design,
$P_u = P_{uc} + P_{us(Total)}$

where,
P_u = external axial compressive load,
P_{uc} = axial compressive resistance offered by concrete,
$P_{us(Total)}$ = total axial compressive resistance offered by steel at different levels in the section.

$M_u = M_{uc} + M_{us(Total)}$

where,
M_u = external moment about centroidal axis,
M_{uc} = moment of resistance offered by concrete in compression,
$M_{us(Total)}$ = total moment of resistance offered by steel at different levels in the section.

i = serial no. of row of reinforcement,
A_{si} = cross-sectional area of reinforcement in the i th row,
f_{si} = stress in the reinforcement in the i th row (compression + ve, tension - ve),
f_{ci} = compressive stress in concrete at the level of i th row of reinforcement,
e_i = strain at the i th row of reinforcement from the stress-strain curve of steel and concrete,
x_i = distance of the bars in the i th row from the centroid of the section

Values of stress in steel :
Design strength in bending compression (f_{yd}) = 0.87 f_y

(a) Fe 250,
For $e_i >= f_{yd} / E_s$, $f_{si} = f_{yd}$
For $e_i < f_{yd} / E_s$, $f_{si} = e_i \times f_{yd}$

(b) Fe 415,
For $e_i >= 0.8 \times f_{yd} / E_s$, f_{si} = value obtained from stress-strain curve
For $e_i < 0.8 \times f_{yd} / E_s$, $f_{si} = e_i \times f_{yd}$

Stress	Stress level (N/mm²)	Strain
0.800 f_{yd}	288.7	0.00144
0.850 f_{yd}	306.7	0.00163
0.900 f_{yd}	324.8	0.00192
0.950 f_{yd}	342.8	0.00241
0.975 f_{yd}	351.8	0.00276
1.000 f_{yd}	360.9	0.00380

Values of stress in concrete :
For $e_i >= 0.002$, $f_{ci} = 0.446\ f_{ck}$
For $e_i < 0.002$, $f_{ci} = (446.e_i \times (1 - 250.e_i)) \times f_{ck}$

Values of strain :
Values of strain at different levels are obtained by taking maximum strain as 0.0035 as the reference value at the highly compressed edge.

Design of CG1 :

General Design Parameters :

Below Floor6 - Default Level at 16.450 m

Column size, (B x D) = 230 x 525 mm   Column height,(L) = 2750 mm

From analysis results,loads on column

Axial load,(P) = 126.08 kN

MomentX,(M_x) = 9.52 kN-m

MomentY,(M_y) = 62.06 kN-m

f_{ck} = 20.00 N/mm²

f_y = 415.00 N/mm²

Load combination = 1.50 DL + 1.50 EQL X+

Column Name	Orientation Angle
C1	0.00

Check For Slenderness :

Slenderness RatioX = (L_o x Effective Length FactorX) / Depth

= (2750 x 1.000) / 525 = 5.24 <= 12.00,

column is not slender in this direction.

Slenderness RatioY = (L_o x Effective Length FactorY) / Width

= (2750 x 1.000) / 230 = 11.96 <= 12.00,

column is not slender in this direction.

$M_{x\_MinEccen}$ = 2.900 kN.m

$M_{y\_MinEccen}$ = 2.522 kN.m

$M_x = \max(M_x, M_{x\_MinEccen}) + M_{uaddX}$ = 9.523 kN.m

$M_y = \max(M_y, M_{y\_MinEccen}) + M_{uaddY}$ = 62.062 kN.m

Calculation Of Eccentricities :

As per IS 456: 2000 Clause 25.4,all columns shall be designed for minimum eccentricity equal to the unsupported length of the column/500 plus lateral dimension/30, subject to a minimum of 20 mm.

Actual $eccen_X$ = Mx / P = 9.52/126.08 = 76 mm

Actual $eccen_Y$ = M_y / P = 62.06/126.08 = 492 mm

$eccen_{XMin}$ = (L/500) + (D/30) = (2750/500) + (525/30) = 23 mm

$eccen_{XMin}$ = max(23,20)= 23 mm

$eccen_{YMin}$ = (L/500) + (B/30) = (2750/500) + (230/30) = 13 mm

$eccen_{YMin}$ = max(13,20) = 23 mm

$eccen_X$ = max(Actual $eccen_X$,$eccen_{XMin}$) = max(76,23) = 76 mm

$eccen_Y$ = max(Actual $eccen_Y$,$eccen_{YMin}$) = max(492,20) = 492 mm

As per IS 456: 2000 Clause 39.3,
when the minimum eccentricity does not exceed 0.05 times the lateral dimension,
the column will be designed as an axially loaded column.
eccenX(76 mm) > 0.05 x 230(12 mm)
and eccenY(492 mm) > 0.05 x 525(26 mm)
Hence the column will be designed as a column with biaxial moments.
Provided steel :
Provide #12 - 10 nos. + #16 - 6 nos. (2337 mm²)

Biaxial Check Calculations :

For X axis :

Row No. (i)	A_{si} (mm²)	e_i	f_{si} (N/mm²)	f_{ci} (N/mm²)	(f_{si} - f_{ci}) (N/mm²)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	402	0.00268	349.76	8.92	340.84	137.06	215	29.40
2	226	0.00162	305.77	8.60	297.18	67.22	152	10.24
3	226	0.00059	118.76	4.51	114.25	25.84	92	2.38
4	226	-0.00043	-86.53	0.00	-86.53	-19.57	32	-0.63
5	402	-0.00149	-293.74	0.00	-293.74	-118.12	-30	3.55
6	226	-0.00255	-346.50	0.00	-346.50	-78.38	-92	7.23
7	226	-0.00358	-358.98	0.00	-358.98	-81.20	-152	12.37
8	402	-0.00464	-360.90	0.00	-360.90	-145.13	-215	31.13
					Total	-212.27		95.68

x_{ux} = 205 mm $\qquad P_{uc} = C_1.f_{ck}.B.D$

C_1 = 0.36 k_u = 0.36 x 205/525 = 0.141

P_{uc} = 0.141 x 20.00 x 230 x 525

= 339.61 kN

$P_{ux1} = P_{uc} + P_{us(Total)}$

= 339.61 + (-212.27)

= 127.34 kN > P ,hence O.K.

$M_{uc} = P_{uc}.(0.5.D - C_2.D)$

C_2 = 0.416 k_u = 0.416 x 205/525

= 0.163

M_{uc} = 339.61 x (0.5 x 525 - 0.163 x 525) = 60.17 kN-m

M_{ux1} = M_{uc} + $M_{us(Total)}$

= 60.17 + (95.68)

= 155.85 kN-m

For Y axis :

Row No. (i)	A_{si} (mm²)	e_i	f_{si} (N/mm²)	f_{ci} (N/mm²)	(f_{si} - f_{ci}) (N/mm²)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	1169	0.00120	240.42	7.50	232.92	272.21	67	18.24
2	1169	-0.00521	-360.90	0.00	-360.90	-421.77	-67	28.26
					Total	-149.57		46.50

x_{uy} = 73 mm $P_{uc} = C_1.f_{ck}.B.D$

C_1 = 0.36 k_u = 0.36 x 73/230 = 0.114

P_{uc} = 0.114 x 20.00 x 525 x 230

= 276.36 kN

P_{uy1} = P_{uc} + $P_{us(Total)}$

= 276.36 + (-149.57)

= 126.79 kN > P ,hence O.K.

M_{uc} = P_{uc}.(0.5.B - C_2.B)

C_2 = 0.416 k_u = 0.416 x 73/230

= 0.132

M_{uc} = 276.36 x (0.5 x 230 - 0.132 x 230) = 23.38 kN-m

M_{uy1} = M_{uc} + $M_{us(Total)}$

= 23.38 + (46.50)

= 69.87 kN-m

P_u/P_{uz} = 0.070

For P_u/P_{uz} <= 0.2, α_n = 1.0

For P_u/P_{uz} >= 0.8, α_n = 2.0

For P_u/P_{uz} = 0.070, α_n = 1.000

$((M_{ux} / M_{ux1})^{\alpha_n)} + ((M_{uy}/M_{uy1})^{\alpha_n})$

= $((9.52/155.85)^{1.000}) + ((62.06/69.87)^{1.000})$

= 0.949 < 1.0,hence O.K.

Ties Details :

Load combination for ties design = 1.50 DL + 1.50 EQL X+

Calculation of diameter of ties :

As per IS 456: 2000 Clause 26.5.3.2 (c), the diameter of the polygonal links or lateral ties shall be not less than the following

i) one-fourth the diameter of the largest longitudinal bar = 16/4 = 4 mm

ii) in no case less than = 6 mm

Required diameter = maximum of (4, 6) = 6 mm

Provide 8 mm diameter for ties.

Calculation of spacing of ties :

As per IS 456: 2000 Clause 26.5.3.2 (c), the pitch of transverse reinforcement shall be not more than the least of the following distances:

i) the least lateral dimension of the compression member = 230 mm

ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied = 16 x 12 = 192 mm

iii) 300 mm

Required spacing = minimum of (230, 192, 300) = 192 mm

Provide ties 8 @ 190 mm c/c

SUMMARY :

CG1 : Floor6 - Default Level at 16.450 m
Provide rectangular section : 230 x 525 mm
Provide #12 - 10 nos. + #16 - 6 nos. (2337 mm²)
Provide ties #8 @ 190 mm c/c
Provide 4 legged ties along width and
2 legged ties along depth.

General Design Parameters :

Below Floor5 - Default Level at 13.400 m

Column size, (B x D) = 230 x 525 mm   Column height,(L) = 2750 mm

From analysis results,loads on column

Axial load,(P) = 256.02 kN

MomentX,(M_x) = 3.35 kN-m

MomentY,(M_y) = 69.83 kN-m

f_{ck} = 20.00 N/mm²

f_y = 415.00 N/mm²

Load combination = 1.50 DL + 1.50 EQL X+

Column Name	Orientation Angle
C1	0.00

Check For Slenderness :

Slenderness RatioX = (L_o x Effective Length FactorX) / Depth

= (2750 x 1.000) / 525 = 5.24 <= 12.00,

column is not slender in this direction.

Slenderness RatioY = (L_o x Effective Length FactorY) / Width

= (2750 x 1.000) / 230 = 11.96 <= 12.00,

column is not slender in this direction.

$M_{x\_MinEccen}$ = 5.888 kN.m

$M_{y\_MinEccen}$ = 5.120 kN.m

M_x = max(M_x,$M_{x\_MinEccen}$) + M_{uaddX} = 5.888 kN.m

M_y = max(M_y,$M_{y\_MinEccen}$) + M_{uaddY} = 69.827 kN.m

Calculation Of Eccentricities :

As per IS 456: 2000 Clause 25.4,all columns shall be designed for minimum eccentricity equal to the unsupported length of the column/500 plus lateral dimension/30, subject to a minimum of 20 mm.

Actual $eccen_X$ = Mx / P = 3.35/256.02 = 13 mm

Actual $eccen_Y$ = M_y / P = 69.83/256.02 = 273 mm

$eccen_{XMin}$ = (L/500) + (D/30) = (2750/500) + (525/30) = 23 mm

$eccen_{XMin}$ = max(23,20)= 23 mm

$eccen_{YMin}$ = (L/500) + (B/30) = (2750/500) + (230/30) = 13 mm

$eccen_{YMin}$ = max(13,20) = 23 mm

$eccen_X$ = max(Actual $eccen_X$,$eccen_{XMin}$) = max(13,23) = 23 mm

$eccen_Y$ = max(Actual $eccen_Y$,$eccen_{YMin}$) = max(273,20) = 273 mm

As per IS 456: 2000 Clause 39.3,
when the minimum eccentricity does not exceed 0.05 times the lateral dimension,
the column will be designed as an axially loaded column.
eccenX(23 mm) > 0.05 x 230(12 mm)
and eccenY(273 mm) > 0.05 x 525(26 mm)
Hence the column will be designed as a column with biaxial moments.

Provided steel :
Provide #12 - 10 nos. + #16 - 6 nos. (2337 mm²)

Biaxial Check Calculations :

For X axis :

Row No. (i)	A_{si} (mm²)	e_i	f_{si} (N/mm²)	f_{ci} (N/mm²)	(f_{si} - f_{ci}) (N/mm²)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	402	0.00277	351.90	8.92	342.98	137.92	215	29.58
2	226	0.00183	319.13	8.85	310.27	70.18	152	10.69
3	226	0.00092	183.34	6.30	177.04	40.04	92	3.69
4	226	0.00000	0.86	0.04	0.82	0.19	32	0.01
5	402	-0.00094	-187.68	0.00	-187.68	-75.47	-30	2.27
6	226	-0.00188	-322.38	0.00	-322.38	-72.92	-92	6.72
7	226	-0.00279	-352.09	0.00	-352.09	-79.64	-152	12.13
8	402	-0.00374	-360.34	0.00	-360.34	-144.90	-215	31.08
					Total	-124.60		96.18

x_{ux} = 231 mm P_{uc} = $C_1.f_{ck}.B.D$

C_1 = 0.36 k_u = 0.36 x 231/525 = 0.158

P_{uc} = 0.158 x 20.00 x 230 x 525

= 382.06 kN

$P_{ux1} = P_{uc} + P_{us(Total)}$

= 382.06 + (-124.60)

= 257.46 kN > P ,hence O.K.

$M_{uc} = P_{uc}.(0.5.D - C_2.D)$

$C_2 = 0.416\ k_u$ = 0.416 x 231/525

= 0.183

M_{uc} = 382.06 x (0.5 x 525 - 0.183 x 525) = 63.62 kN-m

$M_{ux1} = M_{uc} + M_{us(Total)}$

= 63.62 + (96.18)

= 159.81 kN-m

For Y axis :

Row No. (i)	A_{si} (mm²)	e_i	f_{si} (N/mm²)	f_{ci} (N/mm²)	(f_{si} - f_{ci}) (N/mm²)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	1169	0.00160	303.55	8.56	295.00	344.75	67	23.10
2	1169	-0.00372	-360.17	0.00	-360.17	-420.92	-67	28.20
					Total	-76.17		51.30

x_{uy} = 88 mm $P_{uc} = C_1.f_{ck}.B.D$

$C_1 = 0.36\ k_u$ = 0.36 x 88/230 = 0.138

P_{uc} = 0.138 x 20.00 x 525 x 230

= 333.67 kN

$P_{uy1} = P_{uc} + P_{us(Total)}$

= 333.67 + (-76.17)

= 257.50 kN > P ,hence O.K.

$M_{uc} = P_{uc}.(0.5.B - C_2.B)$

$C_2 = 0.416\ k_u$ = 0.416 x 88/230

= 0.160

M_{uc} = 333.67 x (0.5 x 230 - 0.160 x 230) = 26.12 kN-m

$M_{uy1} = M_{uc} + M_{us(Total)}$

= 26.12 + (51.30)

= 77.42 kN-m

P_u/P_{uz} = 0.143

For P_u/P_{uz} <= 0.2, α_n = 1.0

For P_u/P_{uz} >= 0.8, α_n = 2.0

For P_u/P_{uz} = 0.143, α_n = 1.000

$((M_{ux} / M_{ux1})^{\alpha_n)} + ((M_{uy}/M_{uy1})^{\alpha_n})$

$= ((5.89/159.81)^{1.000}) + ((69.83/77.42)^{1.000})$

= 0.939 < 1.0,hence O.K.

Ties Details :

Load combination for ties design = 1.50 DL + 1.50 EQL X+

Calculation of diameter of ties :

As per IS 456: 2000 Clause 26.5.3.2 (c), the diameter of the polygonal links or lateral ties shall be not less than the following

i) one-fourth the diameter of the largest longitudinal bar = 16/4 = 4 mm

ii) in no case less than = 6 mm

Required diameter = maximum of (4, 6) = 6 mm

Provide 8 mm diameter for ties.

Calculation of spacing of ties :

As per IS 456: 2000 Clause 26.5.3.2 (c), the pitch of transverse reinforcement shall be not more than the least of the following distances:

i) the least lateral dimension of the compression member = 230 mm

ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied = 16 x 12 = 192 mm

iii) 300 mm

Required spacing = minimum of (230, 192, 300) = 192 mm

Provide ties   8 @ 190 mm c/c

SUMMARY :

CG1 : Floor5 - Default Level at 13.400 m
Provide rectangular section : 230 x 525 mm
Provide #12 - 10 nos. + #16 - 6 nos. (2337 mm²)
Provide ties #8 @ 190 mm c/c
Provide 4 legged ties along width and
2 legged ties along depth.

General Design Parameters :

Below Floor4 - Default Level at 10.350 m

Column size, (B x D) = 230 x 600 mm   Column height,(L) = 2750 mm

From analysis results,loads on column

Axial load,(P) = 386.43 kN

MomentX,(M_x) = 3.36 kN-m

MomentY,(M_y) = 78.78 kN-m

f_{ck} = 20.00 N/mm²

f_y = 415.00 N/mm²

Load combination = 1.50 DL + 1.50 EQL X+

Column Name	Orientation Angle
C1	0.00

Check For Slenderness :

Slenderness RatioX = (L_o x Effective Length FactorX) / Depth

= (2750 x 1.000) / 600 = 4.58 <= 12.00,

column is not slender in this direction.

Slenderness RatioY = (L_o x Effective Length FactorY) / Width

= (2750 x 1.000) / 230 = 11.96 <= 12.00,

column is not slender in this direction.

$M_{x\_MinEccen}$ = 9.854 kN.m

$M_{y\_MinEccen}$ = 7.729 kN.m

$M_x = \max(M_x, M_{x\_MinEccen}) + M_{uaddX}$ = 9.854 kN.m

$M_y = \max(M_y, M_{y\_MinEccen}) + M_{uaddY}$ = 78.785 kN.m

Calculation Of Eccentricities :

As per IS 456: 2000 Clause 25.4,all columns shall be designed for minimum eccentricity equal to the unsupported length of the column/500 plus lateral dimension/30, subject to a minimum of 20 mm.

Actual $eccen_X$ = Mx / P = 3.36/386.43 = 9 mm

Actual $eccen_Y$ = M_y / P = 78.78/386.43 = 204 mm

$eccen_{XMin}$ = (L/500) + (D/30) = (2750/500) + (600/30) = 26 mm

$eccen_{XMin}$ = max(26,20)= 26 mm

$eccen_{YMin}$ = (L/500) + (B/30) = (2750/500) + (230/30) = 13 mm

$eccen_{YMin}$ = max(13,20) = 26 mm

$eccen_X$ = max(Actual $eccen_X$,$eccen_{XMin}$) = max(9,26) = 26 mm

$eccen_Y$ = max(Actual $eccen_Y$,$eccen_{YMin}$) = max(204,20) = 204 mm

As per IS 456: 2000 Clause 39.3,
when the minimum eccentricity does not exceed 0.05 times the lateral dimension,
the column will be designed as an axially loaded column.
eccenX(26 mm) > 0.05 x 230(12 mm)
and eccenY(204 mm) > 0.05 x 600(30 mm)
Hence the column will be designed as a column with biaxial moments.

Provided steel :
Provide #12 - 12 nos. + #16 - 6 nos. (2564 mm²)

Biaxial Check Calculations :

For X axis :

Row No. (i)	A_{si} (mm²)	e_i	f_{si} (N/mm²)	f_{ci} (N/mm²)	(f_{si} - f_{ci}) (N/mm²)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	402	0.00290	353.00	8.92	344.08	138.36	252	34.87
2	226	0.00209	331.21	8.92	322.29	72.90	188	13.71
3	226	0.00132	263.29	7.88	255.42	57.77	126	7.28

S. B. Bhattacharyya
B.E. (Civil)
ESE- 116 /1

4	226	0.00054	107.69	4.16	103.53	23.42	64	1.50
5	402	-0.00026	-52.94	0.00	-52.94	-21.29	0	0.00
6	226	-0.00107	-213.57	0.00	-213.57	-48.31	-64	3.09
7	226	-0.00185	-320.17	0.00	-320.17	-72.42	-126	9.13
8	226	-0.00262	-348.30	0.00	-348.30	-78.78	-188	14.81
9	402	-0.00343	-357.64	0.00	-357.64	-143.81	-252	36.24
					Total	-72.16		120.62

x_{ux} = 279 mm $\qquad$ $P_{uc} = C_1.f_{ck}.B.D$

C_1 = 0.36 k_u = 0.36 x 279/600 = 0.167

P_{uc} = 0.167 x 20.00 x 230 x 600

= 461.87 kN

$P_{ux1} = P_{uc} + P_{us(Total)}$

= 461.87 + (-72.16)

= 389.71 kN > P ,hence O.K.

$M_{uc} = P_{uc}.(0.5.D - C_2.D)$

C_2 = 0.416 k_u = 0.416 x 279/600

= 0.193

M_{uc} = 461.87 x (0.5 x 600 - 0.193 x 600) = 84.97 kN-m

$M_{ux1} = M_{uc} + M_{us(Total)}$

= 84.97 + (120.62)

= 205.59 kN-m

For Y axis :

Row No. (i)	A_{si} (mm^2)	e_i	f_{si} (N/mm^2)	f_{ci} (N/mm^2)	(f_{si} - f_{ci}) (N/mm^2)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	1282	0.00185	320.13	8.87	311.27	398.97	67	26.73
2	1282	-0.00277	-351.93	0.00	-351.93	-451.09	-67	30.22
					Total	-52.12		56.95

x_{uy} = 102 mm $\qquad$ $P_{uc} = C_1.f_{ck}.B.D$

C_1 = 0.36 k_u = 0.36 x 102/230 = 0.159

P_{uc} = 0.159 x 20.00 x 600 x 230

= 438.58 kN

$P_{uy1} = P_{uc} + P_{us(Total)}$

= 438.58 + (-52.12)

= 386.46 kN > P ,hence O.K.

$M_{uc} = P_{uc}.(0.5.B - C_2.B)$

C_2 = 0.416 k_u = 0.416 x 102/230

= 0.184

M_{uc} = 438.58 x (0.5 x 230 - 0.184 x 230) = 31.91 kN-m

$M_{uy1} = M_{uc} + M_{us(Total)}$

= 31.91 + (56.95)

= 88.87 kN-m

P_u/P_{uz} = 0.192

For P_u/P_{uz} <= 0.2, α_n = 1.0

For P_u/P_{uz} >= 0.8, α_n = 2.0

For P_u/P_{uz} = 0.192, α_n = 1.000

$((M_{ux} / M_{ux1})^{\alpha_n}) + ((M_{uy}/M_{uy1})^{\alpha_n})$

$= ((9.85/205.59)^{1.000}) + ((78.78/88.87)^{1.000})$

= 0.934 < 1.0,hence O.K.

Ties Details :

Load combination for ties design = 1.50 DL + 1.50 EQL X+

Calculation of diameter of ties :

As per IS 456: 2000 Clause 26.5.3.2 (c), the diameter of the polygonal links or lateral ties shall be not less than the following

i) one-fourth the diameter of the largest longitudinal bar = 16/4 = 4 mm

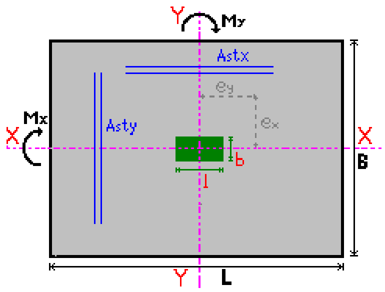
ii) in no case less than = 6 mm

S. B. Bhattacharyya
B.E. (Civil)
ESE- 116 /1

Foundation Design Detail Report :

(Design by Limit State Method as per IS 456 : 2000)

Standard Conventions:



Individual Footing

Design of FG1

Size of col./ ped, (b x l) = 0.230 x 0.525 m
SBC of soil = 80.00 kN/m²
SBC incr.factor = 1.250
SBC of soil = 100.00 kN/m²
f_{ck} = 20.00 N/mm²
f_y = 415.00 N/mm²
Load combination = 1.50DL + 1.50EQL Y-

Loads On Footing

Factored axial load on column,(P) = 1205.72 kN
Working axial load on column,(P_w) = 803.81 kN (From analysis results)
Self Weight of footing assumed (S_w) = 10.00 % of P_w = 80.38 kN
Total Working load,(P_1) = (P_w)+ (S_w) = 884.20 kN
Total Ultimate load,(P_{u1}) = (P)+ 1.5 x (S_w) = 1326.29 kN
BM $_{@\ x\text{-}x}$,(WM_x) = 41.89 kN-m
BM $_{@\ y\text{-}y}$,(WM_y) = 5.16 kN-m
Factored BM $_{@\ x\text{-}x}$,(M_x) = 62.84 kN-m
Factored BM $_{@\ y\text{-}y}$,(M_y) = 7.74 kN-m

In Loadcombination with DL load case :
If load factor for DL loadcase is less than 1.0,this factor will be retained while computing the working load.

Footing Area

Area for axial load,(A_f) = P_1/SBC = 884.20 / 100.00 = 8.842 m²
Eccentricity $_{@\ x\text{-}x}$,(e_x) = WM_x/P_w = 0.0521 m
= 0.052 m
Eccentricity $_{@\ y\text{-}y}$,(e_y) = WM_y/P_w = 0.0064 m
= 0.006 m
Footing size$_Y$,(B) = (0.5 x (b-l))
+ (√((((0.5 x (b-l)) x (0.5 x (b-l)))+A_f)))
= 3.125 m
Footing size$_X$,(L) = A_f/B = 2.830 m

Modified area for M_x,(A_{fx}) = (P_1/SBC) x (1 + 6.e_x/B)
= (884.20/100.00) x (1 + 6 x 0.0521/3.125)
= 9.727 m²
Modified area for M_y,(A_{fy}) = (P_1/SBC) x (1 + 6.e_y/L)
= (884.20/100.00) x (1 + 6 x 0.0064/2.830)
= 8.962 m²
Size,(L x B) = 3.075 x 3.375 m
Area provided ,(A_p) = 10.378 m² > A_f.

SBC Check:

Upward pressure due to axial load,(P_{r1w}) = P1 / (L x B) = 85.20 kN/m²
Upward pressure due to M_x,(P_{r2w}) = WM_x / (L x B x B/6) = 7.1763 kN/m²
Upward pressure due to M_y,(P_{r3w}) = WM_y / (B x L x L/6) = 0.9698 kN/m²
Maximum upward pressure,(Up_{Maxw}) = P_{r1w} + P_{r2w} + P_{r3w} = 93.34 kN/m²
Minimum upward pressure,(Up_{Minw}) = P_{r1w} - P_{r2w} - P_{r3w} = 77.05 kN/m²
Up_{Maxw}(93.43 kN/m²) < SBC (100.00 kN/m²),Hence ,safe.

Pressures for Different Load Combinations :

Load Combination	Design Pressures(kN/m²)		Working Pressures(kN/m²)	
	Maximum	Minimum	Maximum	Minimum
1.50 DL + 1.50 LL	89.03	86.75	59.36	57.83
1.20 DL + 1.20 LL + 1.20 EQL X+	78.15	62.12	65.13	51.77
1.20 DL + 1.20 LL + 1.20 EQL X-	77.49	63.57	64.58	52.97
1.20 DL + 1.20 LL + 1.20 EQL Y+	40.05	23.11	33.37	19.26
1.20 DL + 1.20 LL + 1.20 EQL Y-	118.98	99.43	99.15	82.86
1.50 DL + 1.50 EQL X+	89.06	69.10	59.37	46.06
1.50 DL + 1.50 EQL X-	88.32	70.84	58.88	47.22
1.50 DL + 1.50 EQL Y+	41.44	20.34	27.63	13.56
1.50 DL + 1.50 EQL Y-	140.14	115.70	93.43	77.13
0.90 DL + 1.50 EQL X+	57.11	37.65	54.05	40.82
0.90 DL + 1.50 EQL X-	56.87	38.89	53.63	41.90
0.90 DL + 1.50 EQL Y-	107.97	84.44	88.06	71.92
0.90 DL + 1.50 EQL Y+	-	-	-	-

Tension Force On Footing for Different Load Combinations :

Load Combination	Working Load			Factored load		
	F_z	M_x	M_y	F_{uz}	M_{ux}	M_{uy}
0.90 DL + 1.50 EQL Y+	143.81	39.98	0.76	-8.72	60.42	0.26

Maximum tension force on footing T = -8.72
Total depth of fotting below G.L. D = 1.500 m
Unit weight of soil γ_{soil} = 18.00 kN/m^3
Minimum thickness of footing t_f = 0.150 m
Effective height of soil H_s = D - t_f
= 1.500 - 0.150
= 1.350 m
Volume of column below G.L. V_{col} = H_s x b_{col} x d_{col}
= 1.350 x 0.230 x 0.525
V_{col} = 0.163 m^3
Total volume below G.L. V_{total} = H_s x F_x x F_y
= 1.350 x 3.075 x 3.375
V_{total} = 14.010 m^3
Total weight of soil W_{soil} = (V_{total} - V_{col}) x γ_{soil}
= (14.010 - 0.163) x 18.000
W_{soil} = 249.254 kN
W_{total} > T Hence ok

Bending Moment Calculations :

As per IS 456: 2000 Clause 34.2.3.2,critical section for checking bending moment in the design of an isolated concrete footing which supports a column shall be a section located at the face of the column or pedestal.

Upward pressure due to axial load,(P_{r1}) = P_{u1}/ (L x B) = 127.80 kN/m²

Upward pressure due to M_x,(P_{r2}) = M_x / (L x B x B/6) = 10.7644 kN/m²

Upward pressure due to M_y,(P_{r3}) = M_y / (B x L x L/6) = 1.4547 kN/m²

Maximum upward pressure,(Up_{Max}) = P_{r1} + P_{r2} + P_{r3} = 140.02 kN/m²

Minimum upward pressure,(Up_{Min}) = P_{r1} - P_{r2} - P_{r3} = 115.58 kN/m²

Projection-X,(X_{proj}) = (L - l)/2
= (3.075 - 0.230)/2
= 1.423 m

Critical upward pressure,(Up_{Cr}) = Up_{Min} + (L - X_{proj}) x ((Up_{Max} - Up_{Min})/L)
= 115.70 + (3.075 - 1.423) x ((140.14 - 115.70)/3.075)
= 128.83 kN/m²

Upward pressure area,(Up_{PrArea}) = X_{proj} x (Up_{Cr} + Up_{Max})/2
= 1.423 x (128.83+ 140.14)/2
= 191.31 kN/m

CG distance from column face,(CG_{Dist}) = X_{proj} x (Up_{Cr} + (2 x Up_{Max}))
/(3 x (Up_{Cr} + Up_{Max}))
= 1.423 x (128.83 + 2 x 140.14)
/ (3 x (128.83 + 140.14))
= 0.721 m

Design BMx,(M_{ux}) = Up_{PrArea} x CG_{Dist} x B
= 191.31 x 0.721 x 3.375
= 461.81 kN-m

Projection-Y,(Y_{proj}) = (B - b)/2
= (3.375 - 0.525)/2
= 1.425 m

Critical upward pressure,(Up_{Cr}) = Up_{Min} + (B - Y_{proj}) x ((Up_{Max} - Up_{Min})/B)
= 115.70 + (3.375 - 1.425) x ((140.14 - 115.70)/3.375)
= 129.82 kN/m²

Upward pressure area,(Up_{PrArea}) = Y_{proj} x (Up_{Cr} + Up_{Max})/2
= 1.425 x (129.82+ 140.14)/2
= 192.35 kN/m

CG distance from column face,(CG_{Dist}) = Y_{proj} x (Up_{Cr} + (2 x Up_{Max}))
/(3 x (Up_{Cr} + Up_{Max}))
= 1.425 x (129.82 + 2 x 140.14) / (3 x (129.82 + 140.14))
= 0.722 m

Design BMy,(M_{uy}) = Up_{PrArea} x CG_{Dist} x L
= 192.35 x 0.722 x 3.075
= 402.26 kN-m

Depth Calculations :

Depth required for punching shear :

As per IS 456:2000 Clause 31.6.3.1,the calculated shear stress at the critical section shall not exceed k_s x τ_{auc}

where,

k_s = 0.5 + β_{ac}, but not greater than 1,β_{ac} being the ratio of short side to the long side of column, and

τ_{auc} = 0.25 x $\sqrt{(f_{ck})}$in limit state method of design.

Depth required for punching shear is calculated by equating the actual stress to permissible stress and by solving the resulting quadratic equation.

Total depth assumed,(D) = 0.750 m

Bottom cover = 0.050 m

Effective Depth Calculation :

Effective depth,(d_{effx}) = 0.695 m

Effective depth,(d_{effy}) = 0.685 m

Effective depth,(d_{eff}) = 0.695 m

Depth || to X axis at critical section,(d_{punch_x}) = 0.695 m

Depth || to Y axis at critical section,(d_{punch_y}) = 0.695 m

Resisting area || to X axis,(A_{rx}) = d_{punch_x} x 2 x (l + d_{eff})
= 0.695 x 2 x (0.230 + 0.695)
= 1.286 m²

Resisting area || to Y axis,(A_{ry}) = d_{punch_y} x 2 x (b + d_{eff})
= 0.695 x 2 x (0.525 + 0.695)
= 1.696 m²

Resisting area,(A_r) = A_{rx} + A_{ry}
= 2.982 m²

Shear force,(V) = (P / A_p) x (A_p -((l + d_{eff}) x (b + d_{eff})))
= (1205.72 / 10.378) x (10.378 - (0.925 x 1.220))
= 1074.61 kN

Actual shear stress = V / A_r
= 1074.61 / 2.982
= 360.42 kN/m²

Permissible shear stress = 0.25 x √(f_{ck}) x ks

ks = 0.5 + betaC

betaC = short column side/ long column side
= 0.230 / 0.525 = 0.438

ks = 0.5 + 0.438 = 0.938 = 0.938

Permissible shear stress = 0.25 x 4.4721 x 0.938
= 1.049 N/mm²
= 1048.82 kN/m²

Actual shear stress < Permissible shear stress.
Hence,Safe in punching shear.

Calculation Of Moment Of Resistance :

Total depth,(D) = 0.750 m

Minimum depth,(D_{min}) = 0.150 m

For (Fe415),k = 0.479

R = 0.36 x f_{ck} x k x (1 - 0.42 x k)
= 0.36 x 20.000 x 0.479 x (1 - 0.42 x 0.479)
= 2.756

Moment of resistanceX = R x width at NA x d_{effx} x d_{effx}
= 2.756 x 2.034 x 0.695 x 0.695
= 2184.29 kN-m > 461.81 kN-m

Moment of resistance Y = R x width at NA x d_{effy} x d_{effy}
= 2.756 x 1.716 x 0.685 x 0.685
= 1653.49 kN-m > 402.26 kN-m

Steel Calculations :

For M_{ux} :

Cross sectional area,(C/S_{area}) = width at NA x d_{effx}
= 2.034 x 0.695
= 1.414 m²

val = 1 - [(4.6 x M_{ux})/(f_{ck} x B x d_{effx}^2)]

Required steel area,(A_{stx}) = [0.5(f_{ck}/f_y) x (1 - √(val))] x C/S_{area}
= 0.001894 m²

= 1894 mm²

Required steel pt. = 0.13398 %

Provide #10 - 27 nos.(2121 mm²)

Spacing = (L - dia. - (2 x end cover))/(no - 1)
= (3.375 - 0.010 - (2 x 0.050))/(27 - 1)
= 0.126 m < maximum spacing(0.200 m)

For M_{uy} :

Cross sectional area,(C/S_{area}) = width at NA x d_{effy}
= 1.716 x 0.685
= 1.176 m²

val = 1 - [(4.6 x M_{uy})/(f_{ck} x B x d_{effy}^2)]

Required steel area,(A_{sty}) = [0.5 (f_{ck}/f_y) x (1 - √(val))] x C/S_{area}
= 0.001677 m²
= 1677 mm²

Required steel pt. = 0.14265 %

Provide #10 - 23 nos.(1806 mm²)

Spacing = (B - dia. - (2 x end cover))/(no - 1)
= (3.075 - 0.010 - (2 x 0.050))/(23 - 1)
= 0.135 m < maximum spacing(0.200 m)

Check For One Way Shear :

As per IS 456: 2000 Clause 33.2.4.1,the critical section for this condition shall be assumed as a vertical section located at a distance equal to the effective depth of the footing from the face of the column or pedestal.

For X-axis :

Projection-X,(proj) = (L - l)/2 - d_{effx}
= (3.075 - 0.230)/2 - 0.695
= 0.728 m

Critical upward pressure,(Up_{Cr}) = Up_{Min} + (L - proj) x ((Up_{Max} - Up_{Min})/L)
= 115.70 + (3.075 - 0.728) x ((140.14 - 115.70)/3.075)
= 134.36 kN/m²

Shear force,(V_x) = (proj x B) x (Up_{Cr}+Up_{Max})/2
= (0.728 x 3.375) x 137.25
= 316.51 kN

Actual depth at critical section,(D_{cr}) = 0.468 m

Actual width at critical section,(Wcr) = 1.915 m

Trapezoidal area,(A_{trap}) = (B + Wcr) x (D_{cr} - D_{min})/2
= (3.375 + 1.915) x (0.468 - 0.150)/2
= 0.841m²

Rect. area,(A_{rect}) = B x (D_{min} - eff.Cover)
= 3.375 x 0.095
= 0.321m²

Resisting area,(A_r) = A_{trap} + A_{rect}
= 0.841 + 0.321
= 1.162 m²

Percentage of steel at critical section = (0.002121 x 100)/1.162
= 0.18252

Permissible shear stress = 313.48 kN/m²

Actual shear stress = V_x / A_r= 316.51 / 1.162
= 272.42 kN/m²
< permissible shear stress(313.48 kN/m²)

For Y-axis :

Projection-Y,(proj) = (B - b)/2 - d_{effy}

S. B. Bhattacharyya
B.E. (Civil)
ESE- 116 /1

= (3.375 - 0.525)/2 - 0.685
= 0.740 m

Critical upward pressure,(Up_{Cr}) = Up_{Min} + (L - proj) x ((Up_{Max} - Up_{Min})/L)
= 115.70 + (3.375 - 0.740) x ((140.14 - 115.70)/3.375)
= 134.78 kN/m²

Shear force,(V_y) = (proj x L) x (Up_{Cr}+Up_{Max})/2
= (0.740 x 3.075) x 137.46
= 309.93 kN

Actual depth at critical section,(D_{cr}) = 0.473 m

Actual width at critical section,(Wcr) = 1.600 m

Trapezoidal area,(A_{trap}) = (L + Wcr) x (D_{cr} - D_{min})/2
= (3.075 + 1.600) x (0.473 - 0.150)/2
= 0.755m²

Rect. area,(A_{rect}) = L x (D_{min} - eff.Cover)
= 3.075 x 0.095
= 0.292m²

Resisting area,(A_r) = A_{trap} + A_{rect}
= 0.755 + 0.292
= 1.047 m²

Percentage of steel at critical section = (0.001806 x 100)/1.047
= 0.17254

Permissible shear stress = 309.87 kN/m²

Actual shear stress = V_y / A_r= 309.93 / 1.047
= 304.99 kN/m²
< permissible shear stress(309.87 kN/m²)

Check For Bearing :

As per IS 456: 2000 Clause 34.4, the bearing pressure on the loaded area shall not exceed the permissible bearing stress in direct compression multiplied by a value equal to √(A1/A2), but not greater than 2.

Supporting area for bearing of footing,(A1) = (l + (4 x D)) x (b + (4 x D))
= (0.230 + (4 x 0.750)) x
(0.525 + (4 x 0.750))
= 11.386 m² > footing area(10.378 m²)
= 10.378 m²

Loaded area at the column base,(A2) = 0.121 m²

√(A1/A2) = √(10.378/0.121) = 9.271 > 2
= 2.000

Actual bearing stress = P/A2
= 1205.72/0.121
= 9985.27 kN/m²

Permissible bearing stress = 0.45 x f_{ck} x √(A1/A2)
= 0.25 x 20.00 x 2.000
= 18.000 N/mm²
= 18000.00 kN/m²

Actual bearing stress < Permissible bearing strees.
Hence,Safe in bearing.

Check For Sliding Along X axis :

Load combination = 0.90DL + 1.50EQL X+

Self wt. of footing = 99.16 kN

Stabilizing force,(W) = 545.92 kN

Horizontal force,(H) = 49.41 kN

Soil resistance,(S) = 242.81 kN> H
Hence,check for sliding is not required.

Check For Sliding Along Y axis :

Load combination = 1.50DL + 1.50EQL Y+

Self wt. of footing = 99.16 kN

Stabilizing force,(W) = 389.68 kN

Horizontal force,(H) = 40.29 kN

Soil resistance,(S) = 206.11 kN> H

Hence,check for sliding is not required.

Bond Check Calculations For Footing Bars

As Per IS 456 : 2000, Clause 26.2.1,The Required Development Length, Ld is given as

L_d = $(\phi \times \sigma) / (4 \times \tau_{bd})$

where,

ϕ = Nominal diameter of the footing bar in X or Y direction,

σ = Stress in the bar at the section considered, at design load and

τ_{bd} = Design bond stress.

Hence, L_{dx} = (10 x 0.87 x 415.00) / (4 x 1.92)

= 470.117 mm

L_{dy} = (10 x 0.87 x 415.00) / (4 x 1.92)

= 470.117 mm

Available Development Length = $(F_d - C_d)$ / 2 - End Cover + End Anchorages(If required)

Where,

F_d = Footing size in X / Y direction

C_d = Column size in X / Y direction

End Anchorage = 4 x ϕ, for a L hook

= 8 x ϕ, for a 90 degrees bend

In X direction = (3.075 - 0.230) x 0.5 - 0.050

= 1372.500 mm

In Y direction = (3.375 - 0.525) x 0.5 - 0.050

= 1375.000 mm

Hence, the Bond check for footing bars is satisfied

Bond Check Calculations For Column Bars in Tension

Ld = $(\phi \times \sigma) / (4 \times \tau_{bd})$

where, other terms remaining same,

ϕ = Nominal diameter of column bars

τ_{bd} = Design bond stress in ***Tension*** for column bars

Diameter of Column bar d_1 = 12 mm

Required Development Length for 'd_1' = (12 x 0.87 x 415.00) / (4 x 1.92)

= 564 mm

Diameter of Column bar d_2 = 16 mm

Required Development Length for 'd_2' = (16 x 0.87 x 415.00) / (4 x 1.92)

= 752 mm

Available development length = $Min(d_{effx}/d_{effy}) + H_{proj}$

Where,

H_{proj} = Horizontal projection of columns bars in footing

= Min(695.000 / 685.000) + 300.000

= 985.000 mm

Hence, the Bond check for column bars in tension is satisfied

Bond CheckCalculations For Column Bars in Compression

Required Development Length For Column Bars

Ld $(\phi \times \sigma) / (4 \times \tau_{bd})$

where, other terms remaining same,

ϕ = Dia of column bars

S. B. Bhattacharyya
B.E. (Civil)
ESE- 116 /1

DESIGN OF STAIR PREMISES NO- 15A, RAIPUR MONDAL PARA ROAD

It is proposed to provide two flights for the stairway
Hence the height of each flight 3.05/2= 1.525m
Assuming 150mm Risers
Number of risers required= 1525/150= 10 nos
hence the actual size of each riser= 1525/10= 152.5mm
number of treads in each flight=10-1= 9 nos
width of each tread= 250mm
total going= 250x9= 2250mm= 2.250m
total length available= 4775m
width of each landing= (4.775-2.25)/2= 1.26m= 1260mm

Design of Flight
let the bearing for the flight be 250 mm
effective horizontal Span = 4.775+.25/2+.25/2 = 5.025m
let the thickness of the waist slab be 120mm

Loads
D.L of 120 mm waist slab= 25x120= 3000 N/M /M^2

corresponding load per sq. metre on plan= $((\sqrt{R^2+T^2})/T)X3000$

$((\sqrt{152.5^2+250^2}/250)X3000= 3514N/M^2$

hence actual load per sq. metre of plan area consist of the following waist slab

$= 3514N/M^2$

Dead load of steps(152.5/2 mm average)= 76.25x25= 1906 N/M^2
top finish(12.5mm)= 12.5x24= 300 N/M^2
Live load= 3500 N/M^2
Total= 9220 N/M^2

Maximum bending moment per unit width of stairs= $(9220x5.025^2)/8$ = 29101N-M
Adopting, C= 5 N/M^2; t= 230 N/M^2; m= 18.66
equating the moment of resistance to the bending moment, we have

$.87x\ 1000\ d^2= 29101x1000$

d= 182mm

provide10 mm bars at clear cover of 15mm

effective cover= 15+5= 20mm

effective depth= 200-20= 180 mm

Ast= (29101x1000)/(230x.87x180) = 799 mm^2

spacing of 10 mm dia bars= (79x1000)/718 = 100 mm
provide 10 mm dia @ 100 mm C/C

Distribution steel = (.12x200x1000)/100 = 240mm^2
provide 8 mm dia @ 150 mm C/C

S. B. Bhattacharyya
B.E. (Civil)
ESE- 116 /1