A

Project

Report

on

Analysis and design of G+4 story building

Advanced Concrete

DesignCE 632

Submitted By:

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Acknowledgement:

Completing a task is never a one- man effort. Contribution a number of individuals plays a major role in a direct or indirect manner. It brings me great pleasure and immense satisfaction to express my sincere and deepest sense of profound gratitude towards Dr. Dhiman Basu. And the course TA Narsiram Gurjar for giving the opportunity to learn many new things through this project and helping me guide along with it. I am also thankful to all the professors who have been of help for acquiring the practical knowledge of civil engineering works.

Contents:

- Performing structural analysis in SAP 2000
- Wind Load Calculation and equivalent static analysis for earthquake
- Design of Beam
- Design of Column
- Design of Beam-Column Joint
- Design of Shear Wall
- Design of raft foundation in safe foundation

Problem statement:

A building (G+4) located in Bhuj seismic zone V has to be designed as per IS456, IS13920, IS1893 and IS875 part 3 for 50 year design life for dead loads, live loads and lateral loads acting on it.

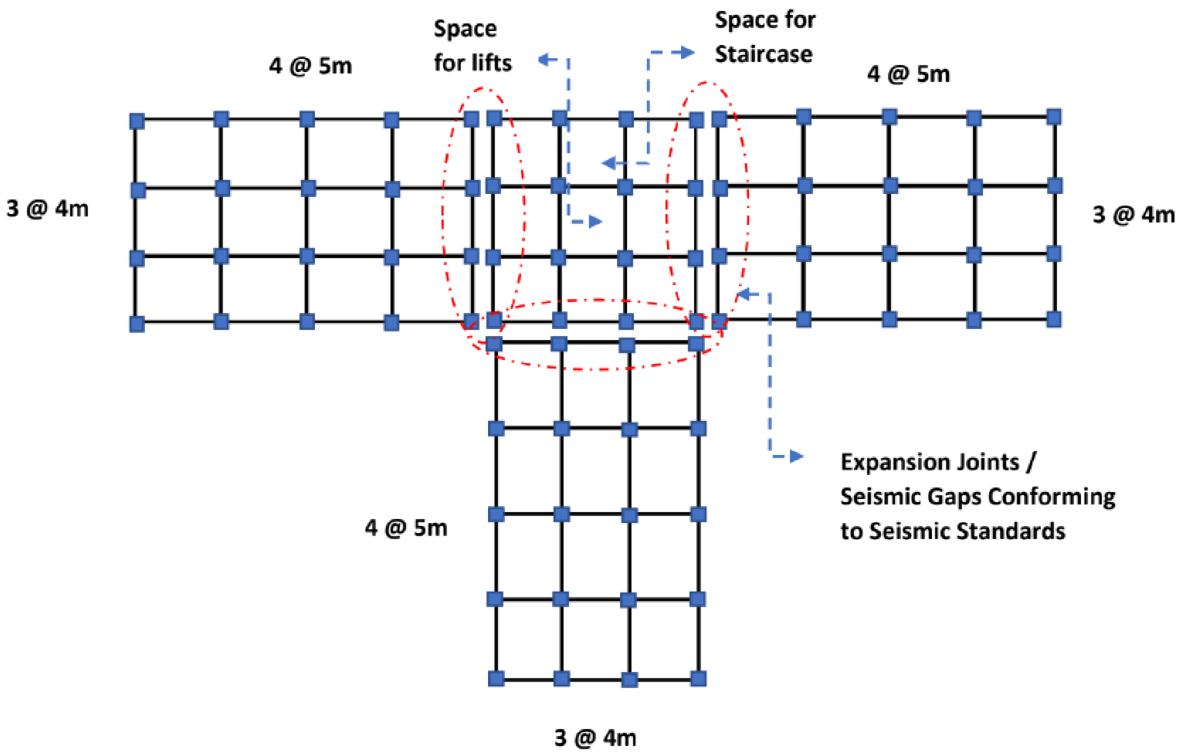


Figure 1: Typical Plan layout (except top storey) of a G+4 storeyed building with open ground storey

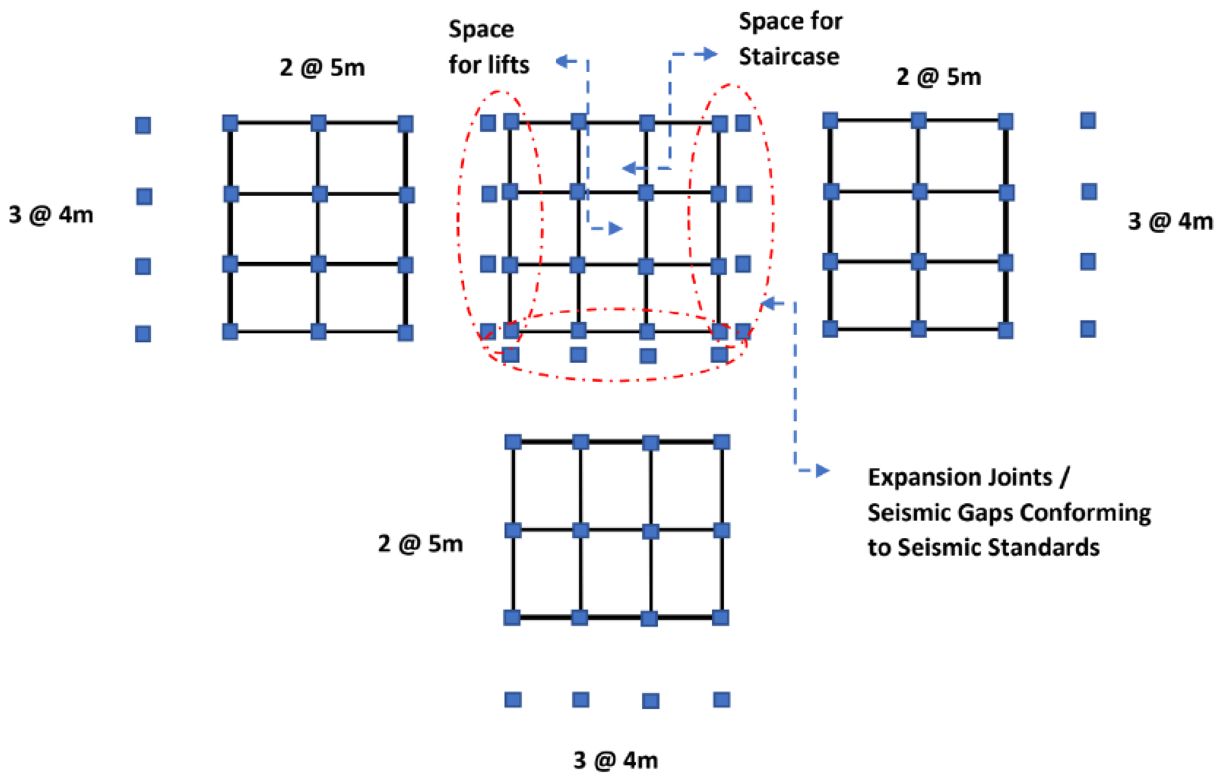


Figure 2: Plan layout of top storey (Dedicated for Amusement Facilities)

In this design and analysis one of the 5 building is considered and analyzed

Objectives: -

- Performing structural analysis in SAP 2000
- Wind Load Calculation and equivalent static analysis for earthquake
- Design of Beam
- Design of Column
- Design of Beam-Column Joint
- Design of Shear Wall
- Design of raft foundation

Wind load calculation:

Wind is a the motion of air relative to the surface of earth, the primary cause of wind is traced to the earth's rotation and terrestrial radiation differences uneven heating of earth surface, when wind blows it exerts a lateral force on the structure and the lateral force can depend on factors such as location, terrain, topology factors etc. IS 875 part 3 identifies that the basic wind speed depends upon these 4 factors Basic wind speed : Vb can be obtained from the following figure 1 and the design wind speed Vz would depend upon these factors:

a. Risk level (k1) (Clause 6.3.1)
b. Terrain roughness and structure height (k2) (Clause 6.3.2)
c. Local topography (k3) (Clause 6.3.3)
d. Importance factor for cyclonic regions (k4) (Clause 6.3.4)

We can mathematically express it as follows:

Vz = Vb*k1*k2*k3*k4

Wind Pressure calculation:

The wind pressure at any height above mean ground level can be given based on design wind speed by,

$$p_z = 0.6\,V_b^2$$

where,
pz = wind pressure at height z, in N/m2
and
Vz = design wind speed at height z, in m/s.

And the design wind pressure will depend on these factors:

a. Wind directionality factor Kd (due to randomness in the directionality of wind factor 0.9 used for rectangular square and triangular and 1 for circular structures)
b. Area coverage factor and Ka (given in table 4 as per the tributary area of nodes)
c. Combination factor Kc

Mathematically it can be expressed as.

$$Pd = K_d K_a K_c p_z < 0.7*pz$$

Force of wind calculation: the force at each floor levels can be obtained by the expression given in code(Clause 7.4):

F = Cf $A_e$pd,… Ae is the effective area
And, C_f is obtained from Cl. 7.4.2.1 and Fig 4

Our problem of structure located in bhuj is having a basic wind speed of Vb = 50 m/s. from figure 1 in IS code
As the terrain of bhuj has very less obstructions so it is assumed a terrain category 3

For 50 year design life k1 = 1 (Cl. 6.3.1 Table 1)

Table 1 Risk Coefficients for Different Classes of Structures in Different Wind Speed Zones
(*Clause* 6.3.1)

Sl No.	Class of Structure	Mean Probable Design Life of Structure in Years	k_1 Factor for Basic Wind Speed m/s					
			33	39	44	47	50	55
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
i)	All general buildings and structures	50	1.0	1.0	1.0	1.0	1.0	1.0
ii)	Temporary sheds, structures such as those used during construction operations (for example, formwork and false work), structures during construction stages and boundary walls	5	0.82	0.76	0.73	0.71	0.70	0.67
iii)	Buildings and structures presenting a low degree of hazard to life and property in the event of failure, such as isolated towers in wooded areas, farm buildings other than residential buildings	25	0.94	0.92	0.91	0.90	0.90	0.89
iv)	Important buildings and structures such as hospitals communication buildings/towers, power plant structures	100	1.05	1.06	1.07	1.07	1.08	1.08

Hence k2 values are as follows with height, (Cl. 6.3.2 Table 2)

Table 2 Factors to Obtain Design Wind Speed Variation with Height in Different Terrains
(*Clause* 6.3.2.2)

Sl No.	Height z m	Terrain and Height Multiplier (k_2) Terrain Category 1	Terrain Category 2	Terrain Category 3	Terrain Category 4
(1)	(2)	(3)	(4)	(5)	(6)
i)	10	1.05	1.00	0.91	0.80
ii)	15	1.09	1.05	0.97	0.80
iii)	20	1.12	1.07	1.01	0.80
iv)	30	1.15	1.12	1.06	0.97
v)	50	1.20	1.17	1.12	1.10
vi)	100	1.26	1.24	1.20	1.20
vii)	150	1.30	1.28	1.24	1.24
viii)	200	1.32	1.30	1.27	1.27
ix)	250	1.34	1.32	1.29	1.28
x)	300	1.35	1.34	1.31	1.30
xi)	350	1.35	1.35	1.32	1.31
xii)	400	1.35	1.35	1.34	1.32
xiii)	450	1.35	1.35	1.35	1.33
xiv)	500	1.35	1.35	1.35	1.34

NOTE — For intermediate values of height *z* in a given terrain category, use linear interpolation.

k2(for heights in range 0-10,10-15 and 15-20m)	0.91	0.97	1.01

Also k3 =1 refers to the local terrain or slope the building is situated in, Bhuj has little mountains region the building is assumed to be in a flat plane assumed to be in a slope less than 3 (As per Clause 6.3.3:)

K4 factor considered as 1 as the building is commercial use it is not considered of special importance in cyclonic storms like hospitals etc. (Clause. 6.3.4)

	k_4
Structures of post-cyclone importance for emergency services (such as cyclone shelters, hospitals, schools, communication towers, etc)	1.30
Industrial structures	1.15
All other structures	1.00

Next we find directionality and tributary area factors,

Kd = 0.9 for non circular and 1 for circular buildings (Clause 7.2.1)
As wind direction is more variable for non circular buildings therefore a reduction factor of 0.9 is taken

Kc = 1 combined effect assumed 1, due to cladding of buildings the resultant force of wind does not come in the same line of incidence therefore a factor 0.9 is used with cladding. (Clause 7.3.3.13)

The tributary areas are obtained by joining center to center lines of the bays
In both x and y wind direction the value of tributary area is less than 25 hence as per (Clause 7.2.2, Table 4)

Ka = 0.9

We summarize our values,

Vb(basic wind speed)	50		
k1(risk factor)	1		50year life
k2(for heights in range 0-10,10-15 and 15-20m) for terrain category 3	0.91	0.97	1.01
k3(local topology factor)	1		
k4(importance to cyclonic region factor)	1		
kd(directionality factor)	0.9		
ka(tributary area factor)	0.9	AreaX=4x3	AreaY=5x3
kc(combined factor)	1		

terrain category	3		
building dimension when wind along X (ax)	20		
building dimension when wind along X (bx)	12		
building dimension when wind along Y (ay)	12		
building dimension when wind along Y (by)	20		
height of building above ground (h)	16		

Table 1: summary of factors of wind analysis

Table 4 Area Averaging Factor (K_a)
(*Clause* 7.2.2)

Sl No.	Tributary Area (A) m^2	Area Averaging Factor (Ka)*
(1)	(2)	(3)
i)	≤10	1.0
ii)	25	0.9
iii)	≥100	0.8

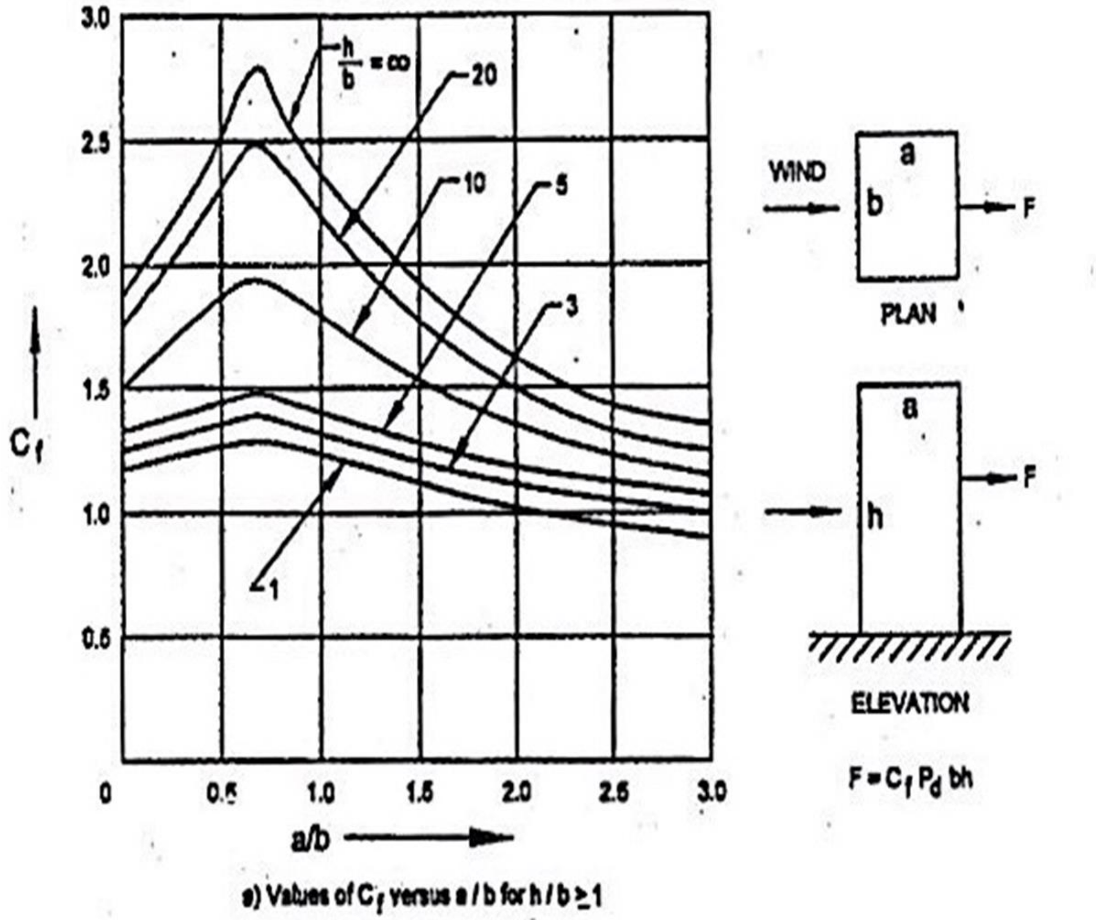
* Linear interpolation for intermediate values of a is permitted.

Calculate Cf values in both direction from table 4 IS875

Summarized shown

(a/b)x	1.67		(a/b)y	0.6
(h/b)x	3.21		(h/b)y	1.93
Cfx (coefficient of wind)	1.1		Cfy	1.3

Table2: Cf calculation



a) Values of C_f versus a / b for h / b $\geq$ 1

The design wind pressure is calculated hence given as,

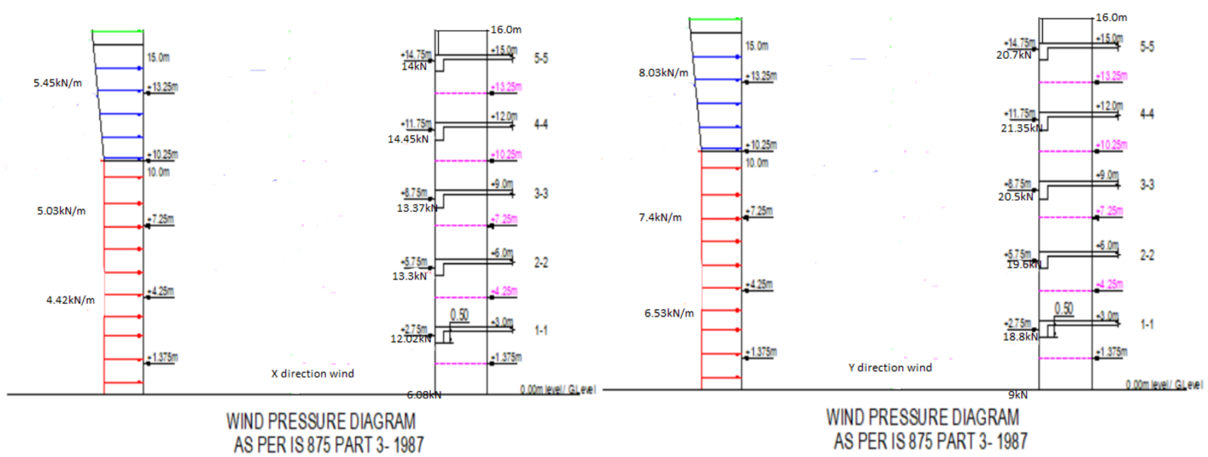
	10m	15m	20m
K1k2k3	0.91	0.97	1.01
Vb	50	50	50
Vd(m/s) = k1k2k3Vb	45.5	48.5	50.5
pz(wind pressure) = 0.6 V_b^2	1.24215	1.41135	1.53015
pd(design wind pressure) = $K_d K_a K_c p_z$ < 0.7*pz	1.006142	1.143194	1.2394215

Design wind pressure calculation Table3:

Wind calculations		per meter height								
Story height	pd	height	dimension along wind load in x direction	area effective Ax	Cf x	Fx =CfxAex pd 'kN/m'	dimension along wind in Y direction	Area Ay	Cf y	Fy ==CfxAex pd 'kN/m'
3	1.0061	10	4	4	1.1	4.427022	5	5	1.3	6.53992
3	1.1431	15	4	4	1.1	5.030051	5	5	1.3	7.43075
3	1.2394	20	4	4	1.1	5.453454	5	5	1.3	8.05624

Table 4: force per unit height calculation in X and Y direction

The wind load is calculated above is per unit height of building we will distribute the wind load on the basis of the tributary area of each node the wind force is assumed to be acting at the center of beam-column joint till a distance of h/2 to top and bottom. the wind pressure diagram is obtained below and the value in between at each story is interpolated to get nodal force.



X direction Y direction

Figure3: wind force diagram

story no.	loading level	height of story	Design force in X from interpolation	Fx (kN)	design force in y from interpolation	Fy (kN)
5-5(14.75m)	+13.25 to 16	2.75	5.11473204	5.11x2.75=14.06551	7.55585415	7.5x2.75=20.77859891

4-4(11.75m)	+10.25 to 13.25	3	4.81899132	4.82x3=14.45697	7.11896445	7.11x3=21.35689335
3-3(8.75m)	+7.25 to 10.25	3	4.45717404	4.46x313.37152	6.83654985	6.83x3=20.50964955
2-2(5.75m)	+4.25 to 7.25	3	4.4270226	4.43x3=13.28107	6.53991975	6.54x319.61975925
1-1(2.75m)	+1.375 to 4.25	2.875	4.4270226	4.42x2.285=12.72769	6.53991975	6.57x2.875=18.80226928

Table 5: Nodal wind force calculation:

To analyze for these force they are applied at each node in sap2000.

Equivalent static analysis:

The equivalent static lateral force method is a simplified method to substitute for the effect of dynamic loading of an excepted earthquake by static force distribution laterally on the structure for design purposes

The whole dynamic behavior of building will not be considered instead the whole structure is considered as an equivalent spring and mass system with some stiffness and damping and with the time period of that structure the peak design acceleration coefficient is calculated based on which a maximum base shear force can be calculated and distributed on each story of the floor to be analyzed for.

To calculate its time period we need to convert the structure into lumped mass and spring system for that we consider the seismic weight of the structure which is calculated for each story
Consider structure is RCC with unit weight 25kN/m3 and masonry wall 19.2kN/m3 live load acting is 2.5kN/m2 on all floors and 1.5kN/m2 on top roof.

We calculate the total dead weight of each story

The assumed dimensions of the structural members are as follows,

Section properties of members:

-Beams: 300 mm X 500 mm

-Columns: 450 mm X 450 mm

Length of beams = 5m,4m in 2orthogonal direction

Length of columns = 3m

Shear wall 4mx0.25mx30m

In a typical mid story the total load acting will be given as,

Total weight of column in one floor = 12column x0.45mx0.45mx3m=7.29m3x(25kN/m3) = 182.25kN
Total beam weight = 14x5x0.3x0.5x10.5x25 + 13x4x0.3x0.5x7.8x25=149.76 = 351kN
Shear wall weight = 2x5x0.25x3x7.5x25 + 2x4x0.25x3x6x25=337.5kN
Boundary element = 8column x0.45mx0.45mx3m=7.29m3x(25kN/m3) = 121.5kN
Walls = 14x5x0.25x2.5x43.75x19.2 + 13x4x0.25x2.5x32.5x19.2 = 1717kN
Live load = 12x5mx4mx2.5kN/m3 = 150kN
Total = 3662.86kN

In top floor,

Total weight of column in one floor = 8column x0.45mx0.45mx3m=7.29m3x(25kN/m3) = 121.25kN
Total beam weight = 6x5x0.3x0.5x10.5x25 + 9x4x0.3x0.5x7.8x25=149.76 = 247.5kN
Shear wall weight = 2x5x0.25x3x7.5x25 =187.5kN
Boundary element = 4column x0.45mx0.45mx3m=7.29m3x(25kN/m3) = 60.75kN
Walls = 6x5x0.25x2.5x43.75x19.2 + 9x4x0.25x2.5x32.5x19.2 = 792kN
Live load = 12x5mx4mx1.5kN/m3 = 90kN
Total = 1278.375kN
At 4th floor,
Total weight of column in one floor = 8column x0.45mx0.45mx3m=7.29m3x(25kN/m3) = 121.25kN
Total beam weight = 6x5x0.3x0.5x10.5x25 + 9x4x0.3x0.5x7.8x25=149.76 = 247.5kN
Shear wall weight = 2x5x0.25x3x7.5x25 =187.5kN
Boundary element = 4column x0.45mx0.45mx3m=7.29m3x(25kN/m3) = 60.75kN
Walls = 6x5x0.25x2.5x43.75x19.2 + 9x4x0.25x2.5x32.5x19.2 = 792kN
Live load = 12x5mx4mx2.5kN/m3 = 150kN
Total = 3131kN

Half of the weight of top story and half the weight of story below it will contribute to lumped mass

The total dead weight of building is 15466kN

Height of building (h) = 16m
Dimensions of building d1xd2 = 20mx12m

Hence from clause 7.6.2 b) IS 1893 we get the time period as,

H = $\frac{\sum A_i h_i}{\sum A_i} = 18m$

$A_{wx} = \sum_i^{Nw}[A_{wi}\{ 0.2 + \left(\frac{L_{wi}}{h}\right)^2\}] = 0.45$

$A_{wy} = \sum_i^{Nw}[A_{wi}\{ 0.2 + \left(\frac{L_{wi}}{h}\right)^2\}] = 0.77$

Tax = $\frac{0.075h^{0.75}}{Aw^{0.5}} \geq \frac{0.09h}{d^{0.5}} = 0.97 > 0.36$

Tay = $\frac{0.075h^{0.75}}{Aw^{0.5}} \geq \frac{0.09h}{d^{0.5}} = 0.74 > 0.36$

From figure 2 IS 1893 we get the Sa/g value as 1/T

Hence (Sa/g)x= 1.03, (Sa/g)y=1.35

The coefficients required to calculate base shear are given below,

soil type	2 (bhuj located)
I	1.2 (commercial building)
zone	5 (bhuj)
Z	0.36(zone 5)
h	16
d in X direction	20
d in Y direction	12
R	5 (table 9, for RC building braced)

$$A_h = \frac{\frac{Z}{2}(\frac{S_a}{g})x}{\frac{R}{I}} = 0.04$$

$$A_h = \frac{\frac{Z}{2}(\frac{S_a}{g})y}{\frac{R}{I}} = 0.05$$

Hence base shear = seismic weight x Ah = 1600kN

The base shear is distributed parabolically over height given by,

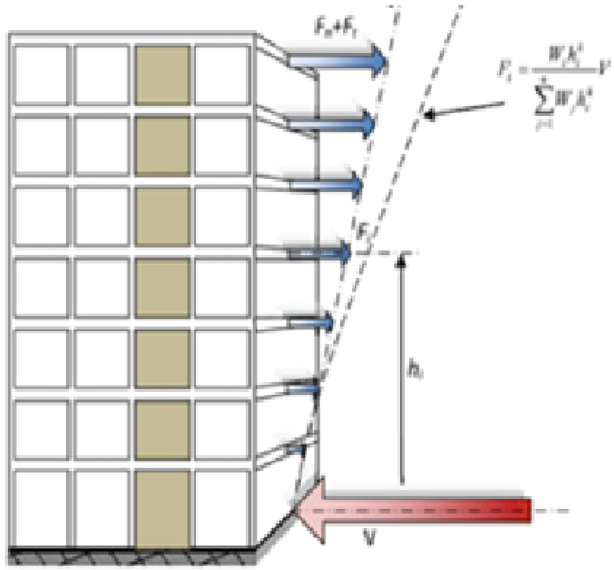
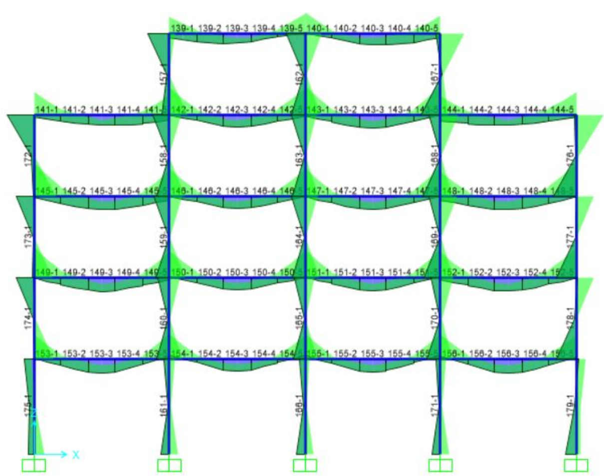


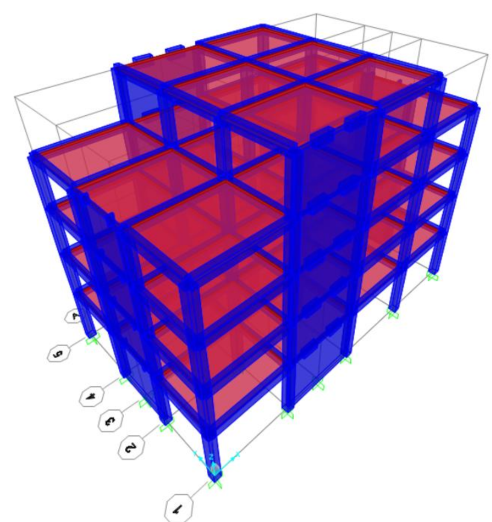
Table shown below calculated the story shear which will be applied at the cg of story in sap2000 for analysis

	story shear						
Wi		Hi	Hi^2	Wi*Hi^2	$(W_i*H_i\text{^}2)/(\Sigma W_i*H_i\text{^}2)$	Qiy(kN)	Qix(kN)
1278.375		15	225	287634.375	0.238583498	383.4737	383.4737
3131.735		12	144	450969.84	0.374065034	601.2323	601.2323
3730.36		9	81	302159.16	0.250631342	402.8382	402.8382
3662.86		6	36	131862.96	0.109376101	175.7995	175.7995
3662.86		3	9	32965.74	0.027344025	43.94988	43.94988
15466.19				1205592.08			

Beam design

Beam 155 and 156 are considered for design





The shear force bending moment diagram
The properties assumed are as follows: -

-Concrete grade: M35

-Steel Grade: Fe 415

Section properties of members:

-Beams: 300 mm X 500 mm

-Columns: 450 mm X 450 mm

Length of beams = 5m

Length of columns = 3m

	Left End	**Mid-Section**	**Right End**
V_u (kN)	96	9	96.6
$M_{+,max}$ (kN-m)	0	45.77	0
$M_{-,max}$ (kN-m)	-93.46	-	-95

For Fe415, 300x500 cross section,M35 with d'=30 the value of $X_{u,max}$ comes out to be,

$X_{u,max}$ = 215.5kNm,

$M_{u,lim} = 0.36 f_{ck} x_{u,lim} b(d - 0.42 x_{u,lim})$

$$= 0.36 \times 35 \times 215.55 \times 300 \times (450 - 0.416 \times 215.55)$$

$= 293.6$ kNm $>$ Mu,applied
Hence singly reinforced design

Design at left and right sections:

Area of reinforcement given by,

$$A_{st} = \frac{0.5fy}{fck}\left(1 - \frac{4.6Mu}{fckbD^2}\right)^{0.5} bd$$

$$A_{st} = \frac{0.5*35}{415}(1 - \left(1 - \frac{4.6*95*10^6}{35*300*450^2}\right)^{0.5})300 * 450$$

= 618 mm^2

Minimum reinforcement check as per IS13920

$$A_{st,min} = \frac{0.24\, fck^{0.5}}{fy}$$

= 513.2 mm^2

Hence provide reinforcement for 611mm^2

Provide 4 number 16 dia bars at top to resist the tensile force

Ast, top = 802.24 mm^2

for bottom reinforcement, as per Cl. 6.2.3 of IS 13920:2016) the reinforcement is taken 0.5*of provided at tension side take minimum reinforcement

hence take 3 number 16 dia bars at botton

$$Ast, bottom = \frac{\pi}{4} \times 16^2 \times 3 = 603\, mm^2$$

Design at mid-section:

Area of steel at mid-section will be given by,

$$A_{st} = \frac{0.5 * 35}{415} (1 - \left(1 - \frac{4.6 * 45 * 10^6}{35 * 300 * 450^2}\right)^{0.5}) 300 * 450$$

=293mm2

Whereas $A_{st,min} = 513.2\, mm^2$

Hence provide minimum reinforcement at top and bottom both 3 number 16 diameter bars

Now we will do ductility design as per IS13920 for beam which will ensure ductile behavior of overall structure in case of earthquake

The basic concept of ductile detailing refers to the ability of a structural system to be able to resist certain deformations beyond its yield point without significant loss in strength, to avoid sudden brittle damage in earthquake or fierce wind the overall structural ductility comes from the member ductility hence it is important to ensure that we design all members in a ductile way

As per clause 6.3.3 IS13920 the design shear force must be maximum of these:

$$max \begin{cases} factored\ shear\ force\ from\ structural\ analysis \\ \qquad Vu, a = V_{u,a}^{D+L} + 1.4 \frac{M_u^{As} + M_u^{Bh}}{L_{beam}} \\ \qquad Vu, b = V_{u,b}^{D+L} - 1.4 \frac{M_u^{As} + M_u^{Bh}}{L_{beam}} \end{cases}$$ considering both sway from left and right

Fig Sway in right

$M_u^{As} = 0.87 f_y A_{st,bottom}(d - d') = 0.87 * 415 * 3 * \frac{3.1415}{4} * 16^2 * (450\text{-}50) = 87.08kNm$

$M_u^{bh} = 0.87 f_y A_{st,top}(d - d') = 0.87 * 415 * 4 * \frac{3.1415}{4} * 16^2 * (450 - 50) = 115.82kNm$

Vu = 96kN ….. from structural analysis.

$Vu, a = V_{u,a}^{D+L} - 1.4 \frac{M_u^{As} + M_u^{Bh}}{L_{beam}} = 76 - 1.4 \frac{87.08 + 115.82}{5}$ = 19.18kN

$Vu, b = V_{u,b}^{D+L} - 1.4 \frac{M_u^{As} + M_u^{Bh}}{L_{beam}} = 76 + 1.4 \frac{115.82 + 87.08}{5}$ = 132.8kN

For left sway,

$M_u^{As} = 0.87 f_y A_{st,bottom}(d - d') =$ 115.82kNm

$M_u^{bh} = 0.87 f_y A_{st,top}(d - d') = 87.08kNm$

Vu = 68.21kN ….. from structural analysis.

$Vu,a = V_{u,a}^{D+L} - 1.4\frac{M_u^{As}+M_u^{Bh}}{L_{beam}} = 76 + 1.4\frac{115.82+87.08}{5}$ = 132.81kN (we take design shear)

$Vu,b = V_{u,b}^{D+L} - 1.4\frac{M_u^{As}+M_u^{Bh}}{L_{beam}} = 76 - 1.4\frac{115.82+87.08}{5}$= 19kN

Calculate τc maximum as per IS456

$100 * \frac{As}{bd}$ = 0.93

Get, τc = 0.64<3.7N/mm2 τc,max for M35 concrete table 19,20 IS456

τv = $\frac{Vu}{bd}$ = 0.97 >concrete strength hence provide shear reinforcement

provide 2 legged 10mm stirrups

$Vus = \frac{0.87A_{sv}f_yd}{S_v} = get\ S_v = 190mm$

As per IS13920 clause 6.3.5 the spacing of a link over a length of 2d shall not exceed:

a. d/4 = 112.5mm
b. 8x dia of smallest bar = 128mm
c. 100mm

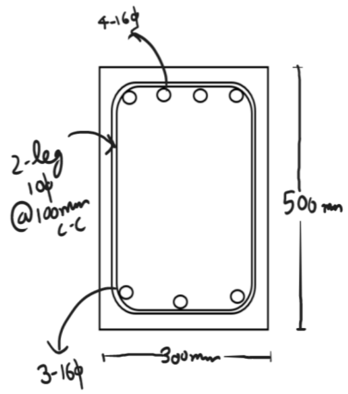
Therefore, provide 100mm c-c spacing for 2d length from ends

And the first link shall not be at a distance 50mm from joint face

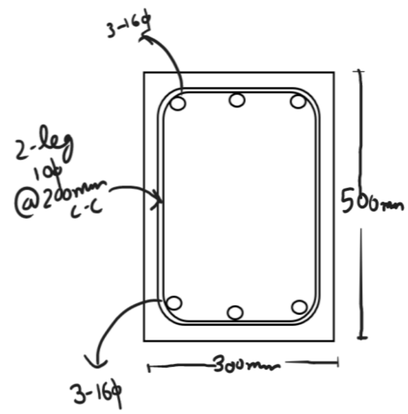
Max spacing = 0.5d = 225mm hence lets provide 200mm c-c outside 2d length

Design summary,

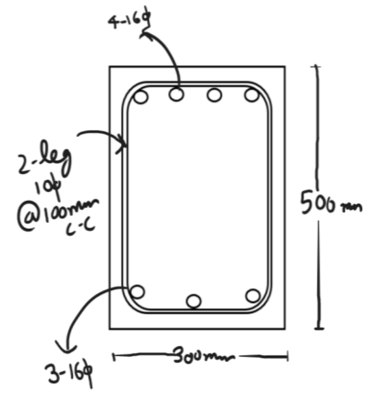
Beam Section	$\mathbf{A_{st,top}}$	$\mathbf{A_{st,bottom}}$	**Shear Stirrup**
Left end	4-16φ	3-16φ	2leg-10φ @ 100 mm c/c
Mid-Span	3-16φ	3-16φ	2leg-10φ @ 200 mm c/c
Right end	4-16φ	3-16φ	2leg-10φ @ 100 mm c/c



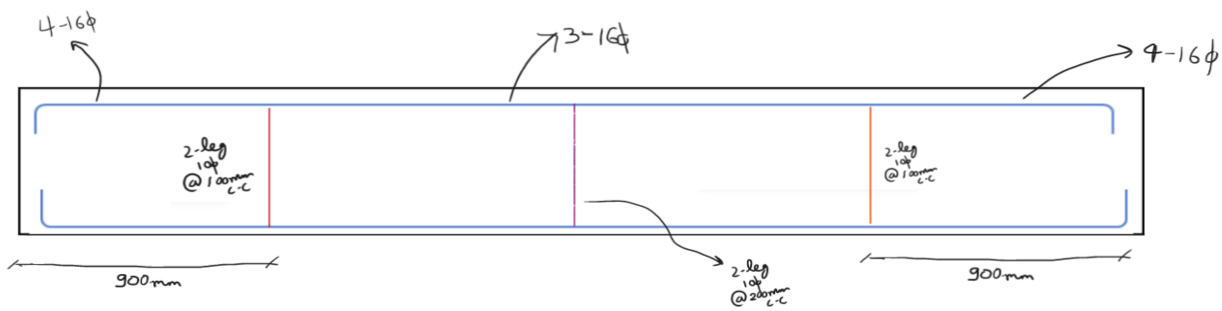
Left section



Mid section



Right section



Details of beam

Design of beam 156:

	Left End	Mid-Section	Right End
V_u (kN)	96	10	96
$M_{+,max}$ (kN-m)	0	50	0
$M_{-,max}$ (kN-m)	-94	-	-94

For this beam the reinforecement area comes,

$$A_{st} = \frac{0.5fy}{fck}\left(1 - \frac{4.6Mu}{fckbD^2}\right)^{0.5} bd$$

$$A_{st} = \frac{0.5*35}{415}(1 - \left(1 - \frac{4.6*94*10^6}{35*300*450^2}\right)^{0.5})300*450$$

= 611 mm^2 Whereas $A_{st,min}$= 513.2 mm^2 in beam 155 also we see the same values, Hence we can provide reinforcement details same as the beam 155

Design of column:

Column	Design Load
Moment, M_2 (kNm)	52
Moment, M_3 (kNm)	50.52
Shear, V_u (kN)	33
Axial force, P_u (kN)	1365.64

Check for slender column,
Braced column fixed at both ends,

$$\lambda_x = \lambda_y = \frac{l_{eff}}{d} = 0.85 * \frac{3}{0.45} = 5.67 < 12(short\ column\ design)$$

Check for minimum eccentricity as per IS456,

$$e_{min} = \max\left(\frac{l_x/l_y}{500}, \frac{D}{30}\right) or \; 20$$

$$= \frac{3000}{500} + \frac{450}{30} = 21mm < 0.05D = 22.5$$

M_{ux} = 52 kNm,
M_{uy} = 50.52 kNm,

Check eccentricity in x and y direction,

$$e_{x,min} = \frac{52*1000}{1365.64} = 38mm$$
$$e_{y,min} = \frac{50520}{1365.64} = 37.64mm$$

We will design for ex and ey eccentricities as they are more than emin

Consider maximum equivalent bending moment,

$$M_u = 1.2\sqrt{M_{ux}^2 + M_{uy}^2} = 87kNm$$

Pu = 1365.64kN

Are our design axial load and equivalent moment

For $\frac{d'}{d} = \frac{45}{450} = 0.1$

And $\frac{P_u}{f_{ck}bD} = \frac{1365.64*1000}{35*450*450} = 0.2$

$\frac{M_u}{f_{ck}bD^2} = \frac{87*10^6}{35*450*450^2} = 0.027$

Looking at chart 44 IS sp16 we get,
$\frac{p}{f_{ck}} = 0$
Hence adopt minimum reinforcement 0.8%

i.e. $A_{st} = \frac{0.8}{100} * 450 * 450 = 1620mm^2$
provide 8 bars of 20 diameter

$Ast, provided = 2513mm^2$

$Ast, max = 4 * 450 * \frac{450}{100} = 8100mm^2$ $as,$ $\text{Ast, provided} < \text{Ast, max Hence, O.K}$

Now we check the moment capacity in x and y direction,

Pt = 1.24%
$\frac{p}{fck} = 0.035$ and $\frac{P_u}{f_{ck}bD} = \frac{1365.64*1000}{35*450*450} = 0.2$

Chart 44 IS sp16,

Get,
$\frac{M_u}{f_{ck}bD^2} = 0.085$

This is uniaxial capacity moment $M_{ux1} = M_{uy1} = 271kNm$ same due to square column

Find axial load capacity,
$P_{uz} = 0.446f_{ck}A_c + 0.75f_yA_{st}$
= 0.446*35*(450*450-2513)+0.75*415*2513
= 3904kN

$\frac{P_u}{P_{uz}} = 0.35$

Calc, $\propto_n = \frac{2}{3}\left(1 + \frac{5}{2}\frac{P_u}{P_{uz}}\right) = 1.25$

Check now,

$(\frac{M_{ux}}{M_{ux1}})^{\propto_n} + (\frac{M_{uy}}{M_{uy1}})^{\propto_n} = (\frac{50}{271})^{1.25} + (\frac{50.2}{271})^{1.25}$= 0.27<1 hence safe

Design for column 171

Column	Design Load
Moment, M_2 (kNm)	35
Moment, M_3 (kNm)	32
Shear, V_u (kN)	17
Axial force, P_u (kN)	1800

Check for slender column,
Braced column fixed at both ends,

$$\lambda_x = \lambda_y = \frac{l_{eff}}{d} = 0.85 * \frac{3}{0.45} = 5.67 < 12(short\ column\ design)$$

Check for minimum eccentricity,

$$e_{min} = \max\left(\frac{l_x/l_y}{500}, \frac{D}{30}\right) or \atop 20$$

$$= \frac{3000}{500} + \frac{450}{30} = 21mm < 0.05D = 22.5$$

M_{ux} = 35 kNm,
M_{uy} = 32 kNm,

Check eccentricity in x and y direction,

$e_{x,min} = \frac{35*1000}{1800}$ = 19.4<emin hence we adopt $M_{ux} = P_u\, e_{x,min} = 38kNm$

$e_{y,min} = \frac{32*1000}{1800}$ = 18<emin hence we adopt $M_{uy} = P_u\, e_{y,min} = 38kNm$

We will design for ex,min and ey,min eccentricities as ex,min and ey,min are more than ex and ey applied

Consider maximum equivalent bending moment,

$$M_u = 1.2\sqrt{M_{ux}^2 + M_{uy}^2} = 91.2kNm$$

Pu = 1800kN

Are our design axial load and equivalent moment

For $\frac{d'}{d} = \frac{45}{450} = 0.1$

And $\frac{P_u}{f_{ck}bD} = \frac{1800*1000}{35*450*450} = 0.25$

$\frac{M_u}{f_{ck}bD^2} = \frac{91.2*10^6}{35*450*450^2} = 0.028$

Looking at chart 44 IS sp16 we get,
$\frac{p}{f_{ck}} = 0$
Hence adopt minimum reinforcement 0.8%

i.e. $A_{st} = \frac{0.8}{100} * 450 * 450 = 1620mm^2$
provide 8 bars of 20 diameter

$Ast, provided = 2513mm^2$

$Ast, max = 4 * 450 * \frac{450}{100} = 8100mm^2\ as, \mathrm{Ast, provided} < \mathrm{Ast, max\ Hence, O.K}$

Now we check the moment capacity in x and y direction,

Pt = 1.24%
$\frac{p}{fck} = 0.035$ and $\frac{P_u}{f_{ck}bD} = \frac{1764.42*1000}{35*450*450} = 0.25$

Chart 44 IS sp16,

Get,
$\frac{M_u}{f_{ck}bD^2} = 0.08$

This is uniaxial capacity moment $M_{ux1} = M_{uy1} = 255kNm$ same due to square column

Find axial load capacity,
$P_{uz} = 0.446f_{ck}A_c + 0.75f_yA_{st}$
= 0.446*35*(450*450-2513)+0.75*415*2513
= 3904kN

$\frac{P_u}{P_{uz}} = 0.45$

Calc, $\propto_n = \frac{2}{3}\left(1 + \frac{5}{2}\frac{P_u}{P_{uz}}\right) = 1.42$

Check now,

$(\frac{M_{ux}}{M_{ux1}})^{\propto_n} + (\frac{M_{uy}}{M_{uy1}})^{\propto_n} = (\frac{38}{255})^{1.42} + (\frac{38}{255})^{1.42}$ = 0.129<1 hence safe column

In ductility design it is important that strength of columns is much more compared to strength of the beams at a joint of beam and column to ensure safe as well as ductile behavior when there is lateral load acting on a structure there it is important to ensure structure is ductile enough to sustain the inelastic rotations which are called plastic hinges and when joint undergoes these plastic hinge formations this increases the damage levels in joint which should be avoided for catastrophic failures hence these plastic hinge should relocate to beams or columns further, but if we relocate to a column the entire structure can be on a verge of failing but if the hinge formations are relocated to the beams with proper detailing we can ensure that beam catastrophic failures are delayed, hence we say a strong column and weak beam

Check strong column weak beam,

Moment strength of beam in when plastic region is reached is given by,

$M_b = 1.4 \sum M_{bi}$ here, M_{bi} is sum of moments of resistance of each beams at the joint.

And for column the strength is $\sum M_{ci}$
M_{ci} and M_{bi} we calculated above,

$M_b = 1.4 \sum M_{bi} = M_{u,left}^{As} + M_{u,right}^{Bh}$ =1.4*(87.11+115.82)

M_{ci} = 255+275 …are moments correspond to provided percentage steel and given axial load and interaction chart

Safe design

Column shear design

As per clause 7.5 IS13920 the design shear force must be maximum of these:

$$max\begin{cases} factored\ shear\ force\ from\ structural\ analysis \\ Vu,a = 1.4\frac{M_u^{As}+M_u^{Bh}}{h_{story}} \\ Vu,b = 1.4\frac{M_u^{Ah}+M_u^{Bs}}{h_{story}} \end{cases}$$ considering both sway from left and right

$$max\begin{cases} factored\ shear\ force\ from\ structural\ analysis = 33kN \\ 1.4\frac{87.08+115.82}{3} = 95kN \\ 1.4\frac{115.82+87.08}{3} = 6 = 95kN \end{cases}$$

$for\ \tau_{c,}$

$\frac{A_s}{bD}$ = 0.93 gives, $\tau_{c,} = \frac{0.65N}{mm2}$<3.7 N/mm2 (max $\tau_{c,}$) table 19,20 IS456

τv $= \frac{\text{Vu}}{bD} = 95 * \frac{1000}{450*450} = 0.47 < concrete\ shear\ strength$

provide minimum reinforcement

sv $\leq 0.87 \times 415 \times \frac{157}{0.4*450}$ = 315mm IS456 clause

The pitch of the transverse reinforcement should not be more than the least of the following: (clause 6.3.3 IS13920)

1) Least lateral dimension of compression member =450mm
2) 16 x dia of smallest longitudinal bar = 320 mm
3) 300 mm

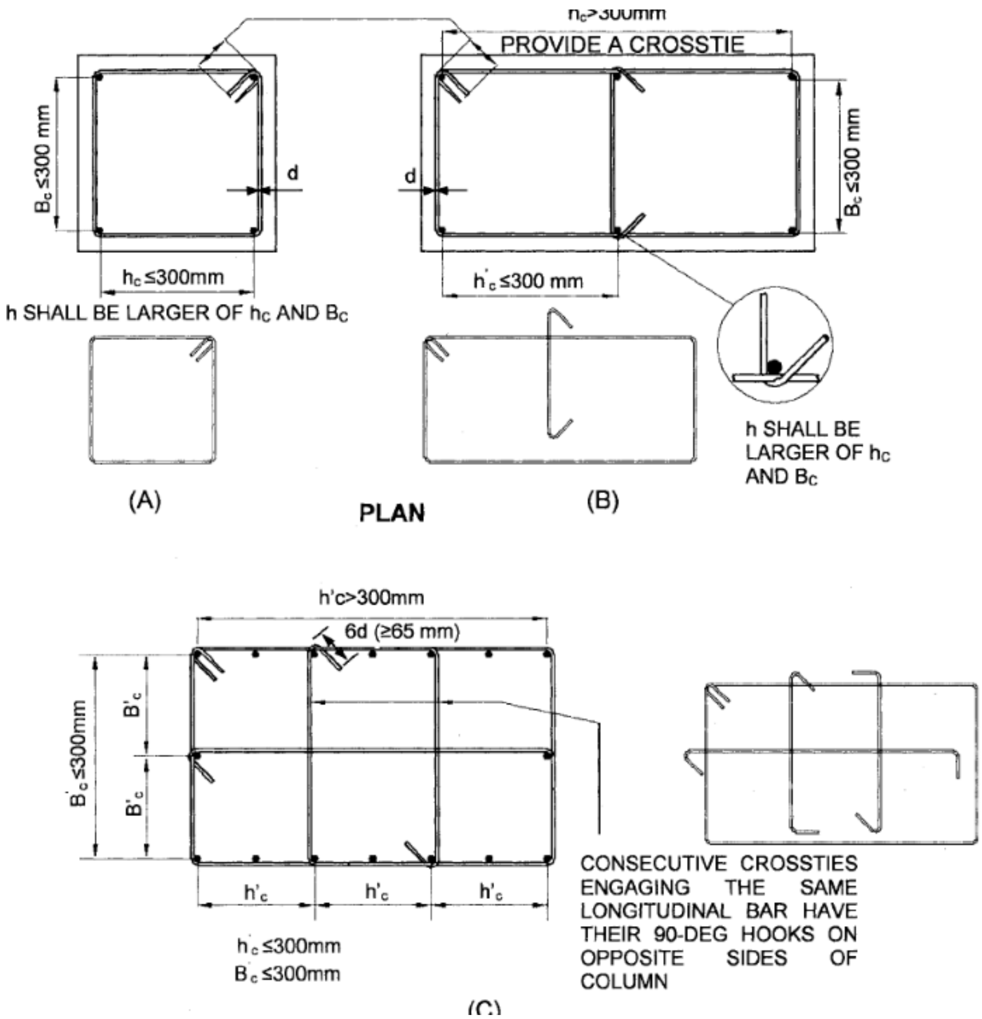
There will be a special confining reinforcement due to flexural yielding is likely in beams during strong earthquake shaking and in columns when the shaking intensity exceeds the expected intensity of earthquake shaking as per (8 clause IS13920)

The length over which the special confining reinforcements is provide is not less than,
1) Larger lateral dimension of a member at a section
2) 1/6 of the clear span of the member
3) 450 mm

Take 450mm

With spacing not more than,

$$\begin{cases} d/4 \\ 6 * dia\ of\ smallest\ longitudal\ reinforecemt \\ 100mm \end{cases} = 100mm\ spacing$$

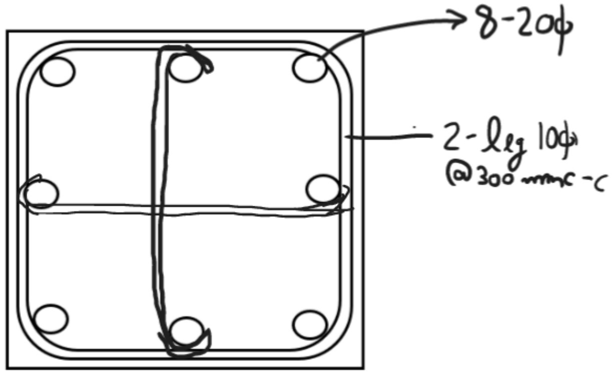


Providing spacing of 100 mm in confining zone Area Ash of cross section of the bar forming links or spiral of at least

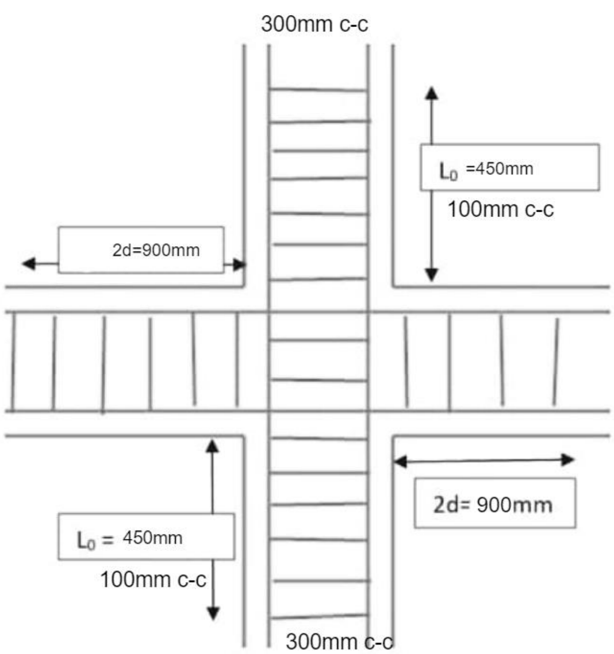
$$A_{sh} = max\begin{cases} \frac{0.18hS_v hf_{ck}}{f_y}(\frac{A_g}{A_k} - 1) \\ 0.05\frac{hS_v hf_{ck}}{f_y} \end{cases}$$

i.e. $max\begin{cases} 0.18 * 100 * 180 * \frac{35}{415}\left(\frac{450*450}{360*360} - 1\right) = 154\ mm^2 \\ 0.05 * 180 * 100 * \frac{35}{415} = 75\ mm^2 \end{cases}$

hence in confining reinforcement, provide 10mm at 100mm c-c



Column section



Joint section

Beam column joint design:

When there is strong ground motion shaking it is not just important to design beams and columns in a ductile way but also the beam-column joint must be properly designed so that the plastic hinge formation are relocated to beams rather than columns and a delayed failure is obtained for the structure as a whole, When structure undergoes right sway,

Factored shear force developed in joint is given by,

$Vu = 1.4\frac{M_u^{As}+M_u^{Bh}}{h_{story}}$=$1.4\frac{87.08+115.82}{3} = 95kN$

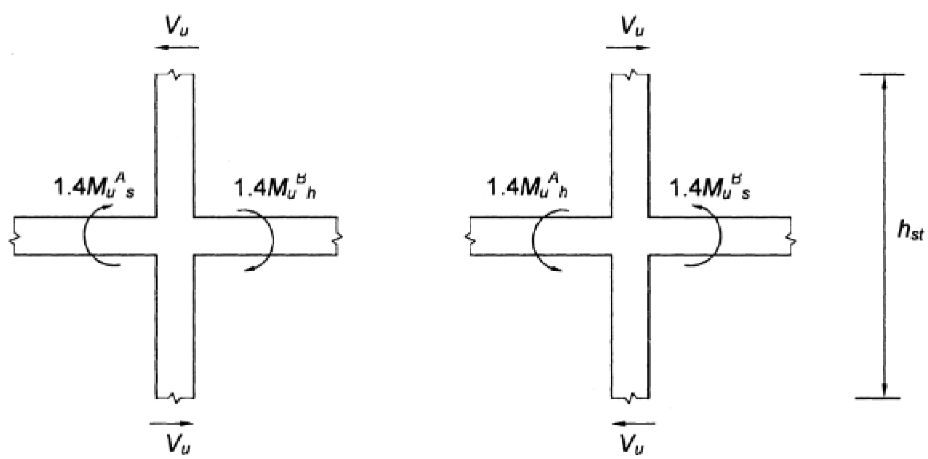
Force developed in left joint beam,

$F_{left} = 1.25A_{st,bottom}f_y = 1.25 * 3 * \frac{3.1415}{4} * 16^2 * 415 = 312kN$

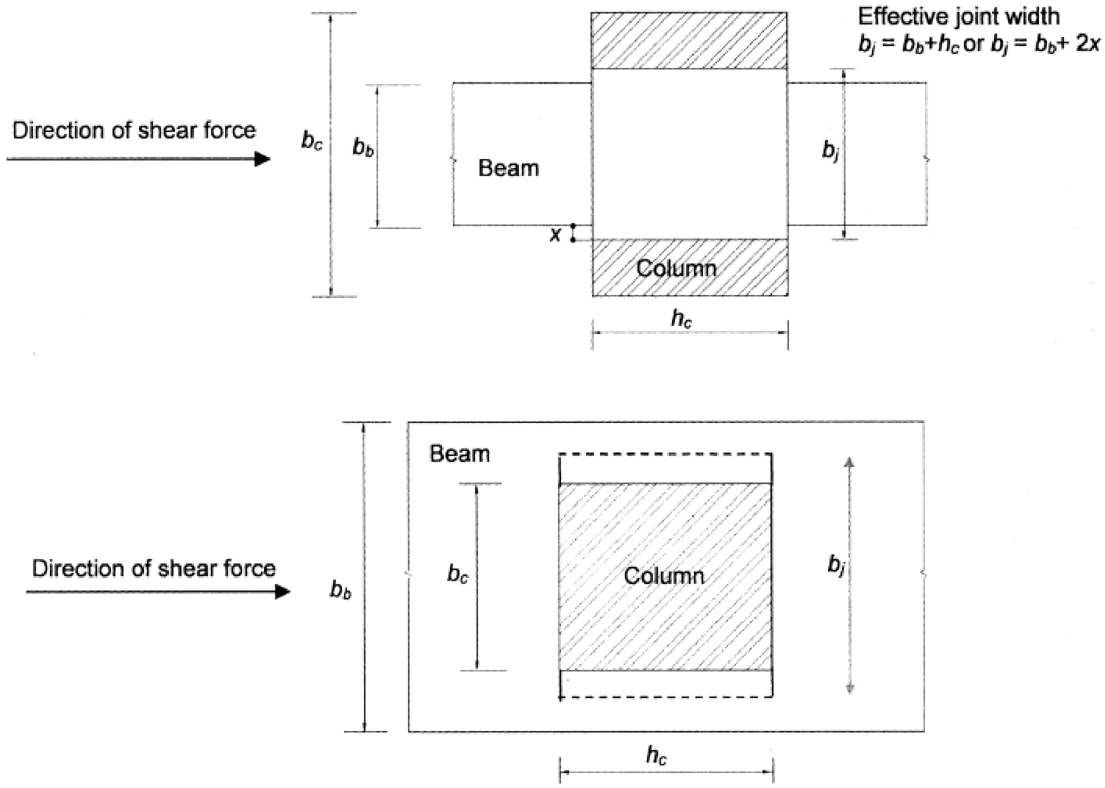
$F_{right} = 1.25A_{st,top}f_y = 1.25 * 4 * \frac{3.1415}{4} * 16^2 * 415 = 417kN$

Hence joint shear force will be,

$F_{left} + F_{right} - Vu$=634kN



For left sway, $Vu,b = 1.4\frac{M_u^{Ah}+M_u^{Bs}}{h_{story}} = \frac{1.4(115.82+87.08)}{3} = 95kN$



Plan

b_j = min{b_c , b + 0.5D}= min(450,300+0.5*450)=450mm
And D_j = 450mm

Shear strength of concrete in a joint: (Cl. 9.1.1 IS 13920:2016)

τ_{jc} = 1.5$\sqrt{fck}$ = 8.87MPa beams on all 4 sides

V_{jc} = $A_{cj}\tau_{jc}$ =1796kN>634kNjoint shear obtained hence we do not need to revise joint

Shear wall

Shear wall	Design values
Moment, M_3 (kNm)	6338
Axial force, P_u (kN)	2770
Shear force, V_u (kN)	1062.3

$length\ of\ wall\ l_w = 5m$
$heights\ of\ wall\ h_w = 15m$
$thickness\ of\ wall\ t_w = 250mm$

$length\ of\ wall\ l_w = 4m$
$\frac{h_w}{l_w} = \frac{33}{4} > 2\ hence\ slender\ shear\ wall\ design$ as per IS13920 (Cl. 10.1. 4 IS 13920:2016)

Extreme fiber stresses in shear wall = $\frac{P}{Lwt_w} + \frac{6M}{t_w L_w^2} = 8.3 > 0.2f_{ck} = 7$hence boundary element required

Boundary element size same as column 450x450 take 0.8% reinforcement minimum reinforcement in boundary element
Ast = 1620mm^2

A_{be} = 450x450 = 202500mm^2

A_{web} = $(L_w - b)t_w$ = 1137500mm^2
A_{wall} = 1340000mm^2

The standard practice of shear wall design neglects the interaction of shear web and boundary elements the flexural strength of wall is estimated by summing up the contributions from boundary element as a force couple and flexural strengrh of web at proportionate axial load
It is assumed that stress is uniformly distributed in shear wall hence to find force in boundary element we can distribute on the basis of areas of the boundary element and wall
$F_{be} = \frac{Abe}{\text{Awall}} = \frac{202500}{1340000} * 2770 = 419kN$

Tension and compression capacity will be given by,

It is assumed that the boundary elements carry only the axial load and no moments hence we can find compression and tension capacity by,

$C_{be} = 0.446f_{ck}A_c + 0.75f_yA_{st}$ for purely axial load compression capacity of column is given by
= 0.446*35*(450*450-1620)+0.75*415*1620
= 3640kN

And tension capacity of boundary element,

T_{be} = 0.87*$f_y A_{st}$ = 585kN
Hence, Tension and compression load to contribute to moment couple

C = C_{be} - F_{be}
= 3221kN

T = T_{be}+F_{be}
= 1004kN

The minimum of these force will cause a force couple which will be governed by tension as tension is lesser than compressive capacity here,
Hence, force couple contributed by a pair of boundary element is given by,

M_{be} = (L-d)*min(C,T)
= T*(5-.45)
= 4568.2kNm

Design the web,

We calculate the rest moment that will be resisted by the web

M_{web} = M - M_{be} =6338-4568.2 = 1770kNm

And P_{web} = P -2F_{be} = 2770-2*419 = 1932kN

Minimum reinforcement in web is 0.25% assume 0.25% provided now check if moment of resistance if able to counter the web moment acting
Moment of resistance is calculated as per IS13920 Annex A

Ast = 2843.75mm2

$$\emptyset = \frac{0.87 f_y \rho}{f_{ck}} = 0.0257$$

$$\gamma = \frac{P_u}{f_{ck} t_w L_w} = 0.05$$

$$\beta = \frac{0.002 + \frac{0.87 f_y}{E_s}}{0.0035} = 1.09$$

$$\frac{x_u}{L_w} = \frac{\emptyset + \gamma}{2\emptyset + 0.36} = 0.184$$

$$\frac{x_u{*}}{L_w} = \frac{0.0035}{0.0055 + 0.87 f_y / E_s} = 0.456$$

Here, $\frac{x_u}{L_w} < \frac{x_u{*}}{L_w}$

Hence case a. we calculate Mu by

$$\frac{M_u}{f_{ck}t_w{L_w}^2} = \emptyset\left(\left(1+\frac{\gamma}{\emptyset}\right)\left(\frac{1}{2}-0.416\frac{x_u}{L_w}\right)-\left(\frac{x_u}{L_w}\right)^2\left(0.168+\frac{\beta^2}{3}\right)\right) = 0.03156$$

5717kNm>Mweb 1770kNm hence it will be able to resist the moment as well as the axial force safe design

Lets provide 8 bars of 16 diameter in two layers as thickness more than 200mm cl. 10.1.7 IS13920

Shear design

Nominal shear stress $\tau_v = \frac{V_u}{t_w d_w} = 1062.3 * \frac{1000}{250*0.8*4550} = 1.17MPa$

τc = 0.37 MPa table 19,20 IS456 for M35 concrete grade

We see $\tau_{v>}$ τ_c design for shear reinforcement

Vus = 1062.3*1000-0.37*4550*250 = 641.425kN

Assume 1 legged 10 dia stirrup calculate its spacing as per IS456

$$\mathrm{sv} \le 0.87 \times 415 \times \frac{\frac{3.14}{4}*10^2*0.8*4550}{323*1000} = 150\mathrm{mm}$$

provide 1leg 10 dia @150mm c-c

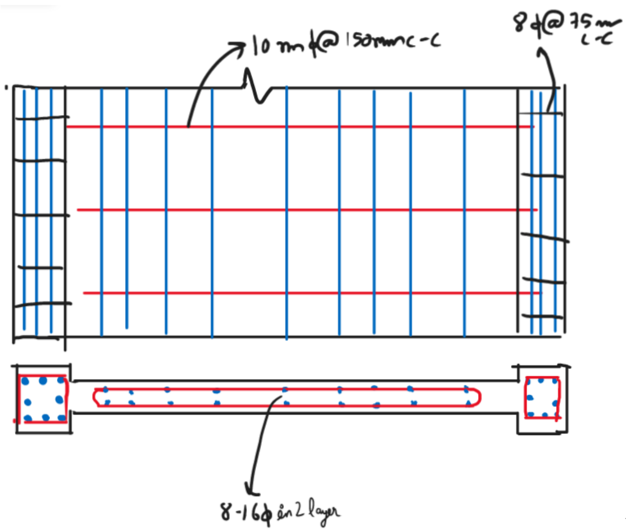
for boundary element, special confining reinforcement is provided with area as per 10.4.1 IS13920

$$\mathrm{Ash} = 0.05\ \mathrm{Sv}\ \mathrm{h}\frac{\mathrm{fck}}{f_y} = 75\mathrm{mm2}$$

with

Sv spacing should be less than:
1. 250/3 = 75 mm
2. 6 x 16 = 96 mm
3. 100 mm

Provide leg 10 dia at 75mm spacing 75mm



Plan and sectional view of shear wall

Length of web 4.55m, boundary element 450mm

Raft foundation:

Raft foundation are used in tall structures where the load coming from building is very high, and soil has less bearing capacity, Raft foundation design in India is governed by the Indian Standard Code of Practice IS 2950:1981 (Part I)

The code specifies the steps of designing as,

- Soil investigation: this is done to determine the bearing capacity and settlement characteristics of the soil

- Load determination: The loads on the raft foundation must be determined based on the structural design of the superstructure, and the loads must be distributed over the foundation area.

- Thickness of raft: the mininmum thickness of raft foundation is 200mm for clays and 250mm for sands, rankine's formula can be used to determine the thickness of raft foundation.

  D =$\frac{P}{\gamma}\left(\frac{1-sin\emptyset}{1+-sin\emptyset}\right)^2$ p is bearing capacity of soil from soil investigation, γ is unit weight of soil and $\emptyset\ is$ angle or repose of soil

  For our problem we can consider the depth of raft foundation to be 1000mm

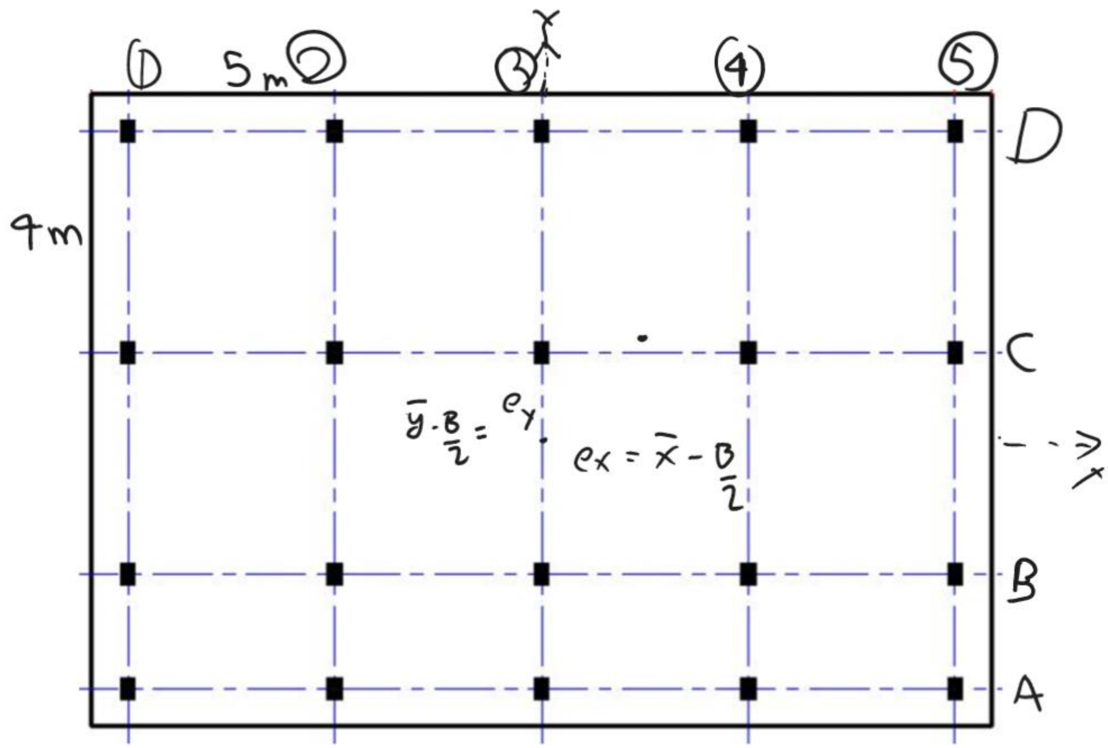
- The reinforcements in raft foundation shall be designed for the maximum shear forces and maximum bending moment acting on it as well as punching shear failure

To do analysis for loads acting on the raft foundation we follow these steps:

- As the foundation is resting on soil it will be acted upon a pressure the pressure distribution will be given by,

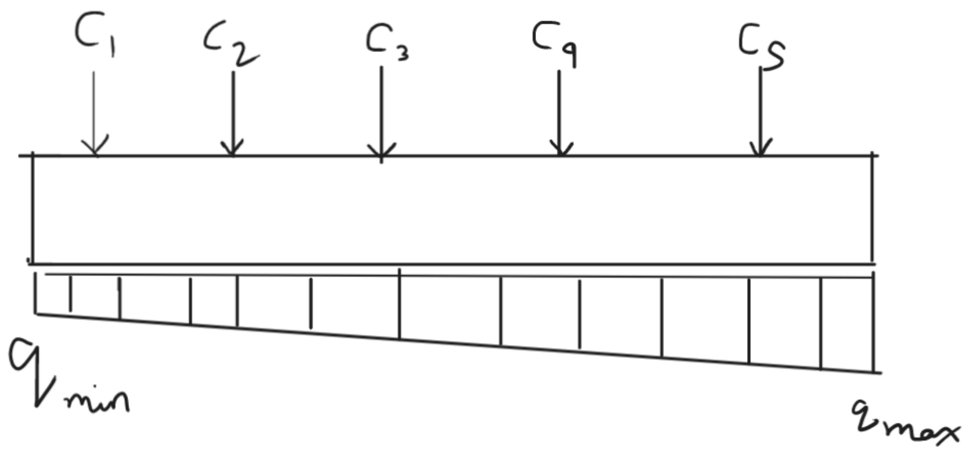
$$q_{max,min} = \frac{Q}{A} \pm \frac{M_y x}{I_y} \pm \frac{M_x y}{I_x}$$

where, A is area of raft foundation,
Q is total vertical load in raft from columns
Ix, Iy moment of inertial in x and y axis
X and y are distances from origin in x and y direction
Mx and My can be calculated based on the eccentricities in x and y direction of the load Mx = Pex,My=Pey,



Calculate q at each point
Check if q is less than Qallowable for the soil

- Section where Q is max will be the critical section then take a strip at that section and perform structural analysis to find shear forces and bending moments at that section

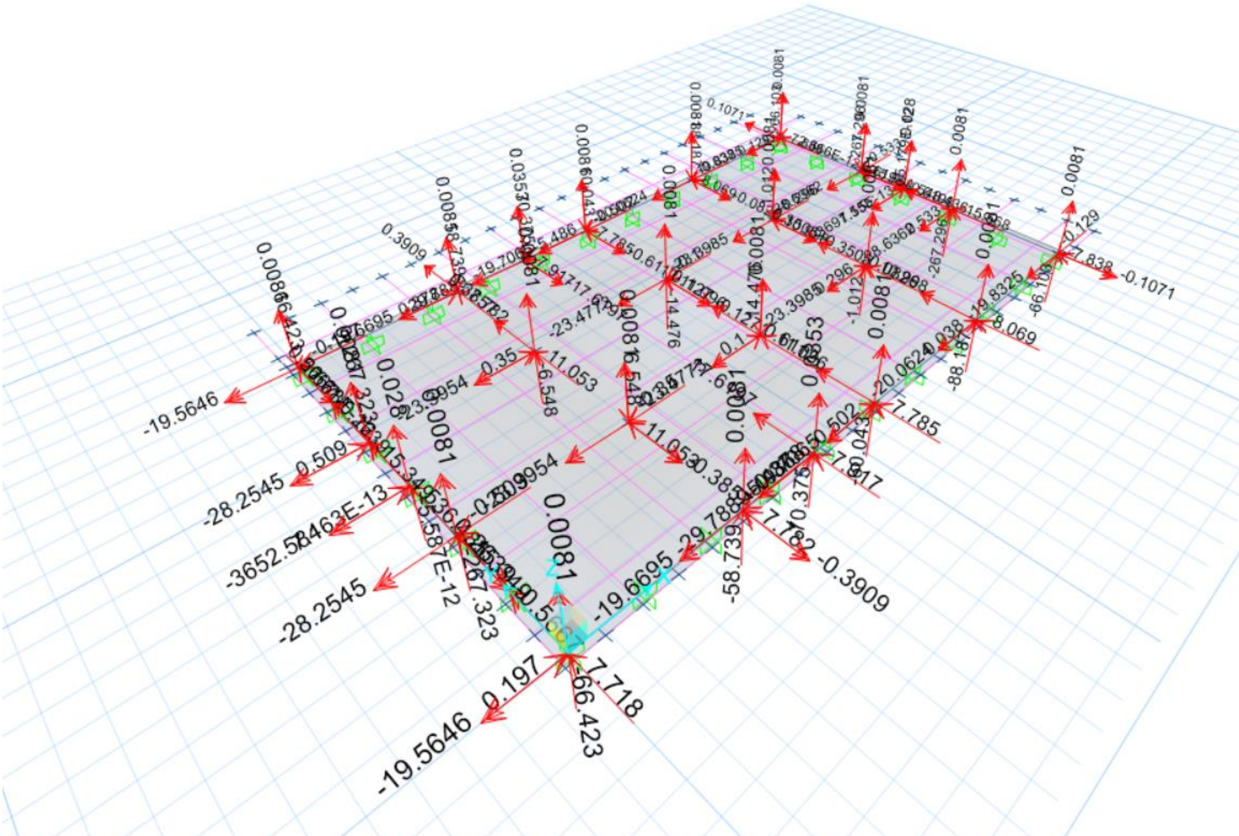


- Use maximum of these to design the section

For the structure after the analysis into sap it is put into analysis into SAFE structural analysis and design of foundation for the same load combination where it will give us these diagrams and on basis of this the reinforcements are calculated

Define material property as per given data.
Unit weight of soil γs = 20 $kN/m3$
Poisson's ratio of soil μ = 0.2
Young's modulus of concrete Ec = 5000$\sqrt{fck}$ = 29580.3989$N/mm2$

Depth = 1000mm and clear cover = 50mm (as per CL 26.4.2.2 of IS 456: 2000, Minimum cover for footing is 50 mm)



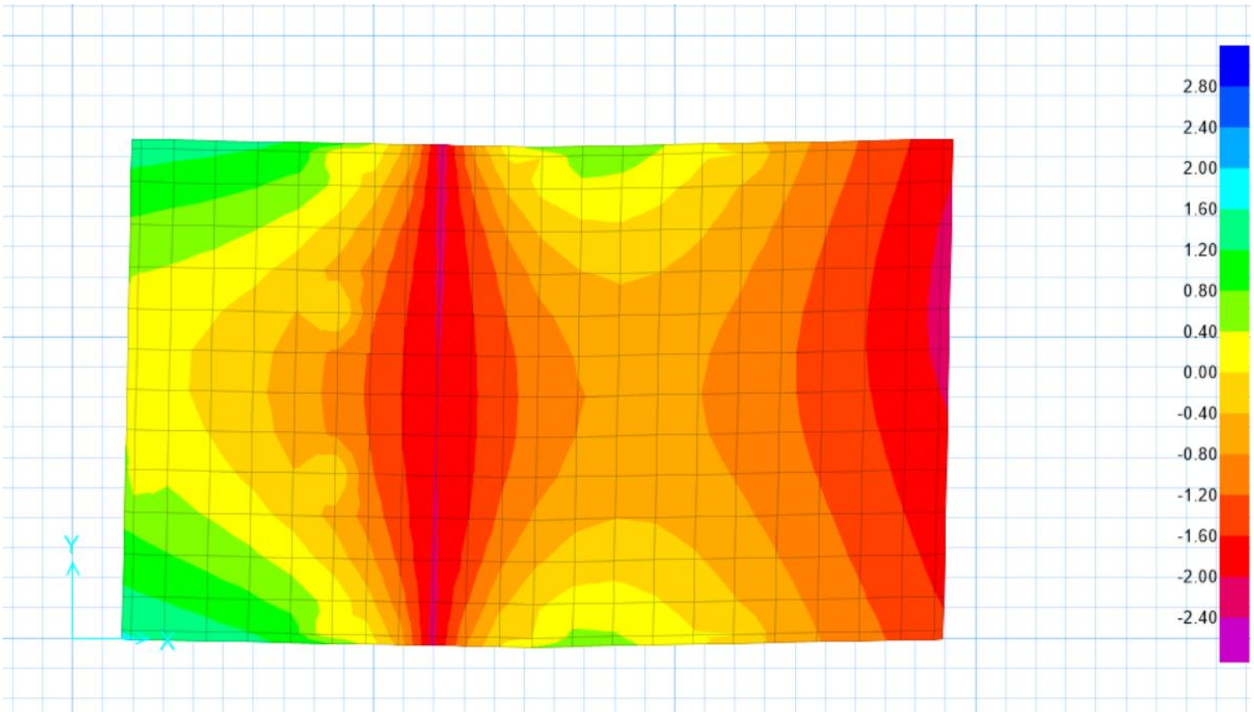
Support reactions after importing from sap2000 into safe foundation

We apply subgrade reaction force which will be determined by soil investigation report
The subgrade formula is given by as per APPENDIX B Clause 3.1(f) IS : 2950 (Part I) – 1981)

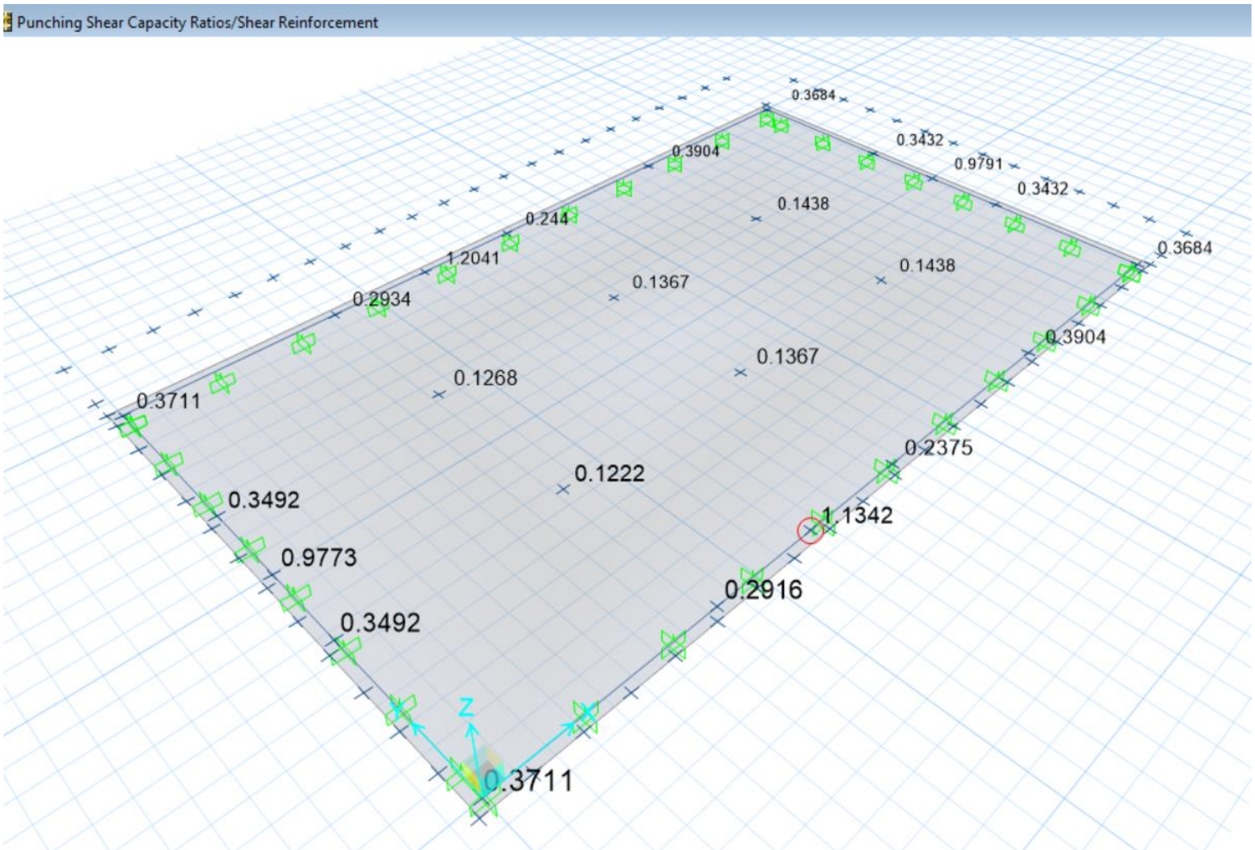
$$Ks = 0.65 \frac{\mathrm{Es}}{1-\mu 2} \left[12 \sqrt{\frac{\mathrm{EsB}^4}{EI}} \right] * \frac{1}{1000\mathrm{B}}$$

Assuming soil subgrade modulus of 12000 KN/ $m3$ applied by soil on raft

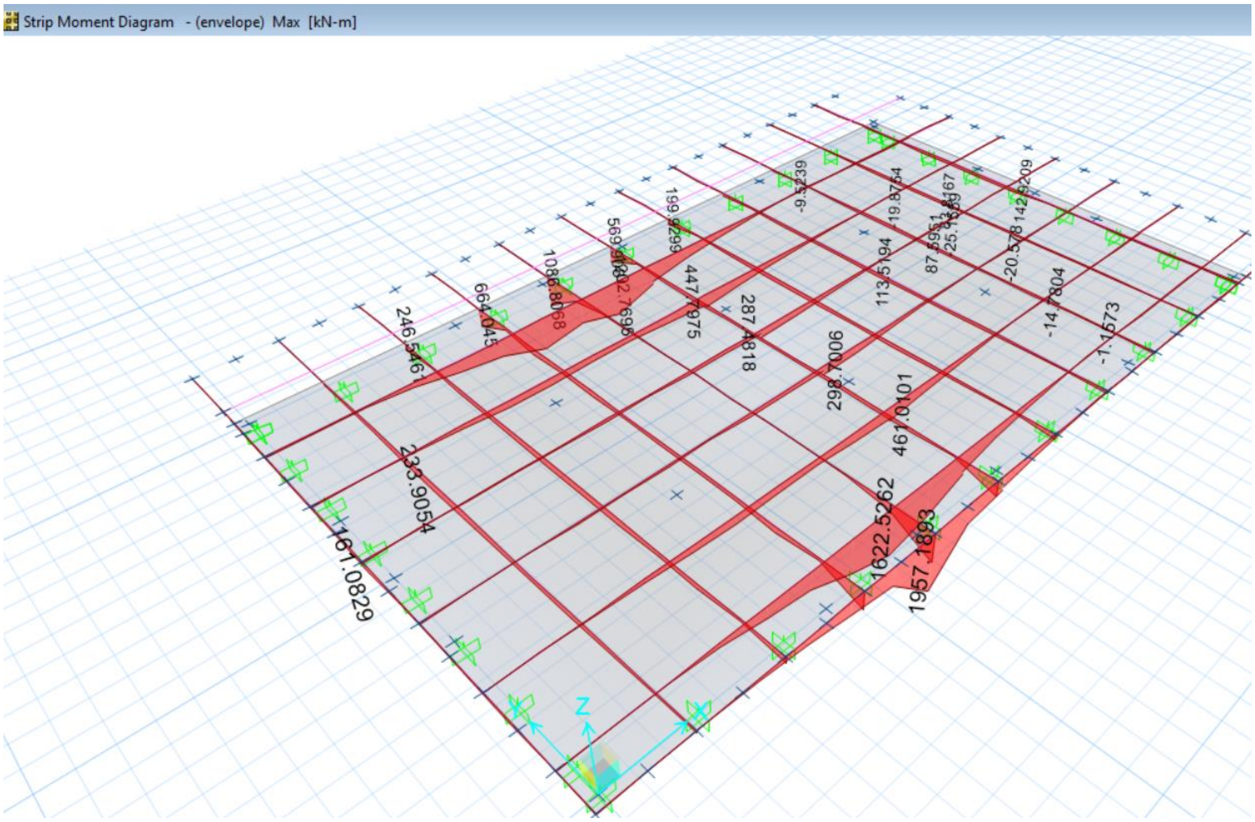
The results of the analysis are as follows:



Max settlement = 2.8mm<50mm (limit) hence safe
Min settlement = -9.2mm safe



Punching shear<1 hence safe



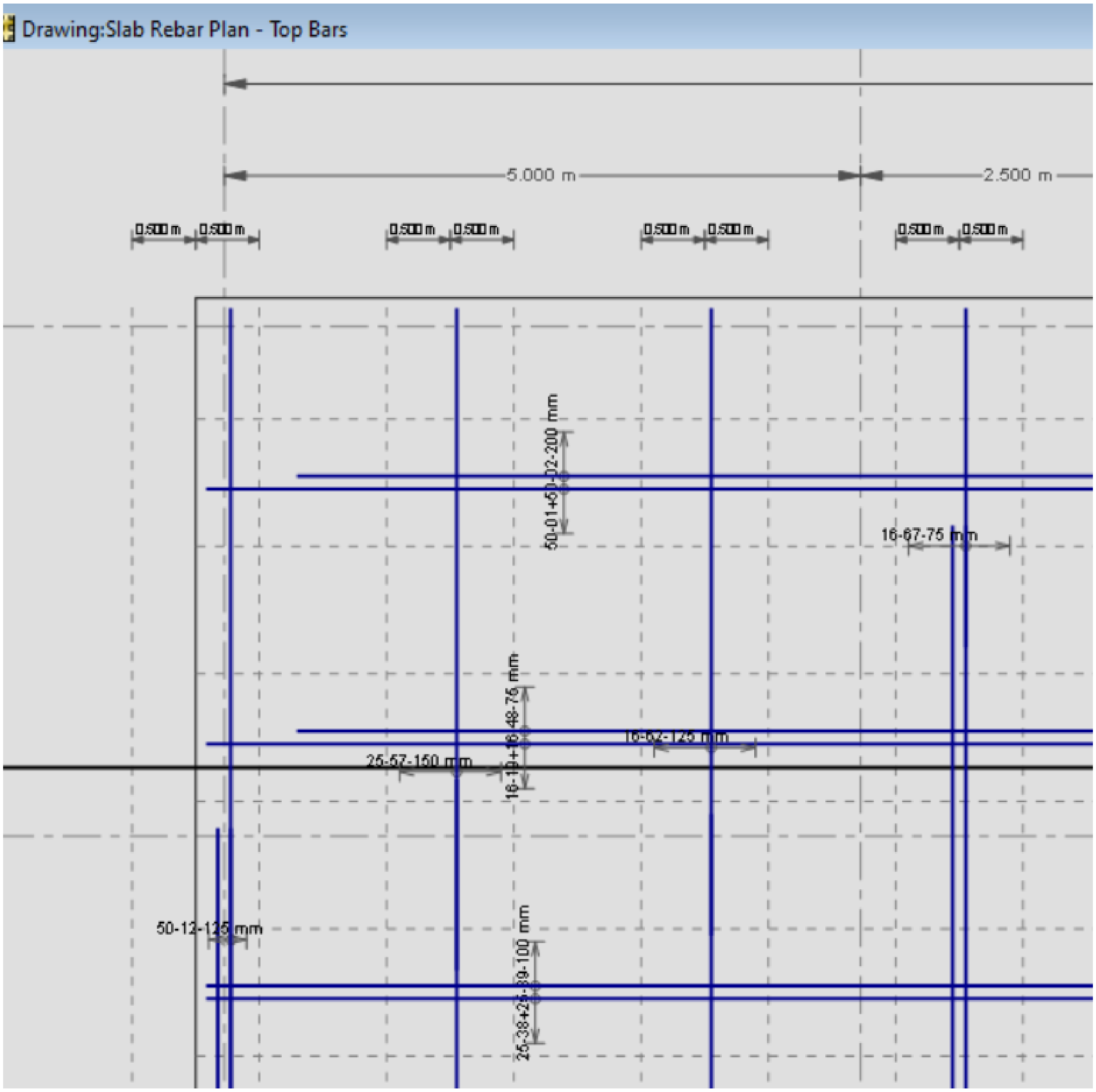
Bending moment diagram from safe

Design for bending moment:

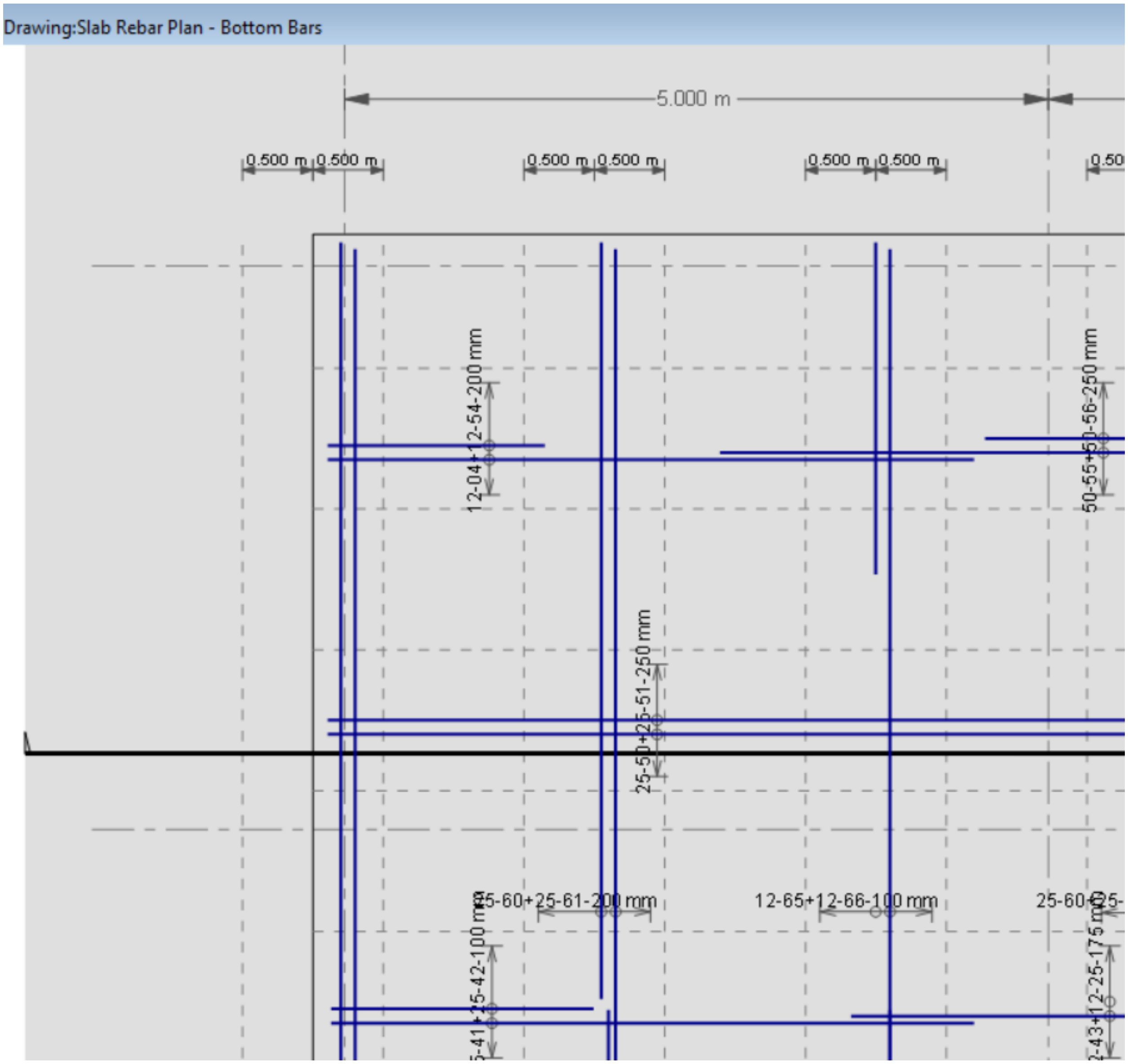
Max bending moment = 1957kNm
Effective depth 900mm
Width b = 1000mm for 1 m width

$$A_{st} = \frac{0.5fy}{fck}\left(1 - \frac{4.6Mu}{fckbD^2}\right)^{0.5} bd$$

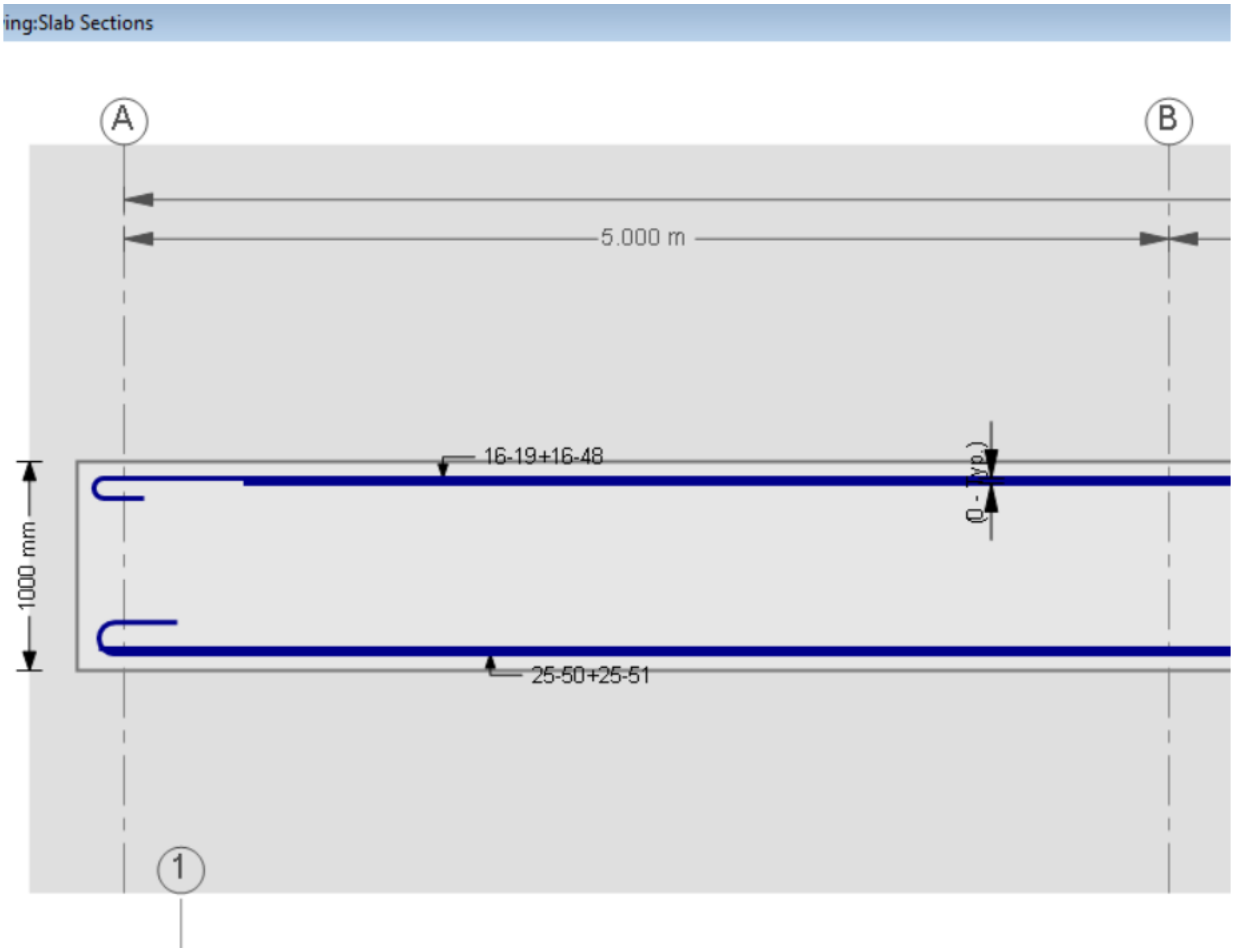
$$A_{st} = \frac{0.5*35}{415}\left(1 - \left(1 - \frac{4.6*1957*10^6}{35*1000*900^2}\right)^{0.5}\right)1000 * 900 = 16865mm^2$$



Top rebar detailing of design


Drawing:Slab Rebar Plan - Bottom Bars
5.000 m
0.500 m
0.500 m
12-04+12-54-200 mm
50-55+50-56-250 mm
25-50+25-51-250 mm
25-60+25-61-200 mm
12-65+12-66-100 mm

Bottom bars



Sectional view