Element Design

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serviceability requirement Slab design

Limited L/d ratio for serviceability Requirement

L/d<Basic $_{l/d}$* S* T*C*K$_{W/F}$

Basic Value of Span to Depth Ratio B$_{l/d}$

Slab type	Description	Z value
1	Cantilever	7
2	Simply Supported	20
3	One side continue	23
4	Two side continue	26

S= Span correction factor, For >10 m slab, = 10/Span,
T – mod. factor for tension reinforcement
C – MF for compression reinforcement
Kw/F – MF depending upon ratio of web/flange width

Preliminary Size of Slab

- For Structural slab, minimum thickness for ease in construction= 100 mm
- Slab thickness is mainly governed by serviceability requirement.
- Total depth required = [span/Allowable L/d] + effective cover

  Where allowable L/d ratio = basic L/d ratio x MF modification factor MF =

- Mod. Factor for tension = 1.4 for preliminary design with Fe415
- For simply supported, L/d = 20*1.4= 28, for continuous, L/d = 26*1.4 = 36.

Preliminary Size of Slab

- Following basic L/d may be taken using mild steel reinforcement:
  - simply supported 2 way = 35
  - continuous two way slab = 40
- Following basic L/d may be taken using HYSD bars of grade Fe415:
  - simply supported 2 way (35*0.8) = 28
  - continous two way slab (40*0.8) = 32

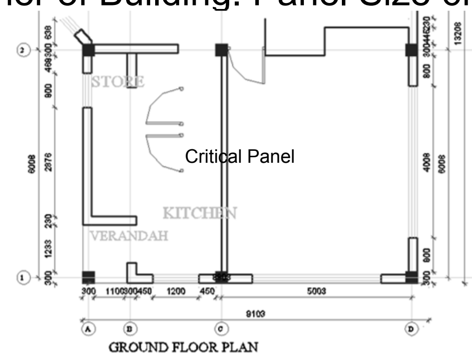
- Required thickness of slab = 4000/32+25 = 150

Preliminary Size of Slab

- Other detailing requirements:
  - The dia of main bar not greater than 1/8 of total thickness of slab
  - the amount of steel in any direction should not be less than 0.12 % of total cross sectional area when using Fe 415.
  - spacing of main steel not greater than 3d or 300 mm; secondary steel not greather than 5d or 450 mm.

Illustrative Example

- Design a RCC slab critical panel situated in corner of Building. Panel Size 6mX5m



- Required depth in general condition for shorter span 5 m= 5000/(40*.8)+20= 176 mm
- Limited modification factor ≤ 2.
- Limited % of tension reinforcement

	Fe 415	Fe 500
M15	0.72	0.60
M20	0.96	0.79
M25	1.20	0.94

- Hence minimum depth required for the span 5 m = 5000/(2*23)+20 = 128 mm
- Adopt thickness = 140 mm; d' = 140 – 20 =120
- Hence adopted l/d ratio = 5000/120 = 41.66
- Whereas standard l/d ratio = 23
- Hence required Modification factor = 41.66/ 23 = 1.81

Load Calculation

material	Thickness	Density	Load kn/m^2
Marble	0.016	26.7	0.4272
Screed	0.025	22	0.55
Slab	0.14	25	3.5
Plaster	0.012	20	0.24
Partition			1
Live load			3
Total			8.7172
Design L=8.72*1.5=13			13.0758

Moment per unit width
Mx=αx.w.lx^2
My=αy.w.lx^2

Table 26 Bending Moment Coefficients for Rectangular Panels Supported on Four Sides with Provision for Torsion at Corners

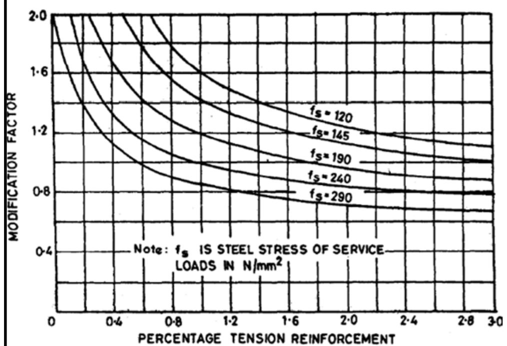
(*Clauses* D-1.1 *and* 24.4.1)

Case No.	Type of Panel and Moments Considered	Short Span Coefficients α_x (Values of l_y/l_x) 1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	Long Span Coefficients α_y for All Values of l_y/l_x
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	*Interior Panels:*									
	Negative moment at continuous edge	0.032	0.037	0.043	0.047	0.051	0.053	0.060	0.065	0.032
	Positive moment at mid-span	0.024	0.028	0.032	0.036	0.039	0.041	0.045	0.049	0.024
2	*One Short Edge Continuous:*									
	Negative moment at continuous edge	0.037	0.043	0.048	0.051	0.055	0.057	0.064	0.068	0.037
	Positive moment at mid-span	0.028	0.032	0.036	0.039	0.041	0.044	0.048	0.052	0.028
3	*One Long Edge Discontinuous:*									
	Negative moment at continuous edge	0.037	0.044	0.052	0.057	0.063	0.067	0.077	0.085	0.037
	Positive moment at mid-span	0.028	0.033	0.039	0.044	0.047	0.051	0.059	0.065	0.028
4	*Two Adjacent Edges Discontinuous:*									
	Negative moment at continuous edge	0.047	0.053	0.060	0.065	0.071	0.075	0.084	0.091	0.047
	Positive moment at mid-span	0.035	0.040	0.045	0.049	0.053	0.056	0.063	0.069	0.035

ly/lx = 1.2;
ax(-) = 0.060; ax(+) = 0.045;
ay(-) = 0.047; ay(+)=0.035

Mx(+) = 0.045*13*5^2 = 14.71 knm
Ast req = 300 mm^2
Pt = .25%

Control of Deflection



$$f_s = 0.58 f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

FIG. 4 MODIFICATION FACTOR FOR TENSION REINFORCEMENT

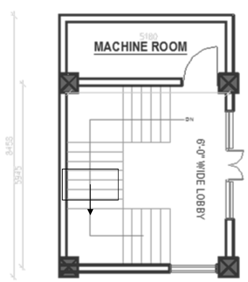
Ast	Pt	fs	MF
300	.25	290	1.35
450	.37	193	1.7
500	.42	174	1.78
550	.46	158	1.82

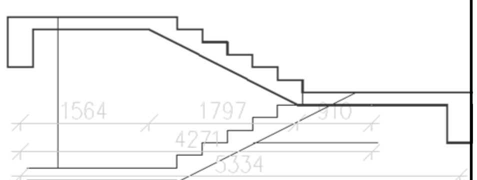
- Hence modification factor sufficient only when Ast = 550 mm^2/m

Rebars for other B.M.

Position	a	B.M.	Ast req	bar dia	spacing
Short Span (-) BM	.060	-19.62 knm	411	8	120
Long Span (-) BM	.047	15.36 knm	315	8	158
Long Span (+) BM	0.035	11.44	230	8	217

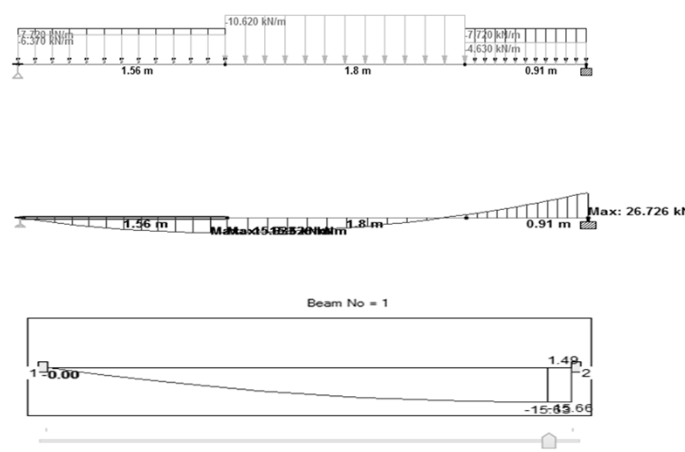
Design of Stairecase





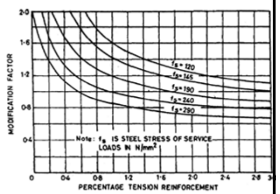
Stair load	Thickness	Load Plain	Inclined
Slab	0.14	3.5	3.91
Riser	0.15		
Tread	0.3		
Slope	1.118034		
Steps			1.88
Screed	0.025	0.55	0.83
Marble	0.016	0.432	0.65
Plaster	0.012	0.24	0.36
Total dead		4.72	7.62
Live		3	3.00

Reaction from landing Dead load =7*.3*7.62/2*1.75 /(1.75*1.75) = 4.57 kn/m
Live = 7*.3*3/2*1.75/(1.75*1.75)=1.8
Floor landing dead = 2.1*4.72/(2*1.75) = 2.832 kn/m
Floor landing live = 1.8 kn/m



Calculation for Serviceability limits

- Thickness of west slab = 140 mm
- Effective thickness = 120 mm
- L/d provided = 4272/120 = 35.6
- Standard L/d = 23
- Modification Factor = 1.54
- Design BM = 15.66*1.5 = 23.49 knm/m
- Required Ast = 502 mm^/m
- Provided Ast = (12#@150c/c) = 753 mm^2/m Pt= .627
- Fs = 192; MF = 1.4
- Provide 12@120c/c = 941 Pt = .79 i.e. limit .
- Fs = 154; MF = 1.55
- Hence thickness is safe



$$f_s = 0.58\, f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

FIG. 4 MODIFICATION FACTOR FOR TENSION REINFORCEMENT

- For negative bending moment, i.e. DBM = 26.76*1.5 = 40.14 knm, Ast = 1092 i.e. 0.82%, Which is > .79%
- Design it for Doubly Reinforcement Section
- Steel for compression 12 mm^2
- Steel for tension 962 mm^2

- Reaction to frame system

Stair.std - Support Reactions:

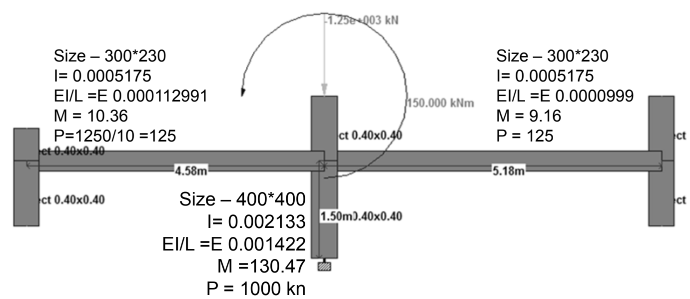
All | Summary | Envelope

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
1	1 LOAD CAS	0.000	17.596	0.000	0.000	0.000	0.000
	2 Live	0.000	3.411	0.000	0.000	0.000	0.000
	11 COMBINA	0.000	21.007	0.000	0.000	0.000	0.000
4	1 LOAD CAS	0.000	24.893	0.000	0.000	0.000	-20.971
	2 Live	0.000	6.435	0.000	0.000	0.000	-5.755
	11 COMBINA	0.000	31.328	0.000	0.000	0.000	-26.726

Design of Plinth Level Tie Beam

- Tie beam is designed for moment coming from column and corresponding 10% of column axial load taken in tension or compression
- For longitudinal steel, designed as tension with bending and for transverse steel, designed as beam by capacity design approach

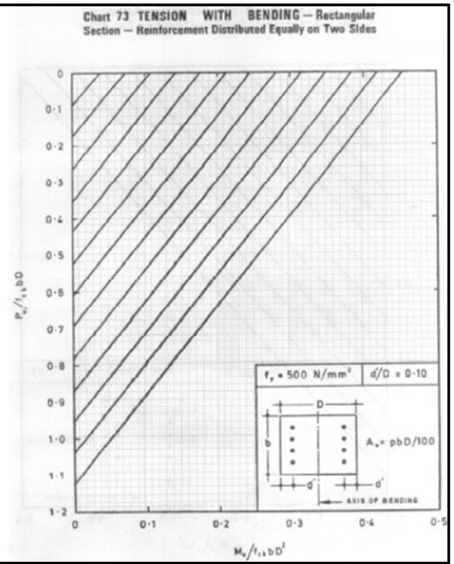
Design Force



Tie beam design	D	w	L	I	I/L	Ratio	ParticiMom
LB	0.3	0.23	4.58	0.000518	0.000112991	0.069103	10.3654297
RB	0.3	0.23	5.18	0.000518	0.00009990	0.061099	9.16480079
C	0.4	0.4	1.5	0.002133	0.001422222	0.869798	130.469769
Moment	150				0.001635117		

Design member as tensile member

- Size – 300*230
- I= 0.0005175
- EI/L =E 0.000112991
- M = 10.36 knm
- P=1250/10 =125 kn
- Design the section as Tension with Bending using SP16 –chart 72 to 75
- d'/D = .084~ .1 for beam
- Mu/(fckbD^2) =0.0375
- Pu/(fckbD)= 0.136
- Hence Pt = 0.035



- Pt = .035*20 = 0.7%
- Ast = 483
- Hence provide 3-12# bars in top and bottom. (Area= 678)

Design the beam for stirrups

- Moment Resistance of beam size with 3-12# top and bottom
  - MR = 36.67 knm
  - Vb =1.4 $\frac{(MuA+MuB)}{l}$ = 22.41 kn

  8#@450 mm c/c

  Hence provide in spacing d/4 = 125 c/c. Min 8#@100 c/c

Preliminary Size of Beam

- breadth not greater than width of column for effective load transfer.
- breadth of beam = 1/3 to 1/2 of the depth of beam.
- Depth of beam = L/10 to L/16.
- For heavy loads and/or large spans, provide D = L/10.
- for light load and/or small spans, provide D = L/16.
- Since D>L/20, deflection is normally satisfied.
- The rare possibility of design shear exceeding the permissible maximum shear is avoided by increasing the section of the beam.

Preliminary Size of Beam

- The usual widths of beam adopted in mm are 150, 200, 230, 250, 300 mm. These widths should be equal to or less than the dimension of the columns into which they frame. For example, 300 mm wide beams can frame into 300 mm or 400 mm dimensions of columns.
- In a simply supported or continuous beam, the clear distance between the lateral restraint should not exceed 60b or 250b^2/d whichever is less.
- For a cantilever, the distance between the free end of the cantilever and the lateral restraint should not exceed 25b or 100b^2/d whichever is less.

Other detailing Requirement of Beam

- Usually same size of beam is used for different spans to use same size of formwork from practical considerations
- Keep the beam width adequate to accommodate all bars with sufficient gaps and covers between rebars.
- at least two bars to be used as tension steel.
- dia of hanger bar not less than 6 mm. Dia of main tension bar not less than 12 mm. Dia used are 10, 12, 16, 20, 22, 25 and 32 mm.
- when using different sized bars in one layer, place the largest size bar near the beam faces. The areas of steel should be symmetrical about the centre line of the beam.
- The width of beam necessary to accommodate the required number of bars will depend upon the specification of the cover and minimum spacing between bars. The max. size of aggregate used is normally 20 mm and hence clear minimum distance between bars should be 25 mm.

Preliminary Column Sizing

- Approximate size may be calculated on the basis of axial load calculated from tributary area and then applying multiplication factor for possible bending moments.
- Dimension should be sufficient to accommodate beam within column rebars.

Column Size Approximation

Approximate Column Sizes

- equivalent Direct Load

  = k*Load based on static reactions

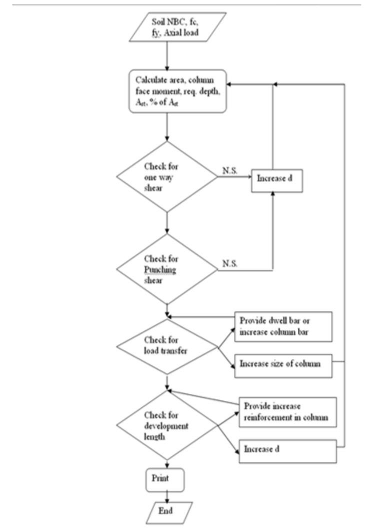
Position	Value of K		
	Top	Next to Top	Lower
Interior	1.0	1.0	1.0
Side	4.5	2.0	1.4
Corner	6.0	2.3	1.8

Design of Isolated Footing

Notes:

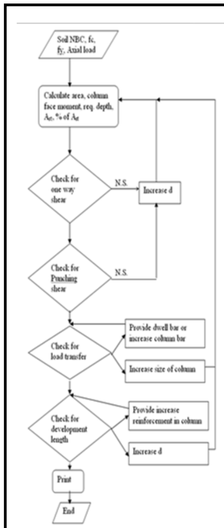
- Only 50% of the analyzed column moments is considered for foundation design as the column continues further down into soil it piled up by surrounding toe wall and soil
- Confining effects of toe wall and soil prevents bending of column
- So moment are ignored and only axial load are considered.

Design flowdiagram of foundation design

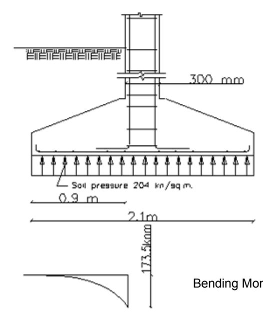


Design Data

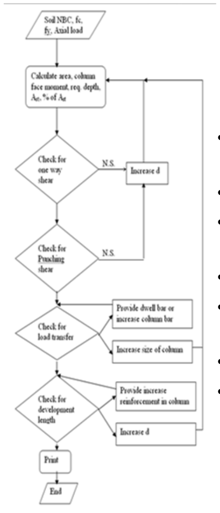
- Net load P= 600 kn
- Net bearing capacity of soil = 150 kn/sq.m.
- Column Size = 300 x 300 m
- Footing Concrete Mark = M15
- Column Concrete mark = M20
- Reinforcement Fe 415
- Column Reinforcement 8-16ϕ



- Self wt = 10 % of design load
- Req. area = 600*1.1 / 150 = 4.4 sq.m.
- Req. size = Sqrt (4.4) = 2.1 m
- Effective width be=b+(B-b)/8 = 525 mm
- Uplift design pressure p = 600*1.5/(2.1*2.1) = 204 kn

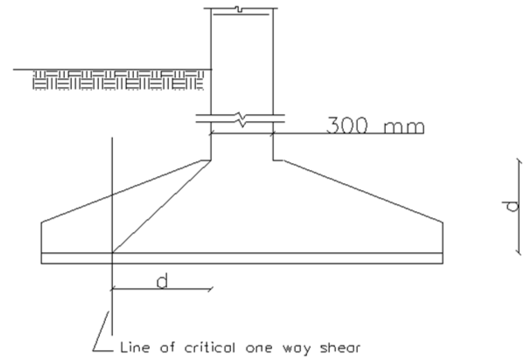


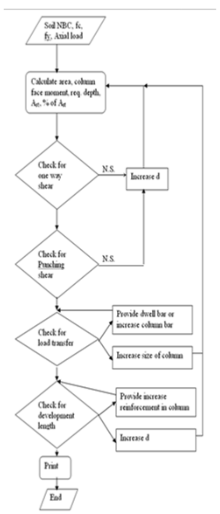
Bending Moment at column Face



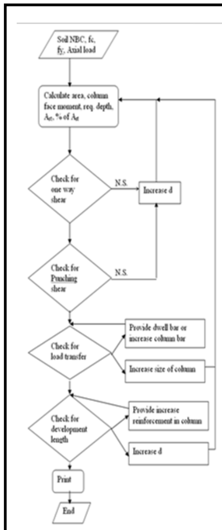
- Moment at column face = 204*2.1*.9^2/2 = 173.5 knm
- d_{min} = √(M/(.138*fb*be) = 400 mm
- Calculate Ast using formula : M = 0.87 * σ_y * Ast (d-($\sigma_y A_{t/(}\sigma_{ck*}$b) i.e.
- Ast = 1496 mm
- Constructive requirement i.e. 12ϕ@200c/c = (2100-100)/200+1)*113 = 1243
- Provide 1496
- Pt=0.71

Design in one way shear





- The critical section is taken at distance d away from face of column
- Vu= 204 *2.1((2.1-.30)/2-.4) = 214 kn
- Effective depth at the location = 150+(400-150)*(2100-300-2*400)/(2100-300) = 289 mm
- Top width = 300+(2100-300)*400/((2100-300)/2) = 1100
- Depth of NA=.48*Effective depth = 138.72
- Width at NA = 1100+(2100-1100)/(400-150)*138.72 = 1654
- τv = 214000/(1654*289) = 0.447 N/mm
- Shear strength of M15 concrete with 0.7% steel = 0.32 <0.447
- Adopt total depth 600 mm with cover 50mm

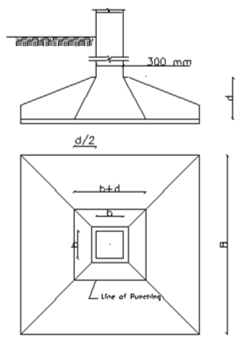


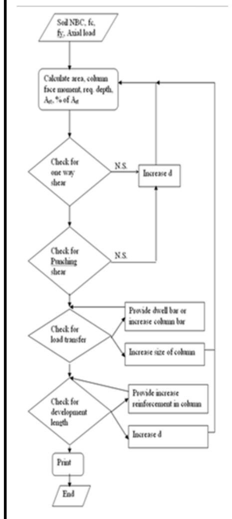
Revised calculation

- Calculate Ast using formula : M = 0.87 * σ_y * Ast (d-($\sigma_y A_{t/(}\sigma_{ck*}$b) i.e.
- Ast = 962 sq.mm.
- Minimum requirement 0.12% of 525*550 = 346 sq.mm.
- Constructive requirement i.e. 12ϕ@200c/c = (2100-100)/200+1)*113 = 1243
- Provide 1243
- Pt=0.43
- Vu= 204 *2.1((2.1-.30)/2-.55) = 150 kn
- Effective depth at the location = 150+(550-150)*(2100-300-2*550)/(2100-300) = 305 mm
- Top width = 300+(2100-300)*550/((2100-300)/2) = 1400
- Depth of NA=.48*Effective depth = 146.4
- Width at NA = 1400+(2100-1400)/(550-150)*146.4= 1656.2
- τv = 150000/(1656.2*305) = 0.297 N/mm ≈ 0.29

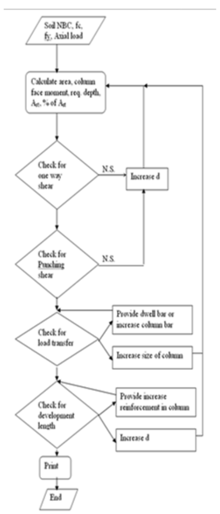
The Section is now safe on one way shear

Design for Punching shear



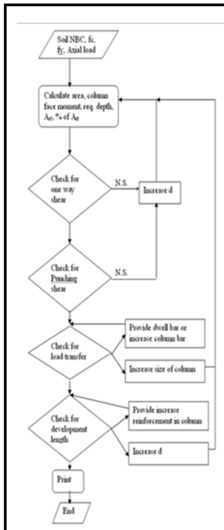


- The critical section is taken at distance d/2 away from face of column
- Punching length = 4*(b+d) =3.4 m
- Punching force = 204*(2.1*2.1-.85*.85) = 752 kn
- Effective depth at the location = 150+(550-150)*.625/.9 = 428 mm
- τ_v = 752000/(3400*428) = 0.52 N/mm
- τ_c = .25*√15 = 0.97 N/mm > 0.52
- The section is safe in punching



Check for load Transformation

- Nominal bearing stress in column concrete = 1.5*600/(300*300) = 10 N/sq.mm.
- Allowable bearing stress = 0.45*20 = 9
- Exceed load = 1*300*300 = 90 kn
- Req area of steel = 90000/.67/415 =323 sq.mm.
- Provided area = 1608 sq.mm.



Check for Development Length

- Development length for compression of 16ϕ bar = 34.75*16 = 556 mm
- Provided dev length = 550
- Req. development length = 323*556/1608 = 111 < 550
- OK

Combine footing for boundary condition

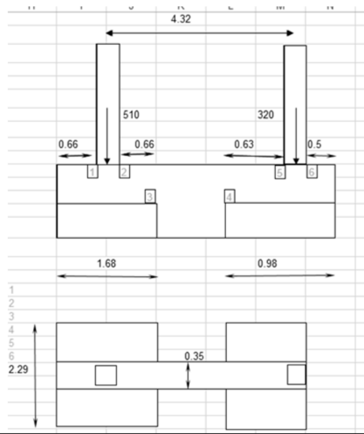
Steps of design

Find out centre of gravity of load

CGl = 320*4.32/(320+510) = 1.66

Develop quadratic equation for CG of Area.

Area req	6.09
Width prov	2.29
Total Length	2.66



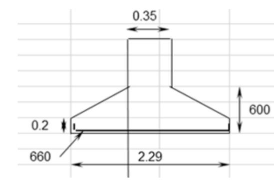
Combine footing for boundary condition

Coincide CG of area and CG of Load. Solve the quadratic equation. It gives length of both footings.

L1=(-(4.32+.35/2+.5)-SQRT((4.32+.35/2+.5)^2-4*0.5*(1.66+2.66))/(2*0.5)

L2 = Total length – L1

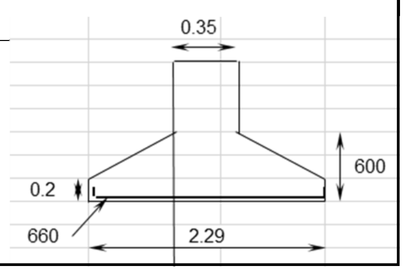
Determine strap beam width.
Find out projection of footing slab.
Design it for bending moment and check it for one way shear.



Slab Design

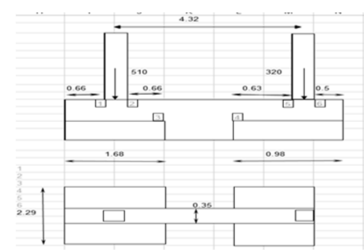
Width of beam			0.35		Req depth	215.23
Projection			0.97		d prov	550
Max B.M at face			95.89		At/m	495.22
Shear Force at distance d			85.56		Min Ast	660
Depth at critical SF			294.40		12@8"	566.00
Appl Shear stress			0.29		Act At/m	660.00
B Value	7.77	Actual B	7.77		Bar dia.	12.00
Lim Sh Stress	0.34				Dev. Lt	OK

The Section is Safe



Determine bending moment and share force in strap beam.

Design the strap beam



Per M pressure	467.7106		Distance	Design_SF	Des_BM
		0	0	0	
at central column face		1	0.66	310.39	102.99
at inner face		2	1.01	-290.91	106.40
at end of central slab		3	1.68	19.47	16.33
at end of corner slab		4	4.85	19.47	78.10
at corner column face		5	4.98	82.45	84.96
at outer face		6	5.33	-233.86	58.46
at corner		7	5.83	0	0.00

Design Procedure of Singly Reinforced Beam using Design Aids

- For Corner DBM=25.89 knM
- Concrete mix=M15
- Fy=415
- Beam Size=230x345
- Effective cover=35mm
- Mu,lim from Table D=2.07*1000*bd^2=45.75
- Section is to be designed as Singly reinforcement
- Mu/bd^2=1.17N/mm^2
- d'/d=0.1
- Refer Table 1
- Pt=0.362
- Ast=258 mm^2

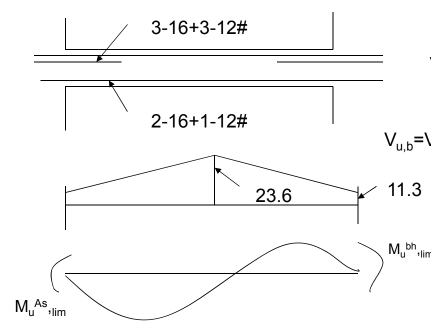
Design Procedure of Doubly Reinforced Beam using Design Aids

- For Corner DBM=90.91 knM
- Concrete mix=M15
- Fy=415
- Beam Size=230x345
- Effective cover=35mm
- Mu,lim from Table D=2.07*1000*bd^2=45.75
- Section is to be designed as doubly reinforcement
- Mu/bd^2=4.11N/mm^2
- d'/d=0.1
- Refer Table 49
- Pt=1.34 Pc=0.65
- Ast=1063 mm^2 ; Asc=516 mm^2

Ductile requirements on Longitudinal Reinforcements

- Tension steel ratio on any face pmin>0.24√(fck/fy)=0.45% i.e. 315mm^2
- Maximum steel ratio on any face – 2.5% i.e. 1782 mm^2
- Minimum positive reinforcement at end = ½ of negative reinforcement
- Minimum reinforcement = .25xmax. Negative reinforcement

Calculation of Design Shear Force



$V_a^{D+L}=V_b^{D+L}=1.2x(D+L)/2=41.88$

$M_u^{bh},_{lim}=89.4$, $M_u^{As},_{lim}=51.03$

$V_{u,b}=V_a^{D+L}+1.4*(M_u^{As}{}_{lim}+M_u^{bh},_{lim})/L_{ab}=91.03$

Stirrups as per design requirement: 6#@108c/c

Minimum stirrups requirements:

Spacing over a length 2d at either ends=d/4=77 or 12times # of main bar=96 but need not less than 100mm c/c

suggestions, comments,
and Questions !!!