DESIGN PROJECT REPORT

on

DESIGN OF A 5-STORY MRF BUILDING WITH SHEAR WALL

for the course

Earthquake analysis and design of structures

CE 629A

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Sincerely,

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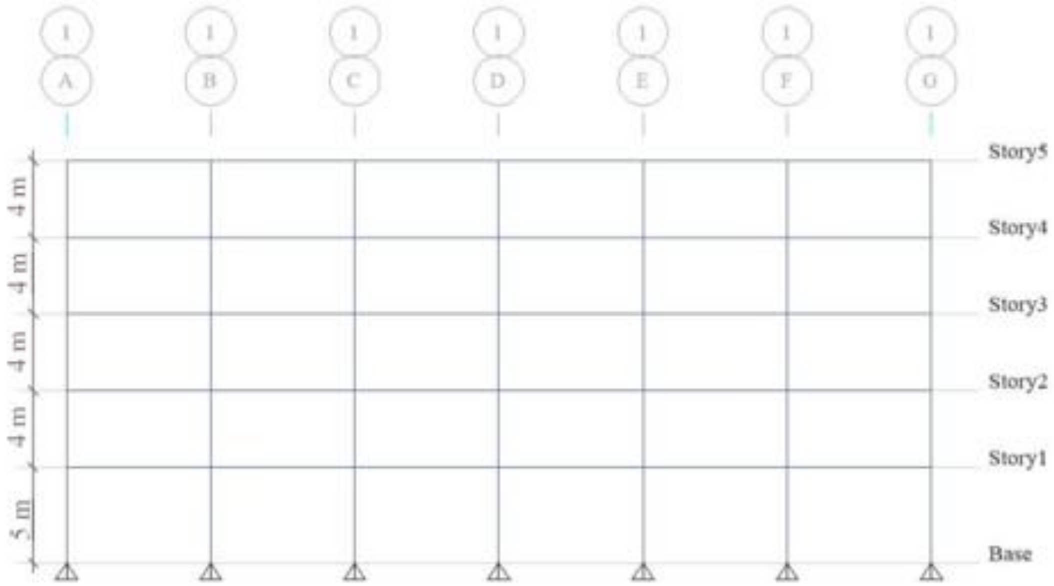
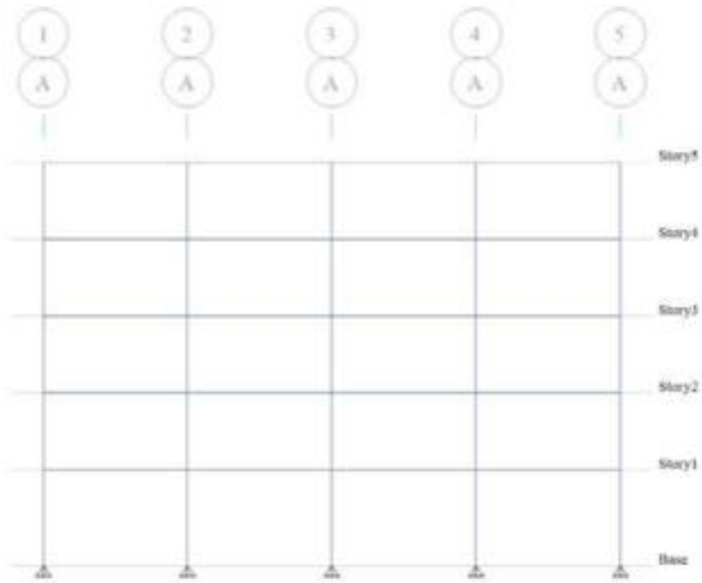
Contents

1. Problem Statement

A five-storey office building needs to be designed for seismic zone V on a medium stiff soil site. Atypical floor plan and two side elevations of the buildings are shown below. The floor plan assumes an RC frame structure and therefore the columns are shown with rectangles. The lateral load resisting system chosen for the purpose is Reinforced Concrete Ductile Shear Wall with Reinforced Concrete Special Moment Resisting Frame (Dual System).



E-W Elevation



N-S Elevation

Figure 1: Elevation of the Office Building

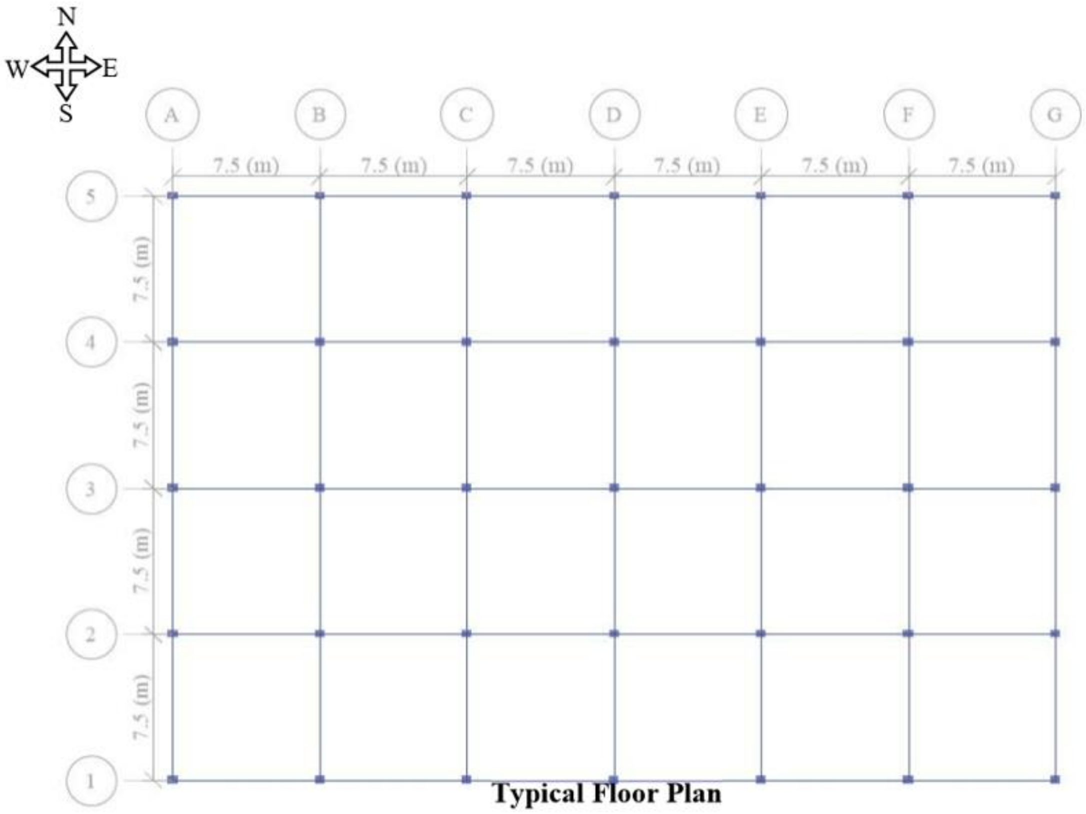


Figure 2 Plan of the Office Building

2. Design Data

Thickness of walls	: 230 mm
Grade of concrete	: M35
Grade of reinforcing steel	: Fe415
Self-weight of concrete	: 25 kN/m^3
Self-weight of masonry (including plaster)	: 20 kN/m^3
Floor finish	: 1 kN/m^2
Slab thickness	: 150 mm
Building type	: Office building
Seismic Zone	: V
Imposed load on floor	: 4 kN/m^2

Soil type	: Medium stiff soil
Imposed load on roof (assumed as per IS 875 (Part 2): 1987)	: 1.5 kN/m^2
Height of Building	:21m
Thickness of Shear Wall	:200mm

3. General Assumptions:

Following Assumptions were taken:

(i) The floor diaphragms at floors are assumed to be rigid as per Cl. 7.6.4 of IS 1893 (Part 1): 2016.

(ii) Centre-line dimensions are followed for analysis and design.

4. Force Calculations

Calculation of minimum design lateral force $(VB)_{min}$ (IS 1893 (Part 1): 2016, Cl. 7.2.2):

For Seismic Zone V, Zone Factor = 0.36. (IS 1893 (Part 1): 2016, Table 2)

Importance factor (I) = 1.2 (IS 1893 (Part 1): 2016, Table 8)

Response reduction factor (R) for Dual System with Ductile RC Structural Wall with RC Special Moment Resisting Frame = 5 (IS 1893 (Part 1): 2016, Table 9)

Design horizontal seismic coefficient (A_h) = $\frac{\left(\frac{Z}{2}\right)\left(\frac{S_a}{g}\right)}{\left(\frac{R}{I}\right)}$ ([IS 1893 (Part 1): 2016, Cl. 6.4.2).

Minimum Design Earthquake Horizontal Lateral Force for Building:$\rho = 2.4\%$

<u>In X direction</u>:

The approximate fundamental translational natural period of oscillation is given by (IS 1893 (Part 1): 2016, Cl. 7.6.2):

$$T_a = \frac{0.075 * h^{0.75}}{\sqrt{A_w}} \geq \frac{0.09 * h}{\sqrt{d}}$$

$$A_w = \sum_{i=1}^{N_w} A_{wi} * \left[0.2 + \left(\frac{L_{wi}}{h}\right)\right]^2 = 1.862m^2$$

Where, $L_{wi} = 7.5m$ and $A_{wi} = 7.5 * 0.2 = 1.5m^2 \; and \; d = 45m$

Therefore, $T_a = 0.539s \geq 0.28s$

Design acceleration coefficient for medium stiff soils $\frac{S_a}{g} = 2.5$ (IS 1893 (Part 1): 2016, Cl. 6.4.2)

<u>In Y direction:</u>

The approximate fundamental translational natural period of oscillation is given by (IS 1893 (Part 1): 2016, Cl. 7.6.2):

$$T_a = \frac{0.09 * h}{\sqrt{d}} = 0.34 sec$$

Where d = 30m.

Design acceleration coefficient for medium stiff soils $\frac{S_a}{g} = 2.5$ (IS 1893 (Part 1): 2016, Cl. 6.4.2).

Therefore, in both X and Y direction:

Design horizontal seismic coefficient (A_h) = $\frac{\left(\frac{Z}{2}\right)\left(\frac{S_a}{g}\right)}{\left(\frac{R}{I}\right)} = \frac{\left(\frac{0.36}{2}\right)(2.5)}{\left(\frac{5}{1.2}\right)} = 0.108$ (IS 1893 (Part 1): 2016, Cl. 6.4.2).

$A_h = \max(A_h, \rho) = 0.108$

Seismic Weight Calculations:

Brief calculations of seismic weight are shown in below Tables 1-3.

Table 1 Calculation of seismic weight at roof level

Weight due to	**No of items**	**Length (m)**	**Width (m)**	**Depth (m)**	**Unit Weight (kN/m^3)**	**Total Weight (kN)**
Beams Moment Resisting Frame	28	7.5	0.4	0.6	25	1260
Infill Walls	54	7.5	0.23	1.75	20	3260.25
Columns Moment Resisting Frame	35	2	0.5	0.5	25	437.5
Slab	24	7.5	7.5	0.15	25	5062.5
Shear wall	4	7.5	0.2	2	25	300
Beams Gravity	26	7.5	0.3	0.5	25	731.25
					Total Weight	11051.5 kN

Table 2 Calculation of seismic weight at floor level 2, 3 and 4

Weight due to	**No of items**	**Length (m)**	**Width (m)**	**Depth (m)**	**Unit Weight (kN/m^3)**	**Total Weight (kN)**
Beams Moment Resisting Frame	28	7.5	0.4	0.6	25	1260
Infill Walls	54	7.5	0.23	3.5	20	6520.5
Columns Moment Resisting Frame	35	4	0.5	0.5	25	875
Slab	24	7.5	7.5	0.15	25	5062.5
Shear wall	4	7.5	0.2	4	25	600
Beams Gravity	26	7.5	0.3	0.5	25	731.25
					Total Weight	15049.25kN

Table 3 Calculation of Seismic Weight at Ground Floor Level

Weight due to	**No of items**	**Length (m)**	**Width (m)**	**Depth (m)**	**Unit Weight (kN/m^3)**	**Total Weight (kN)**
Beams Moment Resisting Frame	28	7.5	0.4	0.6	25	1260
Infill Walls	54	7.5	0.23	1.75	20	3260.25
Columns Moment Resisting Frame	35	7	0.5	0.5	25	1531.25
Slab	24	7.5	7.5	0.15	25	5062.5
Shear wall	4	7.5	0.2	7	25	1050
Beams Gravity	26	7.5	0.3	0.5	25	731.25
					Total Weight	12895.5kN

Seismic Weight due to Floor Finish = 30*45*1*4 = 5400kN.

Seismic Weight due to Live Load = 30*45*4*4*0.5 = 10800kN.

Total Seismic Weight W = 5400+10800+12895.5+3*15049.25+11051.5=85294.5kN.

Therefore, Design lateral force:

$\left(\overline{V_B}\right) = A_h * W = 0.108 * 85294.5 = 9210.43kN$ (IS 1893 (Part 1): 2016, Cl. 7.2.1).

$(\overline{\mathrm{V_B}}) = \overline{V}_{\mathrm{BX}} = \overline{V}_{\mathrm{BY}} = 9210.43$ kN.

<u>Base shear calculated using SAP2000:</u>

V_{BX} = 6938.815 kN; V_{BY} = 3148.149 kN;

Since the base shear estimated using response spectrum method ($\mathrm{V_B}$) is less than the base shear estimated using equivalent static method. So, the multiplying correction factor for force response quantities is given by as per Cl. 7.7.3 of IS 1893 (Part 1) : 2016:

Base shear correction: $\frac{\overline{V}_{\mathrm{BX}}}{V_{BX}} = \frac{9210.43}{6938.815} = 1.327.$ $\frac{\overline{V}_{\mathrm{BY}}}{V_{BY}} = \frac{9210.43}{3148.149} = 2.925$

5. Development of a 3D computational model of structure using SAP2000 and performing analysis:

A 3D model is prepared as per the dimensions and material properties considered for the problem. Along X-direction shear wall was used as lateral load resisting system and along Y-direction special moment resisting frames were used as lateral load resisting system.

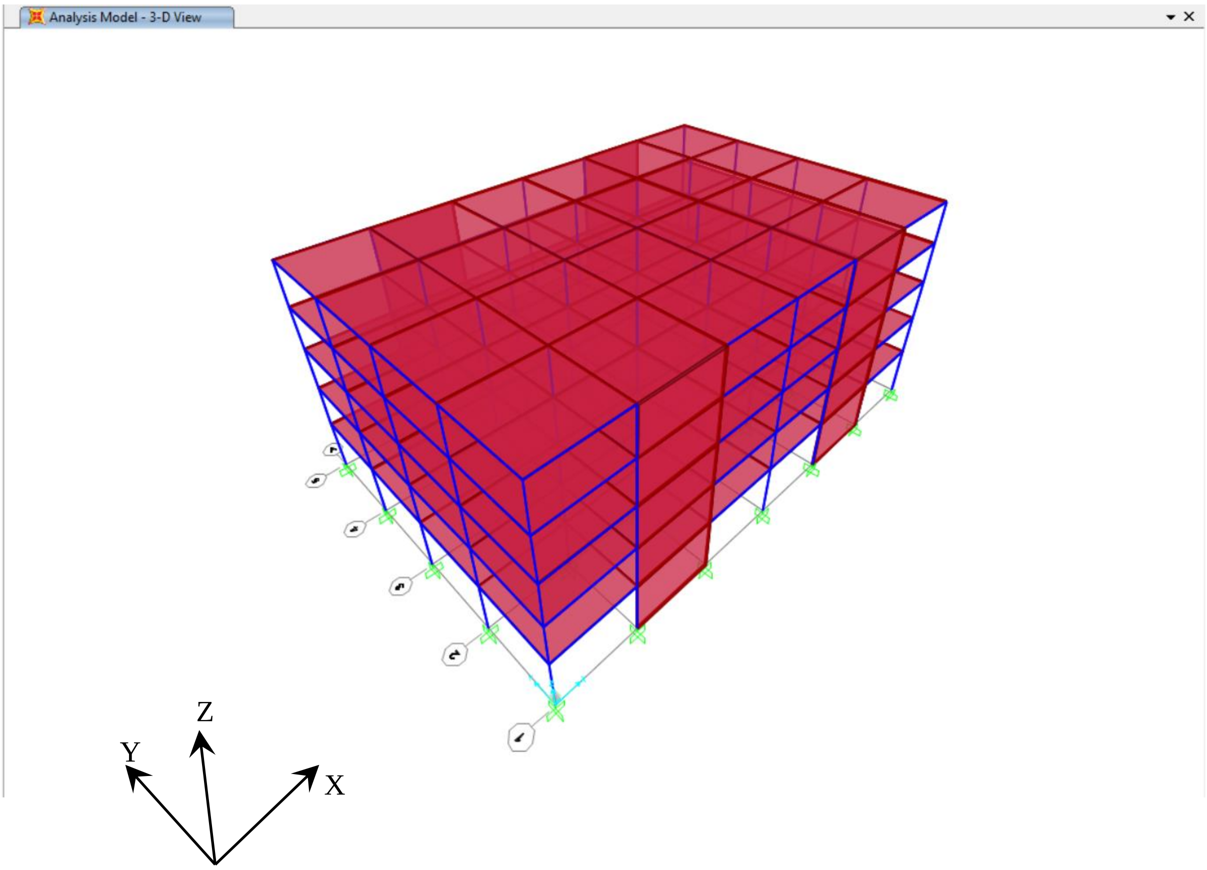


Figure 3 3D computational model of structure developed using SAP2000

All the frames along X-direction are considered as gravity frames and along Y-direction, all frames are considered as special moment resisting frames. Shear wall was modelled as thin shell element. Response spectrum load case is defined as per IS 1893 (Part 1) : 2016. The load combinations defined in SAP2000 are shown in below Table 4.

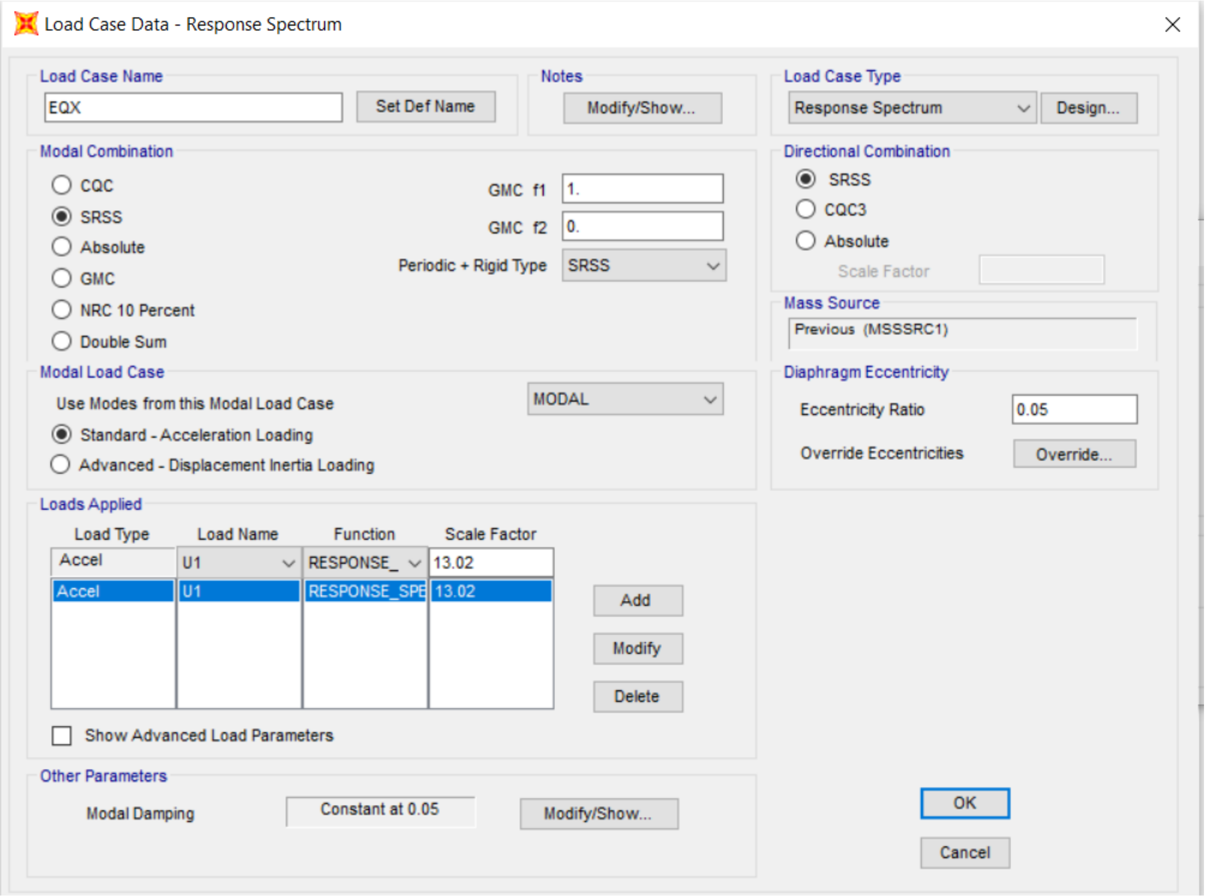


Figure 4 Response spectrum load case defined along X-direction

Table 4 Load combinations considered in SAP2000 analysis

S. No.	Load combination
1	1.5(DL+LL)
2	1.2(DL+LL+EQX)
3	1.2(DL+LL+EQY)
4	1.5(DL+EQX)
5	1.5(DL+EQY)
6	0.9DL+1.5EQX
7	0.9DL+1.5EQY

6. Design of beam

Design calculations for the beam AB of frame A as shown in Figure 5 are shown in this section.

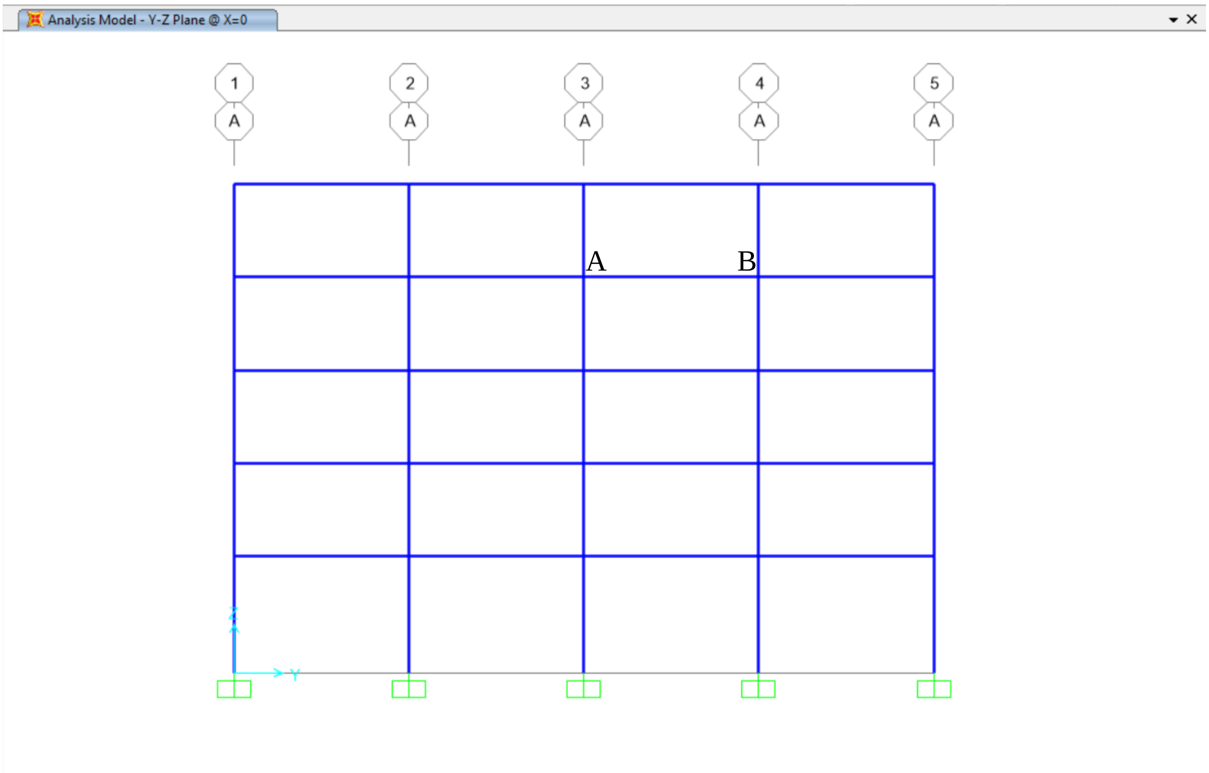


Figure 5 Beam considered for designing

6.1 Member forces

Member forces for the beam AB at different locations and load combinations are shown in below Table5.

Table 5 Forces for the beam AB at different locations and load combinations

S No	**Load Combinations**	**Left end**		**Centre**		**Right end**	
		Shear (kN)	Moment (kN-m)	Shear (kN)	Moment (kN-m)	Shear (kN)	Moment (kN-m)
1	1.5DL+1.5LL	-177.1515	-237.88635	-0.1155	**125.5548**	176.922	-237.01995
2	1.2(DL+LL*+EQY)	-69.4152	81.1302	72.2136	118.524	213.8436	81.24192
3	1.2(DL+LL*-EQY)	-214.0272	-461.74836	-72.3984	82.36368	69.2316	-460.47384
4	1.5(DL+EQY)	-44.6415	167.1408	90.207	109.32615	**225.057**	167.7291
5	1.5(DL-EQY)	**-225.4065**	**-511.4574**	**-90.558**	64.12575	44.292	**-509.4156**
6	0.9DL+1.5 EQY	9.3681	**236.00412**	90.2772	74.63577	171.1872	**236.0664**
7	0.9DL-1.5 EQY	-171.3969	-442.59408	-90.4878	29.43537	-9.5778	-441.0783

* In design, decimals have been rounded up.

6.2 Various Checks

6.2.1 Check for Axial Stress

Factored Axial Force = 0 kN

Factored Axial Stress = 0 MPa < 0.10*fck

Hence, design as a flexural member. (Clause 6.1; IS 13920:2016)

6.2.2 Check for Member Size

Width of beam b = 400mm > 200 mm. Hence, OK (Clause 6.1.2; IS 13920:2016)

Depth of beam D = 600mm.

b/D = 400/600 = 0.67 > 0.3, Hence OK (Clause 6.1.1; IS 13920:2016)

Span L = 7500mm

Clear Span = 7500-500 = 7000;

L/D = 7000/600 = 11.67 > 4; Hence Ok (Clause 6.1.3; IS 13920:2016)

6.2.3 Check for Limiting Longitudinal Reinforcement

Effective depth for moderate exposure with 20 mm diameter bars in two layers and stirrup 8 mm d = 600 – 30 – 8 - 20 – (20/2) = 532 mm.

Minimum Reinforcement:

$$\rho_{\text{min}} = 0.24\frac{\sqrt{f_{\text{ck}}}}{f_y} = 0.24\frac{\sqrt{35}}{415} = 0.00342 = 0.342\%$$

$(A_{\text{st}})_{\text{min}} = 0.00342 \times 400 \times 532 = 727.776\text{mm}^2$ (Clause 6.2.1; IS 13920:2016)

Maximum Reinforcement = 0.025 = 2.5%

$(A_{\text{st}})_{\text{max}} = 0.025 * 400 * 532 = 5332 mm^2$ (Clause 6.2.2; IS 13920:2016)

6.3 Design for Flexure

Detailed calculations at left end are given in the following sections. The calculations for centre and right end are mentioned in table.

6.3.1 Design for Hogging Moment

Mu = 512 kN-m.

$x_{u,lim} = 0.48 * d = 255.36mm$

$M_{u,lim} = 0.36 * f_{ck} * b * x_{u,lim} * (d - 0.42 * x_{u,lim}) = 546.65\ kN - m$

As $M_u < M_{u,lim}$ Designing as Singly reinforced section but providing 50% of Ast as A_{sc}.

$$A_{st} = 0.5 * \frac{f_{ck} * b * d}{f_y}\left(1 - \sqrt{1 - \frac{4.6M_u}{f_{ck} * b * d * d}}\right) = 3258.54\ mm^2$$

Ast calculated is more than minimum and less than maximum.

A_{sc} at bottom = $= 0.5 * A_{st} = 1629.27\ mm^2$

6.3.2 Design for Sagging Moment

M_u = 237 kN-m

The beam is designed as T beam. The limiting capacity of L beam assuming $x_u < D_f$ and $x_u < x_{u,max}$ is calculated as:

D_f = 150mm.

$$B_f = \frac{L_o}{12} + b_w + 3 \times D_f = 1287.5\ mm\ or\ 7100\ mm$$

B_f = 1287.5 mm (lowest of above) (Clause 23.1.2 of IS456:2000)

$$A_{st} = 0.5 * \frac{f_{ck} * B_f * d}{f_y}\left(1 - \sqrt{1 - \frac{4.6M_u}{f_{ck} * B_f * d * d}}\right)$$

$$= 1262.05\ mm^2 (more\ than\ minimum\ and\ less\ than\ maximum\ reinforcement)$$

Checking design assumptions:

$$x_u = \frac{0.87 * f_y * A_{st}}{0.36 * f_{ck} * B_f} = 28.05\ mm < D_f\ \ (OK)$$

A_{sc} must be at least 50% of A_{st} and therefore

Hence revise to 1262.05/2 = 631.025 mm^2.

6.3.3 Required Reinforcement

Top Reinforcement provided = 3258.54 mm^2 (Maximum of two).

Bottom Reinforcement provided = 1629.7 mm^2 (Maximum of two).

6.4 Details of Reinforcement

Below Table 6 shows the reinforcement required at left end, centre and right end of the beam. The procedure followed to calculate the reinforcement at right end is the same as above.

For central bottom part:

A_{st} required = 663.60 mm^2.

This is less than minimum reinforcement required and 1/4th of longitudinal steel provided at top face of beam and face of column.

Therefore, A_{st} provided = 3258.54/4 = 814.635 mm^2.

For Top part:

A_{st} provided = 814.635 mm^2.

Table 6 Required reinforcement at different locations

Summary of Required Reinforcement			
	Left End	Centre	Right End
Top	3258.54 mm^2	814 mm^2	3240.1 mm^2
Bottom	1629.7 mm^2	814 mm^2	1620.05 mm^2

Table 7 Details of longitudinal reinforcement provided at various locations

Beam	**Longitudinal Reinforcement**		
	Left End	Centre	Right End
Top Reinforcement	6-25 Φ and 2-20 Φ Ast provided = 3573.56 mm^2	3-20 Φ Ast provided = 942.47 mm^2	6-25 Φ and 2-20 Φ Ast provided = 3573.56 mm^2
Bottom Reinforcement	6-20 Φ Ast provided = 1884.95 mm^2	3-20 Φ Ast provided = 942.47 mm^2	6-20 Φ Ast provided = 1884.95 mm^2

6.5 Design for Shear

6.5.1 Design Shear Strength of Concrete

Tensile steel provided at left end = 1.68 %.

Permissible design shear stress of concrete = τ_c= 0.81 MPa.

$\tau_{cmax} = 3.7MPa$

Design Shear Strength of Concrete $V_c = \tau_c * b * d = 172.36kN$

Similarly at centre and right design shear strength of concrete is 100.016kN and 172.36 kN.

6.5.2 Shear Force Due to Plastic Hinge Formation at Ends of Beam

The additional shear due to formation of plastic hinges at both ends of the beams is evaluated as per clause 6.3.3 of IS 13920:1993 and is given by:

$$V_{swaytoright} = \frac{\pm 1.4({M_u}^{As} + {M_u}^{Bh})}{L} \quad V_{swaytoleft} = \frac{\pm 1.4({M_u}^{Ah} + {M_u}^{Bs})}{L}$$

The sagging and hogging moments of resistance (${M_u}^{As}$, ${M_u}^{Bs}$, ${M_u}^{Ah}$ and ${M_u}^{Bh}$) at both ends of beam are calculated on the basis of the actual area of steel provided in the section.

The beam is provided with a steel area of 3573.56 mm^2 (i.e., p_t =1.68 %) at top and 1884.95 mm^2 (i.e., p_c = 0.88%) at bottom on the left end of the beam.

${M_u}^{Ah}$ = 604.24 kN-m, corresponding to x_u = 142.41< $x_{u, lim}$.

$M_u^{A_s}$= 357.48 kN-m, corresponding to x_u = 60.63< $x_{u, lim}$.

Similarly, for right end of the beam we obtain, $M_u^{B_h}$= 604.24 kN-m, $M_u^{B_s}$= 357.48 kN-m.

Shear is calculated as:

$V_{swaytoright} = 179.52\ kN\ \ V_{swaytoleft} = 179.52kN$

$DL = (0.4 * 0.6 * 25 + 16.1) * 7.5 + 0.25 * 7.5^2 * 0.15 * 25 + 1 * 7.5^2 * 0.25 = 232.5kN$

$LL = 4 * 7.5^2 * 0.25 = 56.25kN$

Shear at left end for sway to right,

$$V_{u,a} = \frac{1.2(DL + LL)}{2} - 1.4\frac{M_u^{A_s} + M_u^{B_h}}{L_{AB}} = -6.27kN$$

Shear at left end for sway to left,

$$V_{u,a} = \frac{1.2(DL + LL)}{2} + 1.4\frac{M_u^{A_s} + M_u^{B_h}}{L_{AB}} = 352.57kN$$

Shear at right end for sway to right,

$$V_{u,b} = \frac{1.2(DL + LL)}{2} - 1.4\frac{M_u^{A_h} + M_u^{B_s}}{L_{AB}} = -6.27kN$$

$$V_{u,b} = \frac{1.2(DL + LL)}{2} + 1.4\frac{M_u^{A_s} + M_u^{B_h}}{L_{AB}} = 352.57kN$$

As per Clause 6.3.3 of IS 13920:2016, the design shear force to be resisted shall be the maximum of:

i) Calculated factored shear forces as per analysis.

ii) Shear forces due to formation of plastic hinges at both ends of the beam plus factored gravity load on the span (as calculated above).

Hence, design shear force (V_u) will be 352.57kN for left and right end of the beam.

From analysis, the shear at the mid-span of the beam is 90.27kN. However, shear due to formulation of plastic hinges at both the ends of the beams has been calculated as 179.52kN. Hence, the design shear at center of the span is taken as 352.57kN.

The required capacity of shear reinforcement at left end of the beam is $V_{us} = V_u - V_c$ = 180.21kN.

Similarly, the, required capacity of shear reinforcement at the right end and at mid-span is 180.21kN and 79.36kN, respectively.

$$S_v = \frac{0.87 f_y d A_{sv}}{V_{us}}$$

The required spacing of 2 legged 8 Φ stirrups as 107.151 mm, 243.31 mm and 107.151 mm respectively at left end, center and right end.

As per Clause 6.3.5 of IS 13920:2016, the spacing of stirrups in the mid-span shall not exceed $d/2$ = 532/2 = 266 mm. Minimum shear reinforcement as per Clause 26.5.1.6 of IS 456:2000 is given by:

$$S_v \leq \frac{0.87 f_y A_{sv}}{0.4b} = 226.73mm$$

Spacing of links over a length of 2d at either end of beam as per Clause 6.3.5 of IS13920:2016 shall be the least of:

(i) $d/4$ = 532 /4 = 133 mm.

(ii) 6 times diameter of smallest longitudinal bar= 6×20 = 120 mm.

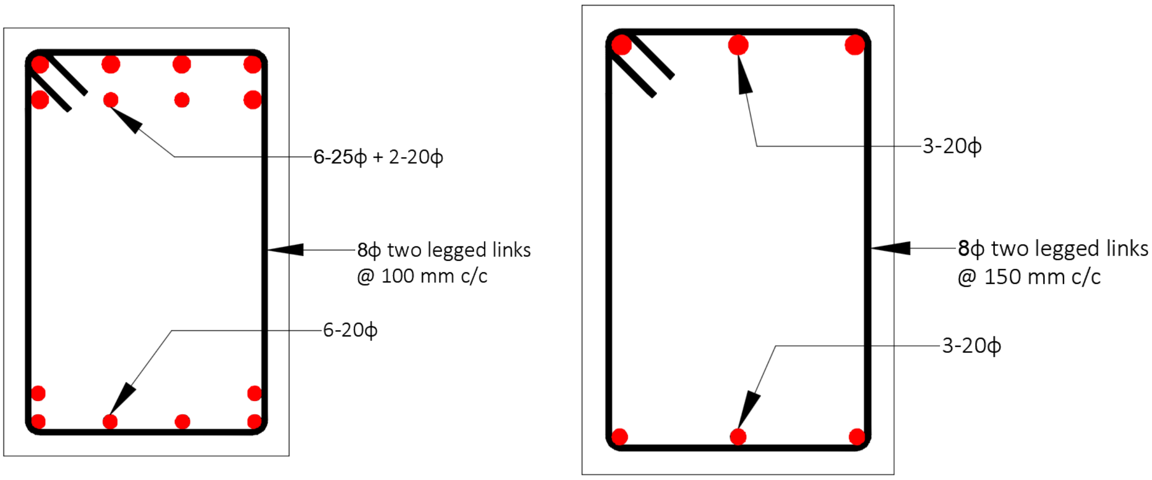
(iii)100mm.

Hence, provide 2 legged 8ϕ stirrups @100mm c/c at left and at right end over a length of 2d = 2 x 532 = 1,064 mm.

Elsewhere, provide stirrups at 150 mm c/c.

6.6 Reinforcement detailing

The details of reinforcement for beam AB at various locations are shown in below Figure 6.



(a) At ends A and B

(b) At center

Figure 6 Reinforcement details of beam

7. Column Design

The design calculations of column AB of frame A at 4th story as shown in below Figure 7 are shown in this section.

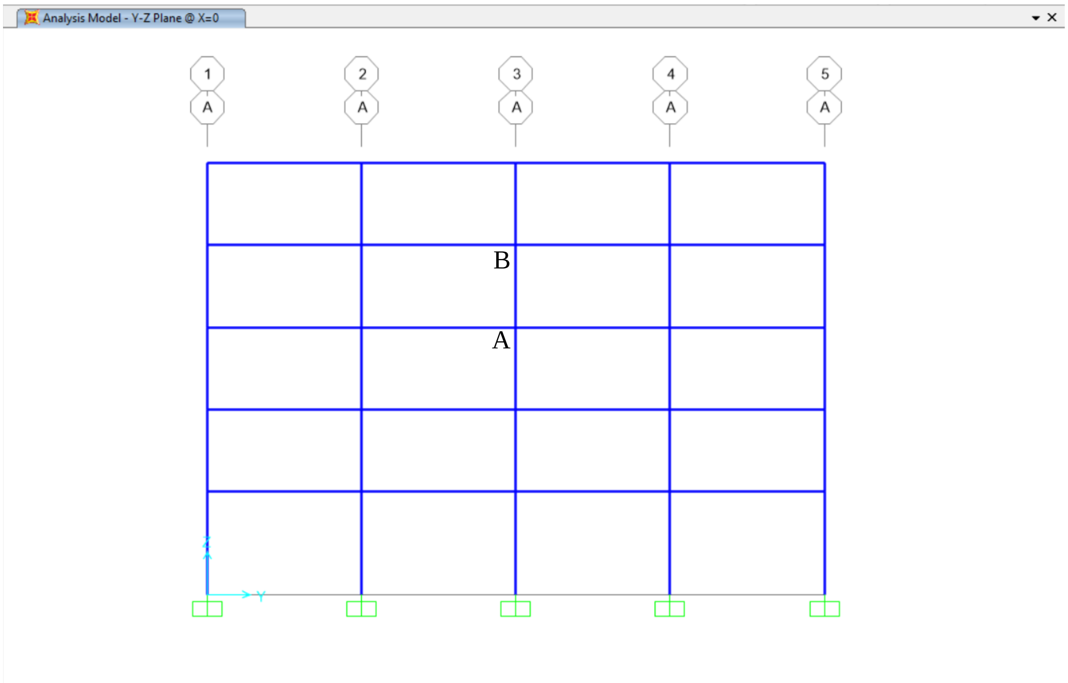


Figure 7 Column considered for designing

7.1 Member forces

Member forces for the column AB at different locations and load combinations are shown in below Table 8.

Table 8 Forces for the column AB at different locations and load combinations

Load Combinations	**A_B**			**A_T**			**B_B**			**B_T**		
	Axial kN	M_2 kN-m	M_3 kN-m	Axial kN	M_2 kN-m	M_3 kN-m	Axial kN	M_2 kN-m	M_3 kN-m	Axial kN	M_2 kN-m	M_3 kN-m
1.5(DL+LL)	**-1786**	0	0	**-1056**	0	0	**-1019**	0	2	**-289**	0	0
1.2(DL+LL+EQX)	-1429	11	2.5	-845	11	2.5	-815	11	22	-231	11	2
1.2(DL+LL-EQX)	-1429	-11	-2.5	-845	-11	-2.5	-815	-11	-19	-231	-11	-2
1.2(DL+LL+EQY)	-1429	561	0	-845	344	0	-815	420	1.5	-231	133	0
1.2(DL+LL-EQY)	-1429	-561	0	-845	-344	0	-815	-420	1.5	-231	-133	0
1.5(DL+EQX)	**-1447**	**13**	**3**	**-886**	**13**	**3**	**-849**	**13**	**28**	**-288**	**14**	**2**
1.5(DL-EQX)	-1447	-13	-3	-886	-13	-3	-849	-13	-24	-288	-14	-2
1.5(DL+EQY)	**-1447**	**701**	**0**	**-886**	**430**	**0**	**-849**	**525**	**2**	**-288**	**166**	**0**
1.5(DL-EQY)	-1447	-701	0	-886	-430	0	-849	-525	2	-288	-166	0
0.9DL+1.5EQX	-869	13	3	-532	13	3	-509	13	27	-173	14	2
0.9DL-1.5EQX	-869	-13	-3	-532	-13	-3	-509	-13	-25	-173	-14	-2
0.9DL+1.5EQY	-869	701	0	-532	430	0	-509	525	2	-173	166	0
0.9DL-1.5EQY	-869	-701	0	-532	-430	0	-509	-525	2	-173	-166	0

7.2 Design Checks

7.2.1 Check for Axial Stress

Lowest factored axial force = 509kN (Lowest at A_t or B_b among all load combination is considered).

Factored axial stress = 509000/ (500×500) = 2.92 MPa > 0.08f_{ck}.

Hence design as a column member (Clause 7.1; IS 13920:2016).

7.2.2 Check for Member Size

Width of Column = 500mm > 300mm.

Hence OK (Clause 7.1.1 (b), IS 13920:2016).

Depth of column, D = 500 mm

B/D = 500/500 = 1 > 0.4. (Clause 7.1.2, IS 13920:2016). Hence, ok.

Span L = 4000mm.

The effective length of column can be calculated using Annex E of IS 456: 2000. In this design, as per Table 28 of IS 456: 2000, the effective length is taken as 0.85 times the unsupported length, which is in between that of fixed and hinged case.

$\frac{L}{D} = \frac{(4000-600)\times 0.85}{500} = 5.78 < 12$. Hence, the member can be designed as short column. (Clause 25.1.2 of IS 456: 2000)

Minimum dimension of column = 500 mm ≥ 20 times the largest diameter of beam longitudinal reinforcement = 20×25 = 500 mm. Hence, ok. (Clause 7.1.1 (a) of IS 13920:2016)

7.2.3 Check for limiting longitudinal reinforcement:

Minimum reinforcement = 0.8% = 0.8×500×500/100 = 2000 mm^2 (Clause 26.5.3.1 of IS 456: 2000).

Maximum reinforcement = 4% = 4×500×500/100 = 10000 mm^2 (Clause 26.5.3.1 of IS 456: 2000).

7.3 Design of Column:

Sample calculation is done for column reinforcement at A_B end.

7.3.1 Design for earthquake in X-direction:

P_u = -1447 kN, M_{u2} = 13 kN-m

$\frac{P_u}{f_{ck}bd} = \frac{1447\times 10^3}{35\times 500\times 500} = 0.165$; $\frac{M_u}{f_{ck}bD^2} = \frac{13\times 10^6}{35\times 500\times 500^2} = 0.0029$

For d'/D = 0.1, we get p/f_{ck} = 0.00, from the design chart (refer SP-16:1980, chart 44).

7.3.2 Design for Earthquake in Y-direction:

P_u = -1447 kN, M_{u2} = 701 kN-m

$\frac{P_u}{f_{ck}bD} = \frac{1447\times10^3}{35\times500\times500}$ = 0.165; $\frac{M_u}{f_{ck}bD^2} = \frac{701\times10^6}{35\times500\times500^2}$ = 0.16

For d'/D = 0.1, we get p/f_{ck} = 0.11, from the design chart (refer SP-16:1980, chart 44).

<u>Longitudinal steel:</u>

The required steel will be governed by the higher of the above two values and hence, take p/f_{ck} = 0.11.

Required steel = (0.11×35)%= 3.85% = 3.85×500×500/100= 9625 mm^2.

Provide 20-25Φ bars with total area of reinforcement (A_s) provided is 9817 mm^2.

Hence, p provided = 3.9% (within maximum and minimum reinforcement limits).

7.3.3 Checking of section:

The column should be checked for bi-axial moment. Moment about other axis may occur due to torsion of building or due to minimum eccentricity of the axial load.

Checking for critical combination with earthquake in X-direction (longitudinal direction):

Width = 500 mm; Depth = 500 mm, P_u = 1,447 kN, M_{u2} = 13 kN-m, M_{u3} = 3 kN-m.

Eccentricity = clear height of column/500 +lateral dimension/30 (Clause 25.4 of IS 456:2000)

= ((4000-600)/500) + (500/30) = 23.47 mm > 20 mm.

Hence, design eccentricity = 23.47 mm.

$M_{u3} = M_{u2}$ = 1447×0.02347 = 33.96 kN-m > applied moment.

For $\frac{P_u}{f_{ck}bD}$ = 0.165 and p/f_{ck} = 0.11, from design chart (refer SP-16:1980, chart 44) $\frac{M_u}{f_{ck}bD^2}$ = 0.17.

Hence, actual moment of resistance is:

$Mu_{2,1}$ = 0.17×35×500×500×500/(1×10^6) = 743.75 kN-m.

$Mu_{3,1}$ = 0.17×35×500×500×500/(1×10^6) = 743.75 kN-m.

$P_{uz} = 0.45f_{ck}\,A_c + 0.75f_yA_s$ (Clause 39.6 of IS 456:2000)

$= 0.45f_{ck}A_g + (0.75f_y - 0.45\,f_{ck})A_s$

= 0.45×35×500×500 + (0.75×415–0.45×35)×9817 = 6838.42 kN

P_u/P_{uz} = 1447/6838.42 = 0.21

The constant α_n which depends on factored axial compression resistance P_{uz} is evaluated as (Clause 39.6, IS 456: 2000): $\alpha_n = 1$

Therefore,

$\left[\frac{M_{u2}}{M_{u2,1}}\right]^{\alpha_n} + \left[\frac{M_{u3}}{M_{u3,1}}\right]^{\alpha_n} = \left[\frac{33.96}{743.75}\right]^1 + \left[\frac{33.96}{743.75}\right]^1$ = 0.045+0.045 = 0.09 < 1.00. Hence, ok.

Checking for critical combination with earthquake in Y-direction (transverse direction):

Width = 500 mm

Depth = 500 mm

P_u = 1447 kN

M_{u3} = 701 kN-m

Eccentricity = clear height of column/500 +lateral dimension/30

(Clause 25.4 of IS 456:2000)

= ((4000-600)/500) + (500/30) = 23.47 mm > 20 mm.

Hence, design eccentricity = 23.47 mm.

M_{u2} = 1447×0.02347 = 33.96 kN-m > applied moment.

For $\frac{P_u}{f_{ck}bD}$ = 0.165 and p/f_{ck} = 0.11, from design chart (refer SP-16:1980, chart 44)

$\frac{M_u}{f_{ck}bD^2}$ = 0.17.

Hence, actual moment of resistance is:

$Mu_{2,1}$ = 0.17×35×500×500×500/(1×10^6) = 743.75 kN-m

$Mu_{3,1}$ = 0.17×35×500×500×500/(1×10^6) = 743.75 kN-m

$P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_s$ (Clause 39.6 of IS 456:2000)

= $0.45f_{ck}A_g + (0.75f_y-0.45\ f_{ck})A_s$

= 0.45×35×500×500 + (0.75×415–0.45×35)×9817 = 6838.42 kN

P_u/P_{uz} = 1447/6838.42 = 0.21

The constant α_n which depends on factored axial compression resistance P_{uz} is evaluated as (Clause 39.6, IS 456: 2000):

$\alpha_n = 1$

Therefore,

$\left[\frac{M_{u2}}{M_{u2,1}}\right]^{\alpha_n} + \left[\frac{M_{u3}}{M_{u3,1}}\right]^{\alpha_n} = \left[\frac{33.96}{743.75}\right]^1 + \left[\frac{701}{743.75}\right]^1$ = 0.045+0.94 = 0.985 < 1.00.

Hence, ok.

7.3.4 Details of longitudinal reinforcement:

Similar to the sample calculations shown above, the steel required at A_t, B_b and B_t is calculated. Table 9 and 10 show brief calculations at all sections. The details of the provided reinforcement are given in Table 11.

Table 9 Critical forces for design of column AB

Load combination	A_b			A_t			B_b			B_t		
	P_u	M_2	M_3	P_u	M_2	M_3	P_u	M_2	M_3	P_u	M_2	M_3
Gravity	-1786	0	0	-1056	0	0	-1019	0	2	-289	0	0
Critical combination with EQX	-1447	13	3	-886	13	3	-849	13	28	-288	14	2
Critical combination with EQY	-1447	701	0	-886	430	0	-849	525	2	-288	166	0

Table 10 Design of column AB for earthquake in X and Y direction

Load combination	A_b			A_t			B_b			B_t		
	$\frac{P_u}{f_{ck}bD}$	$\frac{M_u}{f_{ck}bD^2}$	p	$\frac{P_u}{f_{ck}bD}$	$\frac{M_u}{f_{ck}bD^2}$	p	$\frac{P_u}{f_{ck}bD}$	$\frac{M_u}{f_{ck}bD^2}$	p	$\frac{P_u}{f_{ck}bD}$	$\frac{M_u}{f_{ck}bD^2}$	p
Gravity	0.20	0	0.8	0.12	0	0.8	0.11	0	0.8	0.03	0	0.8
Critical combination with EQX	0.165	0.00	0.8	0.10	0.00	0.8	0.097	0.00	0.8	0.03	0.00	0.8
Critical combination with EQY	0.165	0.16	3.85	0.10	0.1	2.1	0.097	0.12	2.8	0.03	0.04	0.7

Table 11 Longitudinal reinforcement provided in column AB at ends A and B

Column AB	Longitudinal reinforcement
Reinforcement at A	20-25Φ Steel provided = 9817 mm^2, i.e., 3.9%
Reinforcement at B	12-25Φ + 4-20Φ Steel provided = 7147 mm^2 i.e., 2.85 %

7.3.5 Design for shear:

Table 12 shows shear forces in column AB for different load combinations:

Table 12 Shear forces in column AB

S. No.	Load Combinations	A_B		A_T		B_B		B_T	
		V_x	V_y	V_x	V_y	V_x	V_y	V_x	V_y
1	1.5(DL+LL)	0	0	0	0	0	0	0	0
2	1.2(DL+LL+EQX)	2	6	1	6	1	6	1	6
3	1.2(DL+LL-EQX)	-2	-6	-1	-6	-1	-6	-1	-6
4	1.2(DL+LL+EQY)	0	265	0	191	0	191	0	85
5	1.2(DL+LL-EQY)	0	-265	0	-191	0	-191	0	-85
6	1.5(DL+EQX)	2	7	2	7	2	7	1	8
7	1.5(DL-EQX)	-2	-7	-2	-7	-2	-7	-1	-8
8	1.5(DL+EQY)	0	**332**	0	239	0	239	0	106
9	1.5(DL-EQY)	0	**-332**	0	-239	0	-239	0	-106
10	0.9DL+1.5 EQX	2	7	2	7	2	7	1	8
11	0.9DL-1.5 EQX	-2	-7	-2	-7	-2	-7	-1	-8
12	0.9DL+1.5 EQY	0	**332**	0	239	0	239	0	106
13	0.9DL-1.S5 EQY	0	**-332**	0	-239	0	-239	0	-106

Assuming 50% steel provided as tensile steel to be on conservative side, A_{st} = 2.85%/2 = 1.425%.

Permissible shear stress, τ_c = 0.765 MPa (Table 19 of IS 456 : 2000).

Considering lowest P_u = 525 kN, we get

Multiplying factor = $\delta = 1 + \frac{3P_u}{f_{ck}A_g}$ = 1.18 (Cl. 40.2.2 of IS 456 : 2000)

τ_c = 0.765×1.18 = 0.9027 MPa.

Effective depth = 500-40-8-25/2 = 439.5 mm.

V_c = 0.9027×500×439.5/1000 = 198.37 kN

Shear force from capacity design principle for earthquake in y-direction:

The transverse beam of size 500×500 mm, the hogging and sagging moment capacities are evaluated as 604.24 kN-m and 357.48 kN-m, respectively.

Referring to Cl. 7.5; IS 13920 : 2016), the shear force corresponding to plastic hinge formation in longitudinal beam is evaluated as:

For sway to right: $V_u = 1.4\left(\frac{M_u^{As}+M_u^{Bh}}{h_{st}}\right) = V_u = 1.4 \times \frac{(357.48+604.24)}{4} = 336.604$ kN

For sway to left: $V_u = 1.4\left(\frac{M_u^{Ah}+M_u^{Bs}}{h_{st}}\right) = V_u = 1.4 \times \frac{(604.24+357.48)}{4} = 336.604$ kN

Therefore, the design shear demand in column is maximum of above values and maximum shear force generated due to all load combinations, which is 336.604 kN.

Details of shear reinforcement:

$V_{us} = V_u - V_c = 336.604 - 198.37 = 138.234$ kN

Spacing of 3 legged 10Φ links = $\frac{0.87 \times 415 \times 78 \times 3 \times 437.5}{138.234 \times 10^3} = 267.4$ mm.

Nominal links:

The nominal spacing of links is given as half the least lateral dimension of the column i.e., 500/2 = 250 mm. (Cl. 7.4 of IS 13920 : 2016).

Hence provide 8Φ links @ 250 mm c/c in mid-height portion of column.

Confining links:

The area of cross-section, A_{sh}, of the bar forming rectangular hoop, to be used as special confining reinforcement shall not be less than: (Cl. 7.6.1 of IS 13920 : 2016).

$$A_{sh} = \text{maximum of} \begin{bmatrix} 0.18\, s_v\, h\, \frac{f_{ck}}{f_y}\left(\frac{A_g}{A_k} - 1\right) \\ 0.05\, s_v\, h\, \frac{f_{ck}}{f_y} \end{bmatrix}$$

h = longer dimension of the rectangular link measured to its outer face = $\frac{500-40\times2-10\times2-25}{5} \times 3 + 10 = 235$ mm.

$A_g = 500 \times 500 = 250000$ mm^2

$A_k = (500\text{-}40\text{-}40) \times (500\text{-}40\text{-}40) = 176400$ mm^2

Assuming 10Φ stirrups, $A_{sh} = 78$ mm^2

Substituting in above equation, we get s_v = minimum of $\begin{bmatrix} 52.4 \text{ mm} \\ 78.7 \text{ mm} \end{bmatrix}$;

Therefore, $s_v = 52.4$ mm.

Link spacing for confining zone shall not exceed 6 times diameter of the smallest longitudinal reinforcement bars = 6×20 = 120 mm. (Cl. 7.6.1(b) of IS 13920 : 2016, see amendment).

Provide 10Φ confining links @ 50 mm c/c for a distance l_o, which shall not be less than the following (Cl. 7.6.1(a) of IS 13920 : 2016):

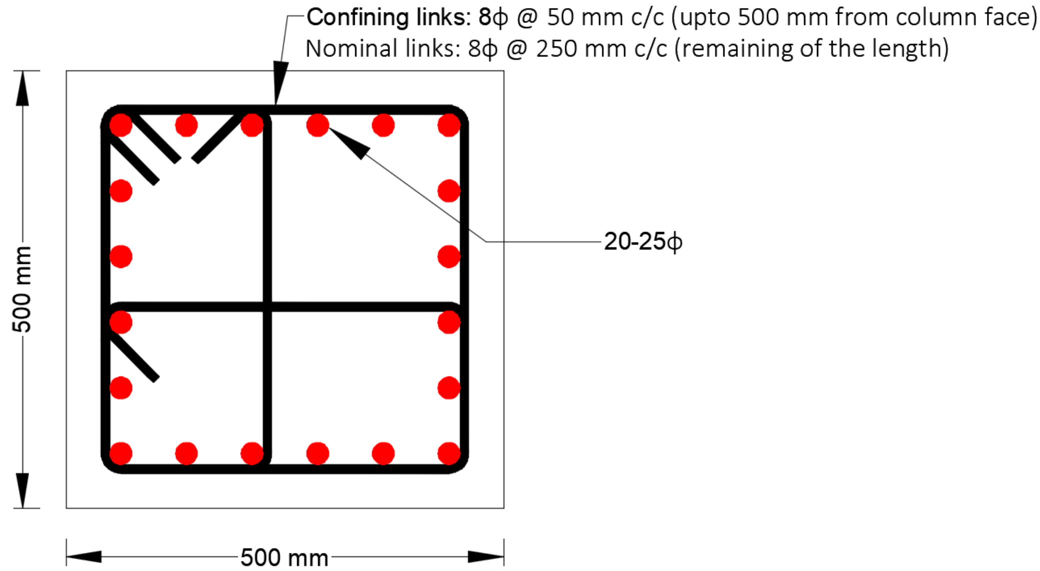
(i) Larger lateral dimension = 500 mm

(ii) 1/6 of clear span = (3100-350)/6 = 458 mm (iii)450 mm

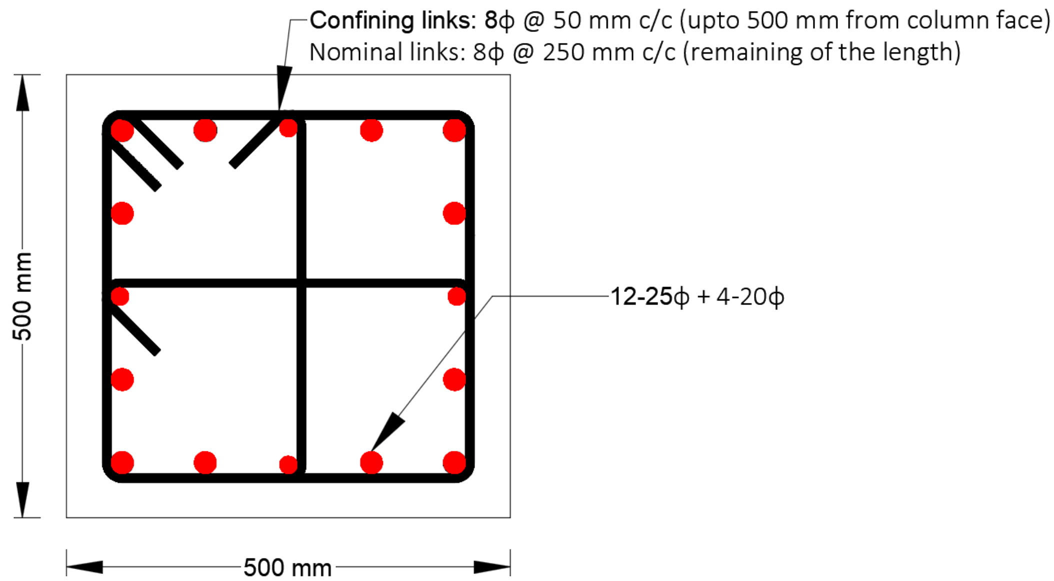
Therefore, provide confining links @ 50 mm c/c for a distance of 500 mm from the face of beam.

7.4 Reinforcement detailing

The details of reinforcement for column AB at various locations are shown in below Figure 8.



(a) At end A



(b) At end B

Figure 8 Reinforcement details of beam

8. Design of Shear Wall

8.1 Design forces

The force resultants in shear wall at the base obtained for various load combinations are shown in below Table 13.

Table 13 Force resultants in shear wall

S No	Load Combinations	Bending Moment (kN-m)	Axial Force (kN)	Shear Force (kN)
1	1.5DL+1.5LL	0	5413.5465	0
2	1.2(DL+LL*+EQX)	25753.48056	4341.4968	3027.1524
3	1.2(DL+LL*-EQX)	-25753.48056	4320.1776	-3027.1524
4	1.5(DL+EQX)	32191.8507	4279.9575	3783.9405
5	1.5(DL-EQX)	-32191.8507	4253.3085	-3783.9405
6	0.9DL+1.5 EQX	32191.8507	2573.3043	3783.9405
7	0.9DL-1.5 EQX	-32191.8507	2546.6553	-3783.9405

8.2 Structural Parameters:

Length of Wall L_w= 7000mm. Height of wall h_w= 21m.

Length of Web L_{web} = 6500mm. Thickness of web t_w= 200mm.

Boundary Element: Depth d_{be} = 0.25m thickness t_{be} = 0.5m.

Material: Concrete f_{ck} = 35MPa; Steel f_y = 415MPa E_s = 200GPa.

Design Loads from load combination are to be taken with taking maximum case.

8.3 Some General Requirements:

Wall Thickness t_w= 200mm > 150 mm OK (CL. 10.1.2, IS 13290-2016).

Length to thickness ratio $\frac{L_w}{t_w} = \frac{7000}{4} = 35 > 4$ Ok (CL. 10.1.3, IS 13290-2016).

$\frac{h_w}{L_w} = \frac{21000}{7000} = 3 > 2$ Therefore, the wall is a slender wall. (CL. 10.1.4, IS 13290-2016).

Min. reinforcements in slender wall: (Table 1, IS 13920-2016)

ρ_{vbemin} = 0.008; $\rho_{vwebmin}$ = 0.0025;

$$\rho_{hmin} = 0.0025 + 0.5 * \left(\frac{h_w}{L_w} - 2\right) * (\rho_{vwebmin} - 0.0025) = 0.0025$$

Maximum spacing of horizontal and vertical reinforcement:

$$S_{max} = min\left(\frac{L_w}{5}, 3 * t_w, 450mm\right) = \min(1300,600,450) = 450mm$$

8.4 Design for Shear

Using the load combinations, taking the maximum case:

$V_u = 3784kN$

Max. design shear strength of concrete $\tau_{cmax} = 4MPa$ (Table 20, IS 456-2000).

Effective depth in resisting shear $d_w = L_{web} + d_{be} = 6750mm$

Nominal shear stress demand $\tau_v = \frac{V_u}{d_w * t_w} = 2.80 < \tau_{cmax}$ Ok (Cl. 10.2.3, IS 13920-2016).

Assuming min. vertical reinforcement ($\rho_v = \rho_{vbemin}$ =0.0025) to be provided, shear strength of concrete $\tau_c = 0.37MPa$. (Table 1, IS 13920-2016 & Table 19, IS 456-2000).

Shear carried by concrete $V_{uc} = \tau_c * d_w * t_w = 0.37 * 6750 * 200 = 499.5kN$

Shear to be resisted by horizontal reinforcement $V_{us} = V_u - V_{uc} = 3280.5kN$.

Therefore, the ratio $\left(\frac{A_h}{S_v}\right)_{required} = \frac{V_{us}}{0.87 * f_y * d_w} = 1.34mm$ (Cl. 10.2.3(b), IS 13920-2016).

Based on min. horizontal reinforcement requirement, the $\left(\frac{A_h}{S_v}\right)$ $ratio = \rho_{hmin} * t_w = 0.5mm$.

Therefore, the former ratio of $\left(\frac{A_h}{S_v}\right)$ governs.

As the wall thickness t_w =200 mm, two curtains of reinforcement need to be provided. (Cl. 10.1.7, IS 13920-2016).

Thus, the required spacing for two layers of horizontal reinforcement assuming d_s = 10 mm:

$$A_h = 2 * \frac{\pi}{4} * 10^2 = 157.07mm^2$$

$$S_v = \frac{157.07}{max\left(\frac{V_{us}}{0.87 f_y * d_w}, \rho_{hmin} * t_w\right)} = 117.21mm < 450mm.$$

So, provide horizontal reinforcement of d_s =10mm diameter bars @ 100 mm c/c in two layers.

8.5 Flexural Strength of Web

Using the load combinations, taking the maximum case:

$M_u = 32192kN - m$

Assuming this axial load to be uniformly distributed, the web takes:

$$\alpha_{web} = \frac{L_{web} * t_w}{L_{web} * t_w + 2 * d_{be} * t_{be}} * 100 = 83.87\%$$

Therefore, the load on web $P_{uweb} = \alpha_{web} * P_u = 4540.72kN$

Moment of resistance of a slender rectangular shear wall section with uniformly distributed vertical reinforcement can be estimated as per IS 13920-2016 (Annex A).

Assume that the minimum vertical reinforcement is provided in the web (i.e., 10 mm dia bars at 100 mm c/c as provided in the horizontal direction).

$$\phi = \frac{0.87 * f_y * \rho_{vwebmin}}{f_{ck}} = 0.082, \qquad \lambda = \frac{P_{uweb}}{f_{ck} * t_w * L_{web}} = 0.099$$

Depth of neutral axis expressed as a fraction of length of web (i.e., $\frac{x_u}{L_{web}}$):

$$\frac{x_u}{L_w} = \frac{\phi + \lambda}{2 * \phi + 0.36} = 0.35$$

Limiting depth of NA for flexural tension failure expressed as a fraction of length of web:

$$\frac{x_{ustar}}{L_w} = \frac{0.0035}{0.0035 + \left(0.002 + \frac{0.87 * f_y}{E_s}\right)} = 0.48$$

$$\beta = \frac{0.002 + \frac{0.87 * f_y}{E_s}}{0.0035} = 1.087$$

Since $\frac{x_u}{L_w} < \frac{x_{ustar}}{L_w}$, therefore, moment of resistance of the web:

$$M_{uv} = \phi * \left(\left(1 + \frac{\lambda}{\phi}\right) * \left(0.5 - 0.416 * \frac{x_u}{L_w}\right) - \left(\frac{x_u}{L_w}\right)^2 * \left(0.168 + \frac{\beta^2}{3}\right)\right) * f_{ck} * t_w * L_{web}^2$$

$$= 17302.13kN - m$$

Remaining Moment $M_u - M_{uv} = 14892kN - m$ is to be resisted by boundary elements.

8.6 Design of Boundary Elements

Assuming no boundary elements, the extreme fibre compressive stress due to factored gravity loads plus factored earthquake forces

$$\sigma = \frac{P_u}{L_w * t_w} + \frac{6 * M_u}{t_w L_w^2} = 27MPa > 0.2f_{ck}$$

Hence, boundary elements are required (Cl. 10.4.1, IS 13920-2016).

Centre to centre distance between boundary elements $C_w = L_{web} + d_{be} = 6750mm$

Axial force in the boundary element due to earthquake loading $P_{EQBE} = \frac{M_u - M_{uv}}{C_w} = 2205.92kN$

(This includes the axial force due to gravity induced moment).

Maximum factored axial compression on the boundary element:

$$P_{uCBE} = P_{EQBE} + \frac{1 - \alpha_{web}}{2} * 1.5 * (P_{grav} + P_{EQ}) = 2642.55kN$$

Maximum factored axial tension on the boundary element:

$$P_{uTBE} = -P_{EQBE} + \frac{1 - \alpha_{web}}{2} * (0.8 * P_{grav} - 1.5 * P_{EQ}) = -1023kN$$

Assuming short column action the axial load capacity of the boundary elements can be calculated using IS 456. (Cl. 10.4.2, IS 13920-2016).

Gross area of boundary element $A_g = d_{be} * t_{be} = 125000mm^2$

Let's provide steel in boundary element as $\rho = 0.03$

Axial compressive strength $= 0.4 * f_{ck} * A_g * (1 - \rho) + 0.67 * f_y * \rho * A_g$

$= 2740kN > P_{uCBE}$

Axial tensile strength of the section $= 0.87 * f_y * \rho * A_g = 1353kN > |P_{uTBE}|$

Provide $0.03 * 125000 = 3750\text{mm}^2$, provide 12 no of 20 mm diameter.

8.7 Special Confining Reinforcement

Assuming $d_{sbe} = 10mm$ diameter confining stirrups,

$$A_{sh} = \frac{\pi}{4} * 10^2 = 78.54mm^2$$

$h_1 = d_{be} - 2 * 40 = 170mm < 300mm$ Ok

$h_2 = \frac{t_{be} - 2*40 - 2*d_{sbe} - 20}{3} + 20 + 2 * d_{sbe} = 166.67 < 300mm$ Ok (Cl. 7.4.2, IS 13920-2016).

$h = \max(h_1, h_2) = 170mm$

Required Spacing:

$$S_v = \frac{A_{sh} * f_y}{0.05 * h * f_{ck}} = 109.56mm$$

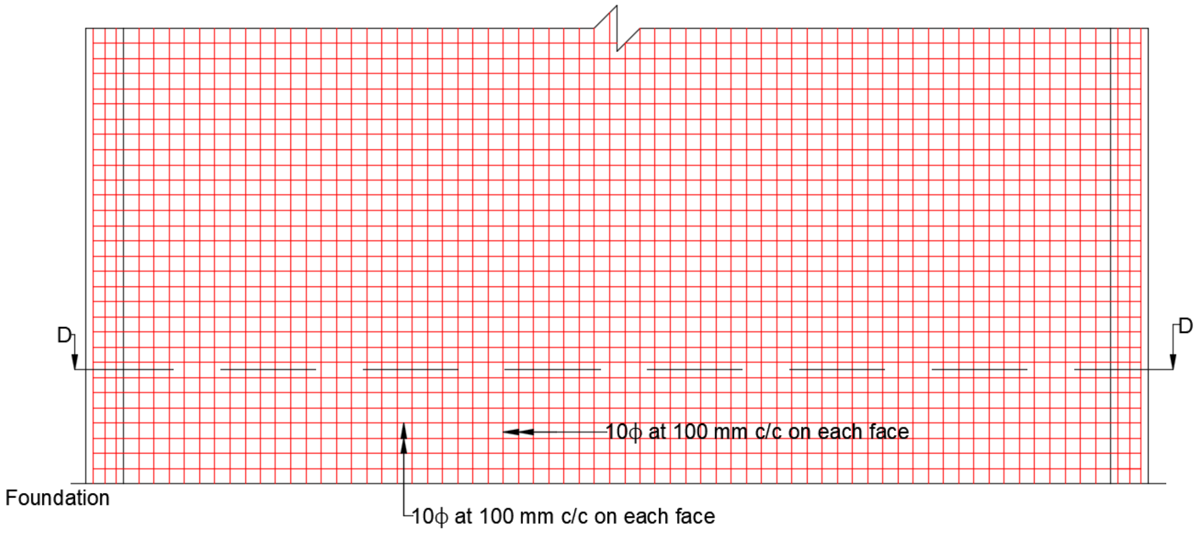
$$Maximum\ Spacing = min\left(\frac{1}{3} * 500, 6 * 20, 100\right) = 100mm$$

Also, the spacing need not be more than 100 mm. (Cl. 10.4.4, IS 13920-2016).

Therefore, provide 10 mm diameter rectangular hoops at 100 mm centre to centre.

8.8 Reinforcement detailing

Figure 9 shows the reinforcement details for the shear wall.



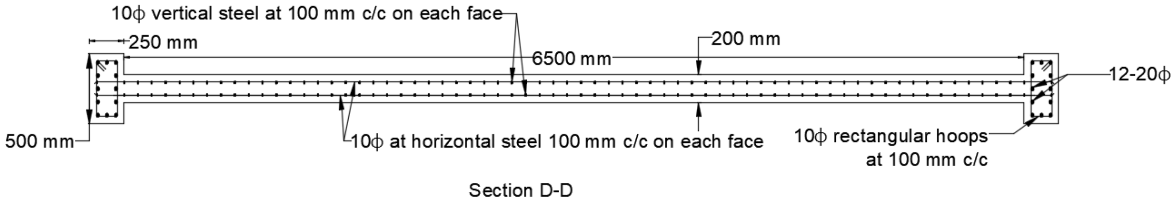


Figure 9 Reinforcement detailing of shear wall

9. Design of Beam-Column Joint

9.1 Preliminary Design Data

The details of the column and beam reinforcement meeting at the joint are shown.

The longitudinal beam of size 400 x 600 is reinforced with 6-25Φ + 2-20Φ (3573.56 mm^2) at top and 6-20Φ (1884.95 mm^2) at bottom. The hogging and sagging moment capacity is evaluated as 604.24 kN-m and 357.48 kN-m, respectively.

9.2 Check for Earthquake in Y direction

9.2.1 Joint Shear

The joint equilibrium is shown in figure.

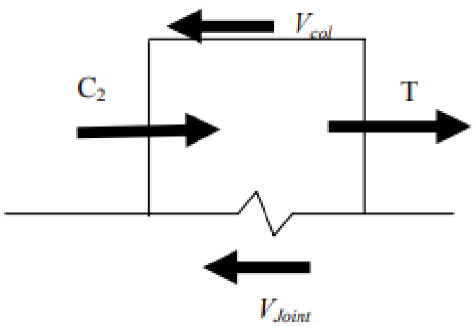


Figure 10 Joint Shear (source: Explanatory Examples for Ductile Detailing of RC Buildings IITK-GSDMA_EQ22-V3.0)

Column Shear:

The Column Shear is explained below.

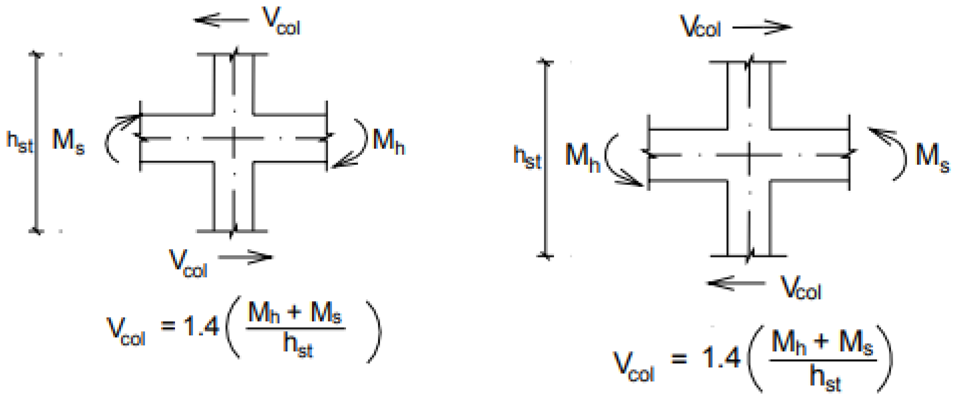


Figure 11 Column with sway to left and right (source: Explanatory Examples for Ductile Detailing of RC Buildings IITK-GSDMA_EQ22-V3.0)

For both the above cases:

$$V_{col} = 1.4 * \left(\frac{M_s + M_h}{h_{st}}\right) = 1.4 * \left(\frac{604 + 357}{4}\right) = 336.35kN$$

Force Developed in Beam Reinforcement:

Force developed in the top bars: $T_1 = A_{st} * 1.25 * f_y = 3573.56 * 1.25 * 415$

$$= 1853.78kN = C_1$$

The factor 1.25 is to account for the actual ultimate strength being higher than the actual yield strength. (Draft revision of IS 13920).

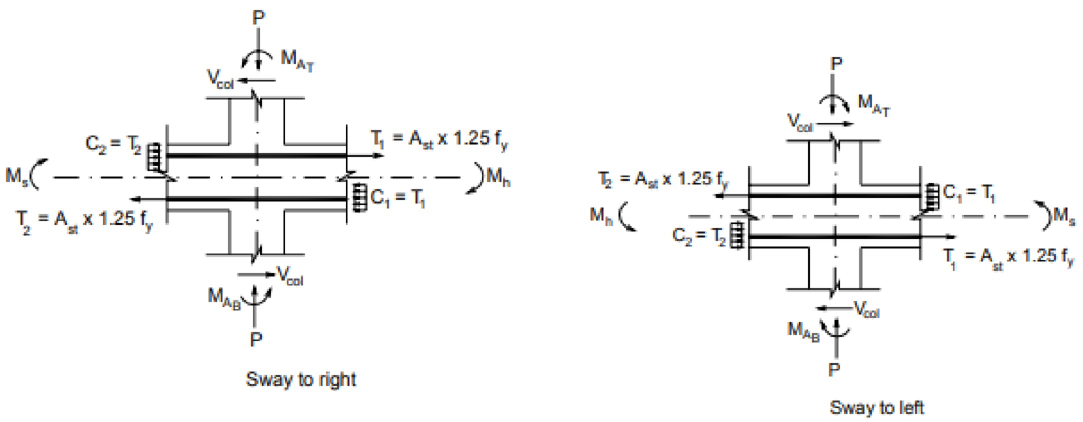


Figure 12 Free Body Diagram of Joint (source: Explanatory Examples for Ductile Detailing of RC Buildings IITK-GSDMA_EQ22-V3.0)

Force developed in the bottom bars:

$T_2 = A_{st} * 1.25 * f_y = 1884.78 * 1.25 * 415 = 977.817kN$

$V_{joint} = T_1 + C_2 - V_{col} = 1853.78 + 977.817 - 336.35 = 2495.25kN$

9.2.2 Check for Joint Shear Strength

In our case, $b_b = 400mm, b_c = 500mm, h = 500mm$

The effective width of the joint is lesser of the following:

1. $b_j = b_b + 0.5 * h = 400 + 0.5 * 500 = 650mm.$
2. $b_j = b_c = 500mm$

Therefore, $b_j = 500mm$

Effective Shear Area $A_c = b_j * h = 500 * 500 = 250000mm^2$

Shear Strength of concrete for joint connected by beams on 3 sides $= 1.2 * f_{ck}{}^{0.5} * A_c$

$= 1774.82kN < 2495.25kN$

Hence, Not Safe.

9.2.3 Check for Flexural Strength Ratio

The hogging and sagging moment capacities of the longitudinal beam are 604.24kN-m and 357.48kN-m.

Column is reinforced with 12-25Φ+4-20Φ. $A_{sc} = 7147mm^2 (p = 2.85\%)$

$$\frac{p}{f_{ck}} = 0.08$$

For conservative design, we calculate the moment capacity of column with zero axial loads. Therefore, referring to charts of SP:16 corresponding to $\frac{P}{f_{ck}*b*D} = 0$ at A_b to be conservative side for $\frac{p}{f_{ck}} = 0.08$ and $\frac{d\prime}{D} = 0.1$, we get

$$\frac{M_u}{f_{ck} * b * D^2} = 0.11$$

$$M_u = 0.11 * 35 * 500 * \frac{500^2}{10^6} = 612.5kN - m$$

Now, checking for strong column and weak beam condition:

$$\sum M_c = 612.5 * 2 = 1225kN - m$$

$$\sum M_b = 604.24 + 357.48 = 961.73kN - m$$

The ratio, $\frac{\sum M_c}{\sum M_b} = 1.27 < 1.4$

9.3 Confining Links

For columns bounded on 3 sides by beams, special confining reinforcements as calculated for columns shall be provided through the joint within the depth of the shallowest beam framing into it (as per clause 9.2.1(b) of IS 13920 : 2016). Therefore, provide 10 mm diameter confining links @ 50 mm c/c in the joint region with all 135º hooks of cross ties along the outer face of columns.

Revision

As can be seen from the checks in section above, the joint is not safe. In such cases the following three alternatives can be tried.

1. Increase the column section so that the joint area is increased. This will also reduce the main longitudinal steel requirement in the column owing to larger column size.
2. Increase the size of the beam section. If this option is adopted, it is advisable to increase the depth of the beam. This will reduce the steel required in the beam and hence will reduce the joint shear. In case of depth restriction in the beam, increase in beam width can be considered if the difference between the shear strength of joint and joint shear is small.
3. Increase the grade of concrete. This option will increase the shear strength of joint and also reduce the steel required in columns.

It is proposed to increase the size of beam from 400×600 mm to 500×750 mm and column sizes from 500×500 mm to 600×600 mm. Then, the structure will be reanalysed, and design forces should be reobtained from analysis and the design procedure is repeated.

10. References

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