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Explanatory Examples for Ductile Detailing of RC Buildings

by

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- The solved examples included in this document are based on a draft code being developed under IITK-GSDMA Project on Building Codes. The draft code is available at *http://www.nicee.org/IITK-GSDMA/IITK-GSDMA.htm* (document number *IITK-GSDMA-EQ11-V3.0*).
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Example 1 - Beam Design of an RC Frame Building in Seismic Zone V

1 Problem Statement:

A ground plus four storey RC office building of plan dimensions 19 m x 10 m located in seismic zone V on medium soil is considered. It is assumed that there is no parking floor for this building. Seismic analysis is performed using the codal seismic coefficient method. Since the structure is a regular building with a height less than 16.50 m, as per Clause 7.8.1 of IS 1893 (Part 1): 2002, a dynamic analysis need not be carried out. The effect of finite size of joint width (e.g., rigid offsets at member ends) is not considered in the analysis. However, the effect of shear deformation is considered. Detailed design of the beams along the grid line '2' as per recommendations of IS 13920:1993 has been carried out.

Solution:

1.1 Preliminary Data

Plan of the building and sectional elevations of different RC frames are shown in Figures 1.1, 1.2 and 1.3. The sizes of the beams and columns are given in Table 1.1. Figure 1.4 shows beam-loading diagram for dead load and live load, respectively, on an intermediate frame in the transverse direction.

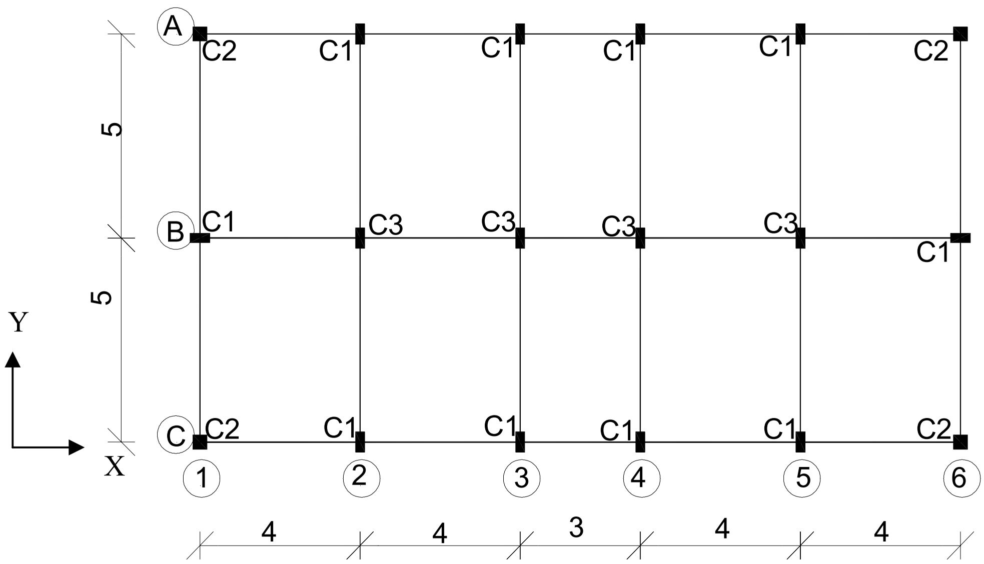


Figure 1.1: Plan of building (All dimensions in meters)

Table 1.1 :Schedule of member sizes

Column		Beam	
C1	300 x 500	RB1, FB1	300 x 600
C2	400 x 400	RB2, FB2	300 x 500
C3	400 x 500	PB1	300 x 400
		PB2	300 x 350
Slab thickness: 125			

Note: All dimensions in mm.

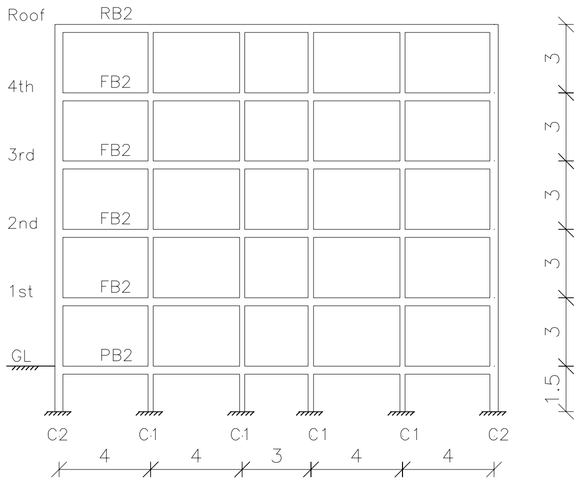


Figure 1.2: Elevation of frame A, B & C

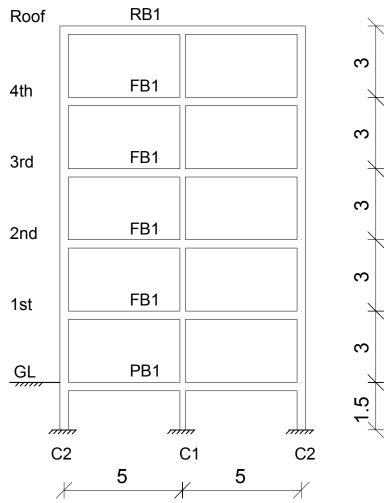
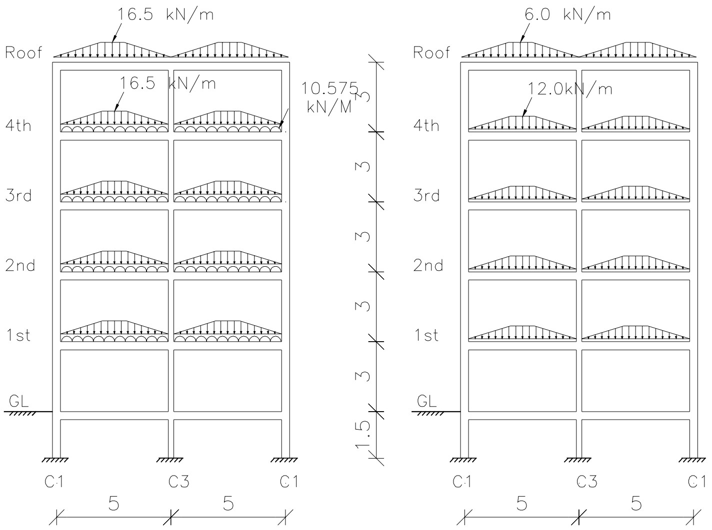


Figure 1.3: Elevation of transverse frame 1&6



a. Dead Load b. Live Load

Figure 1.4: Loading diagram for an intermediate frame 2-5

1.2 General

Other relevant data are as follows:

Grade of concrete: M20

Grade of steel = Fe 415

Live load on roof = 1.5 kN/m^2 (Nil for earthquake)

Live load on floors = 3 kN/m^2 (25% for earthquake)

Roof finish = 1 kN/m^2

Floor finish = 1 kN/m^2

Brick wall on peripheral beams = 230 mm thick

Brick wall on internal beams = 150 mm thick

Density of concrete = 25 kN/m^3

Density of brick wall including plaster = 20 kN/m^3

1.3 Load Combinations

Load combinations are considered as per IS 456: 2000 and are given in Table 1.2. EQX implies earthquake loading in X direction and EQY stands for earthquake loading in Y direction.

The emphasis here is on showing typical calculations for ductile design and detailing of building elements subjected to earthquakes. In practice, wind load should also be considered in lieu of earthquake load and the critical of the two load cases should be used for design.

Beams parallel to the Y direction are not significantly affected by earthquake force in the X direction (except in case of highly unsymmetrical buildings), and vice versa. Beams parallel to Y direction are designed for earthquake loading in Y direction only. Torsion effect is not considered in this example.

Table 1.2: Load combinations for earthquake loading

S.No.	Load Combination	DL	LL	EQ
1	1.5DL+1.5LL	1.5	1.5	-
2	1.2(DL+LL*+EQX)	1.2	0.25/0.5*	+1.2
3	1.2(DL+LL*-EQX)	1.2	0.25/0.5*	-1.2
4	1.2(DL+LL*+EQY)	1.2	0.25/0.5*	+1.2
5	1.2(DL+LL*-EQY)	1.2	0.25/0.5*	-1.2
6	1.5(DL+EQX)	1.5	-	+1.5
7	1.5(DL-EQX)	1.5	-	-1.5
8	1.5(DL+EQY)	1.5	-	+1.5
9	1.5(DL-EQY)	1.5	-	-1.5
10	0.9DL+1.5 EQX	0.9	-	+1.5
11	0.9DL-1.5 EQX	0.9	-	-1.5
12	0.9DL+1.5 EQY	0.9	-	+1.5
13	0.9DL-1.5 EQY	0.9	-	-1.5

*Note: Reduced Live loads are considered as per Clause 7.3.1 of IS 1893 (Part 1): 2002, even though it is proposed to drop this clause in the new edition of the Code. For the present case, (live load of 3 kN/m^2) 25% of live load is considered for seismic weight calculations.

1.4 Design of Middle Floor Beam

Beam marked ABC in Figure 1.5 for frame 2 is considered for design. Since the beam consists of two symmetrical spans, calculations need to be performed for one span only.

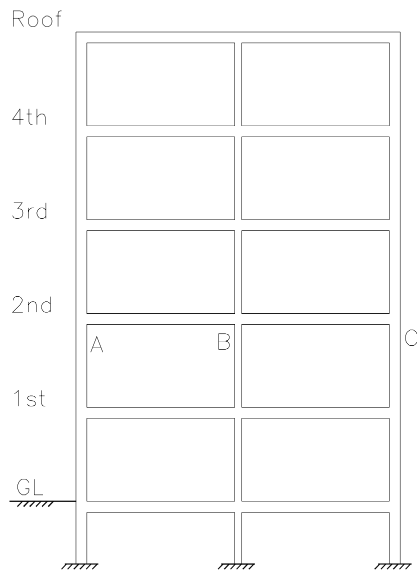


Figure 1.5: Beam ABC

1.5 Member Forces

For the beam AB, force resultants for various load cases and load combinations have been obtained from computer analysis and are summarised in Table 1.3 and Table 1.4 which show force resultants for different load combinations; with the maximum values to be used for design being underlined.

As the beam under consideration is parallel to Y direction, earthquake loads in Y direction are predominant and hence the 13 load combinations of Table 1.2 reduce to 7 as shown in Table 1.4

Table 1.3 : Force resultants in beam AB for various load cases

Load Case	Left end		Centre		Right end	
	Shear (kN)	Moment (kN-m)	Shear (kN)	Moment (kN-m)	Shear (kN)	Moment (kN-m)
DL	-51	-37	4	32	59	-56
LL	-14	-12	1	11	16	-16
EQY	79	209	79	11	79	-191

Note: The results are rounded of to the next higher integer value.

Table 1.4 Force resultants in beam AB for different load combinations

S. No.	Load Combination	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	-98	-74	8	65	113	-108
2	1.2(DL+LL*+EQY)	29	203	100	55	170	-301
3	1.2(DL+LL*-EQY)	-160	-299	-90	29	-19	157
4	1.5(DL+EQY)	42	258	**125**	**65**	**207**	**-371**
5	1.5(DL-EQY)	**-195**	**-369**	-113	32	-30	203
6	0.9DL+1.5 EQY	73	**280**	122	45	172	-337
7	0.9DL-1.5 EQY	-164	-347	-115	12	-65	**236**

* ***Appropriate fraction of live load has been used***

1.6 Various Checks

1.6.1 Check for Axial Stress

Factored axial force = 0.00 kN

Factored axial stress = 0.0 MPa < 0.10 f_{ck}

Hence, design as flexural member.

(Clause 6.1.1; IS 13920:1993)

1.6.2 Check for Member Size

Width of beam, B = 300 mm > 200 mm,

Hence, ok (Clause 6.1.3; IS 13920:1993)

Depth of beam, D = 600 mm

$$\frac{B}{D} = \frac{300}{600} = 0.5 \quad > 0.3, \text{ hence ok}$$

(Clause 6.1.2; IS 13920:1993)

Span, L = 5,000 mm

$$\frac{L}{D} = \frac{5{,}000}{600} = 8.33 > 4, \text{ hence ok}$$

(Clause 6.1.4 of IS: 13920-1993)

1.6.3 Check for Limiting Longitudinal Reinforcement

Effective depth for moderate exposure conditions with 20 mm diameter bars in two layers on an average

= 600 – 30 – 8 – 20 – (20/2)

= 532 mm.

Minimum reinforcement

$$= 0.24\frac{\sqrt{f_{ck}}}{f_y} = \frac{0.24 \times \sqrt{20}}{415}$$

= 0.26%.

= 0.26 x 300 x 532/100

= 415 mm^2

(Clause 6.2.1(b) of IS 13920: 1993)

Maximum reinforcement

= 2.5%

= 2.5 x 300 x 532 /100

= 3,990 mm^2

(Clause 6.2.2 of IS 13920: 1993)

1.7 Design for Flexure

Table 1.5 shows, in brief, the reinforcement calculations at left end, centre and right end of the beam AB as per IS 13920:1993. Design aid SP: 16 has been used for this purpose. Detailed calculations at left end are given in the following sections. In actual practice, a spread sheet can be used conveniently.

1.7.1 Design for Hogging Moment

M_u = 369 kN-m

$$\frac{M_u}{bd^2} = \frac{369 \times 10^6}{300 \times 532 \times 532} = 4.35$$

Referring to Table-50 of SP: 16,

For d'/d = 68 / 532 = 0.13, we get

A_{st} at top = 1.46 %

= 1.46 x 300 x 532 /100

= 2,330 mm^2

> Minimum reinforcement (415 mm^2) < Maximum reinforcement (3,990 mm^2)

A_{sc} at bottom = 0.54 %

But A_{sc} must be at least 50% of A_{st}, hence, revise to 1.46/2 = 0.73 %

(Clause 6.2.3 of IS: 13920-1993)

Hence, A_{sc} at bottom

= 0.73 x 300 x 532 /100

= 1,165 mm^2

1.7.2 Design for Sagging Moment

M_u = 280 kN-m

The beam is designed as T beam. The limiting capacity of the T-beam assuming $x_u < D_f$ and $x_u < x_{u,max}$ may be calculated as follows.

$$M_u = 0.87\, f_y\, A_{st}\, d(1 - \frac{A_{st}\, f_y}{b_f d\, f_{ck}})$$

(Annex G of IS 456: 2000)

Where,

D_f = depth of flange

= 125 mm

x_u = depth of neutral axis

$x_{u,max}$ = limiting value of neutral axis

= 0.48 x d

= 0.48 x 532

= 255 mm

b_w = width of rib

= 300 mm

b_f = width of flange

$$= \frac{L_o}{6} + bw + 6d_f \text{ or c/c of beams}$$

$$= \frac{0.7 \times 5000}{6} + 300 + 6 \times 125 \text{ Or 4,000}$$

= 1,633 mm or 4,000 mm

= 1,633 mm (lowest of the above)

(Clause 23.1.2 of IS 456: 2000)

Substituting the values and solving the quadratic equation, we get

A_{st} at bottom = 1,512mm^2 > 415 mm^2

< 3,990 mm^2

It is necessary to check the design assumptions before finalizing the reinforcement.

$$x_u = \frac{0.87\, f_y\, A_{st}}{0.36\, f_{ck}\, b_f}$$

$$= \frac{0.87 \times 415 \times 1512}{0.36 \times 20 \times 1633} = 47.44\, mm$$

$< d_f$ ok

$< x_{u,max}$ i.e. < 255 mm ok

A_{sc} at top = not required.

But A_{sc} must be at least 50% of A_{st} hence,

revise to 1,512 /2 = 756 mm^2

(Clause 6.2.3 of IS 13920: 1993)

1.7.3 Required Reinforcement

Top reinforcement required is larger of 2,330 mm^2 and 756 m^2. Hence, provide 2,330 mm^2.

Bottom reinforcement required is larger of 1,165 mm^2 and 1,512 mm^2. Hence, provide 1,512 mm^2.

1.8 Details of Reinforcement

Table 1.6 shows summary of reinforcement provided at left end, at centre, and at right end of the beam AB.

A total of 3-16Φ straight bars each are provided throughout the length of the beam at both top and bottom. 5-20Φ+1-16Φ extra at top (i.e., a total of 1.487 %) and 3-20Φ extra at bottom (i.e., a total of 0.97%) are provided at the left end. At the right end, i.e., over the central support, 5-20Φ + 1-16Φ extra at top (i.e. a total of 1.487%) and 1-20Φ + 2-16Φ extra at bottom (i.e. a total of 0.83%) bars are provided.

In an external joint, both the top and bottom bars of the beam shall be provided with an anchorage length beyond the inner face of the column equal to the development length in tension + 10 times bar diameter minus the allowance for 90 degree bend (Clause 6.2.5 of IS 13920:1993) as shown in Figure 1.6.

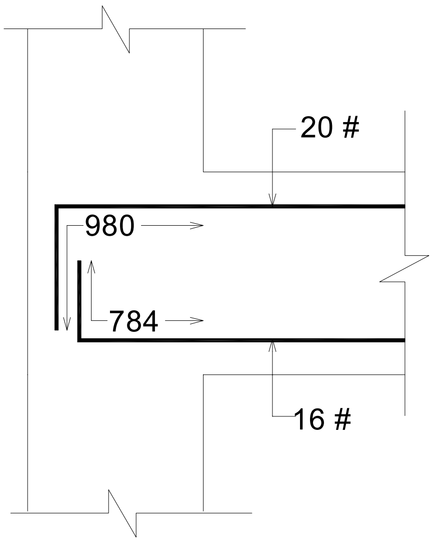


Figure 1.6: Anchorage of reinforcement bars in an external joint

In this case, for Fe415 steel and M20 grade concrete, from Table 65 of SP: 16,
l_d = 47 Φ + 10 Φ - 8 Φ = 49 Φ
= 980 mm for 20 Φ bar
= 784 mm for 16 Φ bar

1.9 Design for Shear

1.9.1 Design Shear Strength of Concrete

Tensile steel provided at left end = 1.487%

Permissible design shear stress of concrete,

τ_c = 0.715 MPa (IS 456:2000 Table 19)

Design shear strength of concrete

= $\tau_c\, b\, d$

= 0.715 x 300 x 532 /1,000

= 114 kN

Similarly, design shear strength of concrete at center and right end is evaluated as 69 kN and 114 kN, respectively.

1.9.2 Shear Force Due to Plastic Hinge Formation at the ends of the 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

$$V_{swaytoright} = \frac{\pm 1.4\,(M_u^{As} + M_u^{Bh})}{L}$$

$$V_{swaytoleft} = \frac{\pm 1.4\,(M_u^{Ah} + M_u^{Bs})}{L}$$

The sagging and hogging moments of resistance (M_u^{As}, M_u^{Bs}, M_u^{Ah} and Mu^{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 2,374 mm^2 (i.e., p_t =1.487 %) at top and 1,545 mm^2 (i.e. p_c = 0.97%) at bottom on the left end of the beam.

For p_t = 1.487% and p_c = 0.97%, referring to Table 50 of SP: 16, (for p_t = 1.487% or p_c = 0.97% whichever gives lowest value in the table),

$$\frac{M_u^{Ah}}{bd^2} = 4.44$$

Hogging moment capacity at A,

M_u^{Ah} = 4.44 x 300 x (532)^2/(1 x 10^6) = 377 kN-m

The limiting moment carrying capacity of a beam section can also be evaluated from the first principle. This method is iterative but gives more appropriate values of M_u.

For calculation of M_u^{As}, the tensile steel p_t = 0.97% and compressive steel p_c = 1.487% is used. The contribution of the compressive steel is ignored while calculating the sagging moment capacity as T-beam. Referring to Annex G of IS: 456-2000, sagging moment capacity at A for $x_u < D_f$ and $x_u < x_{u,max}$ may be calculated as given below.

$$M_u^{As} = M_u = 0.87\, f_y\, A_{st}\, d\left(1 - \frac{A_{st}\, f_y}{b_f d\, f_{ck}}\right)$$

= 286 kN-m

Table 1.5: Flexural design for beam AB

Beam AB	Top reinforcement		
	Left end	Center	Right end
Hogging moment (kN-m)	-369	-	-371
$-M_u/bd^2$	4.35	-	4.37
A_{st} at top	**1.46**%	-	**1.47**%
A_{sc} at bottom	0.54% < 1.46/2 Hence revise to 0.73% (Clause 6.2.3; IS13920: 1993)	-	0.55% < 1.47/2 Hence revise to 0.7335% (Clause 6.2.3; IS13920: 1993)
	Bottom reinforcement		
Sagging moment (kN-m)	280	65	236
A_{st} at bottom	A_{st} required = 1512 mm^2 = 0.945% > 1.46/2 i.e. 0.73 ok.	A_{st} required = 335 mm^2 = 0.21% < 0.26% < 1.47 /4 = 0.37 %, Hence revise to **0.37%** (Clause 6.2.1(b) and 6.2.4 of IS13920: 1993)	A_{st} required = 1264 mm^2 = 0.79 % > 1.47/2 > 0.735 % ok.
A_{sc} at top	0.33/2 = 0.165 % < 0.26% < 1.47/4=0.37% Hence, revise to 0.37%.	0.37/2 = 0.185 % < 1.47/4=0.37% Hence, revise to 0.37%.	0.79/2 = 0.395% > 0.26% > 1.47/4=0.37% ok
	Summary of required reinforcement		
	Top = 1.46% Bottom = 0.945%	Top = 0.37% Bottom = 0.37%	Top = 1.47% Bottom = 0.79%

Table 1.6 Summary of reinforcement for beam AB

Beam AB	Longitudinal Reinforcement		
	Left end	Center	Right end
Top reinforcement	3-16Φ straight + 5-20Φ +1-16Φ extra Steel Provided = 2,374 mm^2 i.e. 1.487%	3-16Φ straight Steel Provided = 603 mm^2 i.e. 0.378%	3-16Φ straight + 5-20Φ +1-16Φ extra Steel Provided = 2,374 mm^2 i.e. 1.487%
Bottom reinforcement	3-16Φ straight + 3-20Φ extra Steel Provided = 1,545 mm^2 i.e. 0.97%	3-16Φ straight Steel Provided = 603 mm^2 i.e. 0.378%	3-16Φ straight + (2-16Φ+1-20ϕ) extra Steel Provided =1,319 mm^2 i.e. 0.83%

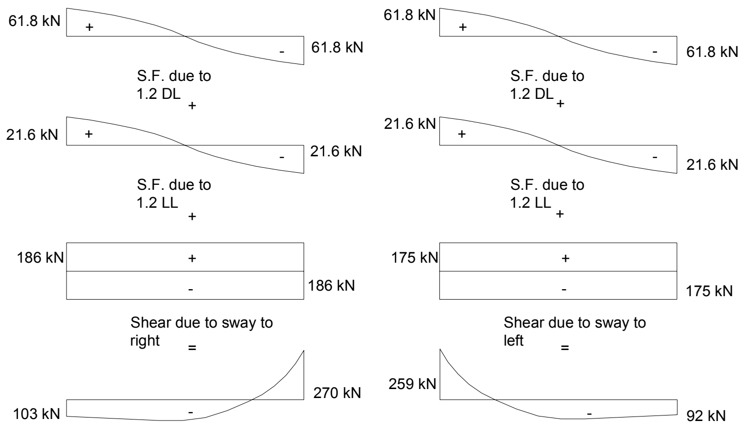


Figure 1.7: Shear diagram

Similarly, for the right end of the beam we obtain, M_u^{Bh} = 377 kN-m and M_u^{Bs} = 246 kN-m,

Shear is calculated as below:

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

= ±1.4 (286 + 377) / 5

= ±186 kN

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

= ±1.4(377 + 246)/5

= ±175 kN

1.9.3 Design Shear

Referring to the dead and live load diagrams (Figure 1.4),

DL = Trapezoidal dead load + Wall and self load

= 16.5 x (1 + 5) /2 + 10.575 x 5

= 103 kN

LL = 12 x (1 + 5) / 2 = 36 kN

Figure 1.7 shows the shear force diagram due to DL, LL and due to hinge formation at the ends of beam.

Shear at left end for sway to right,

$$V_{u,a} = \frac{1.2(DL + LL)}{2} - \frac{1.4(M_u^{As} + M_u^{Bh})}{L}$$

= 1.2 x (103 + 36) /2 - 186

= 103 kN

Shear at left end for sway to left,

$$V_{u,a} = \frac{1.2(DL + LL)}{2} + \frac{1.4(M_u^{Ah} + M_u^{Bs})}{L}$$

= 1.2 x (103 + 36) /2 + 175

= 259 kN

Shear at right end for sway to right,

$$V_{u,b} = \frac{1.2(DL + LL)}{2} + \frac{1.4(M_u^{As} + M_u^{Bh})}{L}$$

= 1.2 x (103 +36) /2 + 186

= 270 kN

Shear at right end for sway to left,

$$V_{u,b} = \frac{1.2(DL + LL)}{2} - \frac{1.4(M_u^{Ah} + M_u^{Bs})}{L}$$

= 1.2 x (103 +36) /2 - 175

= 92 kN

Figure 1.7 shows the shear force diagram for the beam considering plastic hinge formation at ends.

As per Clause 6.3.3 of IS 13920:1993, the design shear force to be resisted shall be the maximum of:

i) Calculated factored shear forces as per analysis (Refer Table 1.4)

ii) Shear forces due to formation of plastic hinges at both ends of the beam plus factored gravity load on the span (as calculated in Section 1.9.3)

Hence, design shear force (V_u) will be 259 kN (maximum of 195 kN from analysis and 259 kN corresponding to hinge formation) for left end of the beam and 270 kN (maximum of 207 kN and 270 kN) for the right end.

From analysis, the shear at the mid-span of the beam is 125 kN. However, shear due to formulation of plastic hinges at both the ends of the beams has been calculated as 186 kN and 175 kN. Hence, the design shear at centre of the span is taken as 186 kN.

The required capacity of shear reinforcement at the left end of the beam is:

$V_{us} = V_u - V_c$

$= 259\text{-}114$

$= 145$ kN

Similarly the, required capacity of shear reinforcement at the right end and at mid-span is 156 and 117 kN, respectively.

Referring to Table 62 of SP:16, we get the required spacing of 2 legged 8ϕ stirrups as 145 mm, 165 mm and 135 mm respectively at left end, centre and right end.

As per Clause 6.3.5 of IS 13920:1993, 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:

$S_v = A_{sv} \times 0.87 f_y /(0.4\ b)$

$= 2 \times 50 \times 0.87 \times 415 / (300 \times 0.4)$

$= 300$ mm.

Spacing of links over a length of 2d at either end of beam as per Clause 6.3.5 of IS13920: 1993 shall be the least of:

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

ii) 8 times diameter of smallest bar

$= 8 \times 16 = 128$ mm

However, it need not be less than 100 mm.

Hence, provide 2 Legged - 8 ϕ stirrups @125mm c/c at left and at right end over a length of $2d$ = 2 x 532 = 1,064 mm.

Elsewhere, provide stirrups at 165 mm centers.

In case of splicing of reinforcement, the spacing of links shall be limited to 150 mm centers as per clause 6.2.6 of IS 13920:1993.

The reinforcement detailing is shown in Figure1.8.

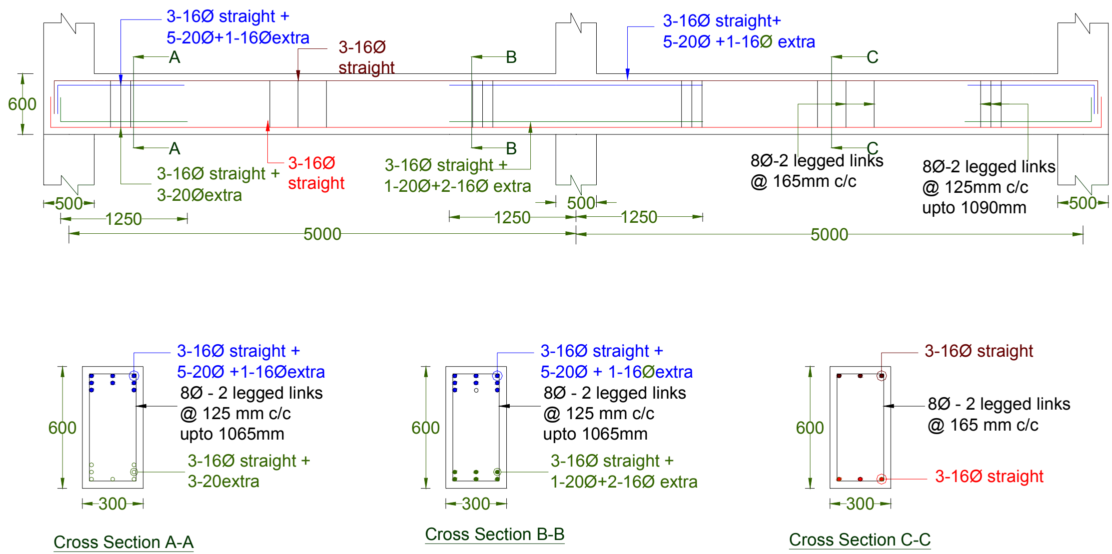


Figure 1.8: Reinforcement details for the beam ABC

1.10 Impact of Ductile Detailing on Bill of Quantities

To compare the impact of ductile detailing (as per IS 13920:1993) on the bill of quantities, the beam under consideration has been redesigned as follows:

a) Design and detailing as per IS 456:2000; seismic forces are the same as computed earlier, i.e, with response reduction factor R = 5.0. The reinforcement details are shown in Figure 1.9.

b) Design and detailing as per IS 456:2000; seismic forces increased by a factor of 5/3 to account for R = 3.0. The reinforcement details are shown in Figure 1.10.

Table 1.7 compares the quantity of reinforcement for the three cases. For the purpose of comparison, only the steel between c/c of columns is considered.

Table 1.7 Comparison of bill of quantities for steel in the beam ABC

Description	Detailing as per IS 13920: 1993		Detailing as per IS 456: 2000 (Seismic loads as per R = 5)		Detailing as per IS 456:2000 (Seismic loads as per R = 3)	
Steel required in kg	Longitudinal	Transverse	Longitudinal	Transverse	Longitudinal	Transverse
	95	25	93	14	135	28
Total steel in kg	120		107		163	
Ratio	1.0		0.89		1.36	

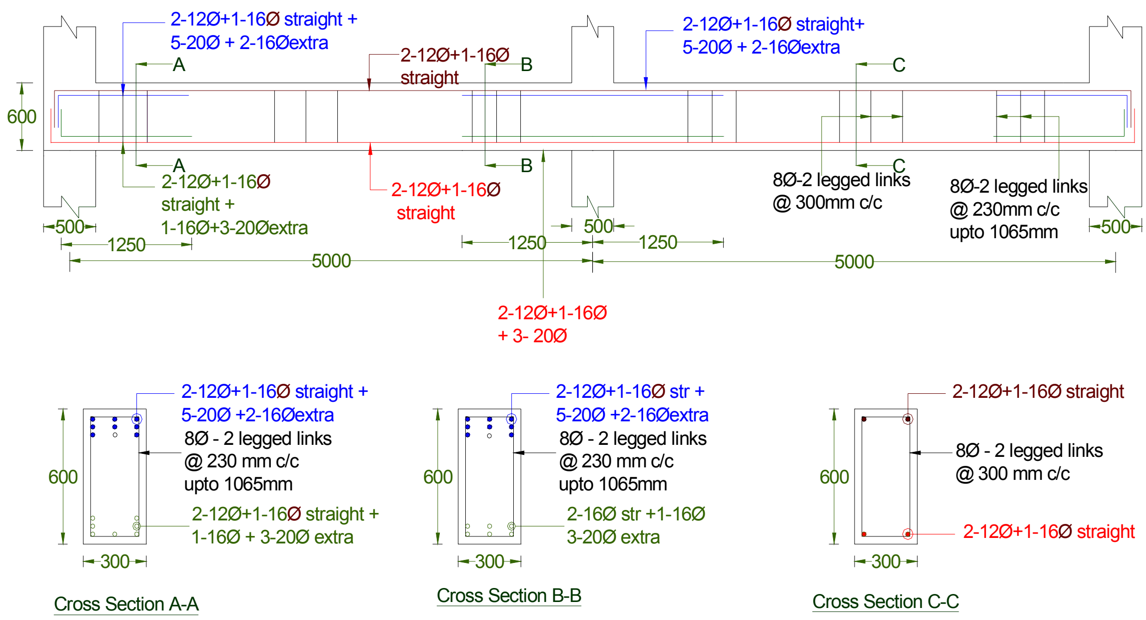


Figure 1.9: Reinforcement details for the beam ABC as per IS 456:2000 (with R = 5)

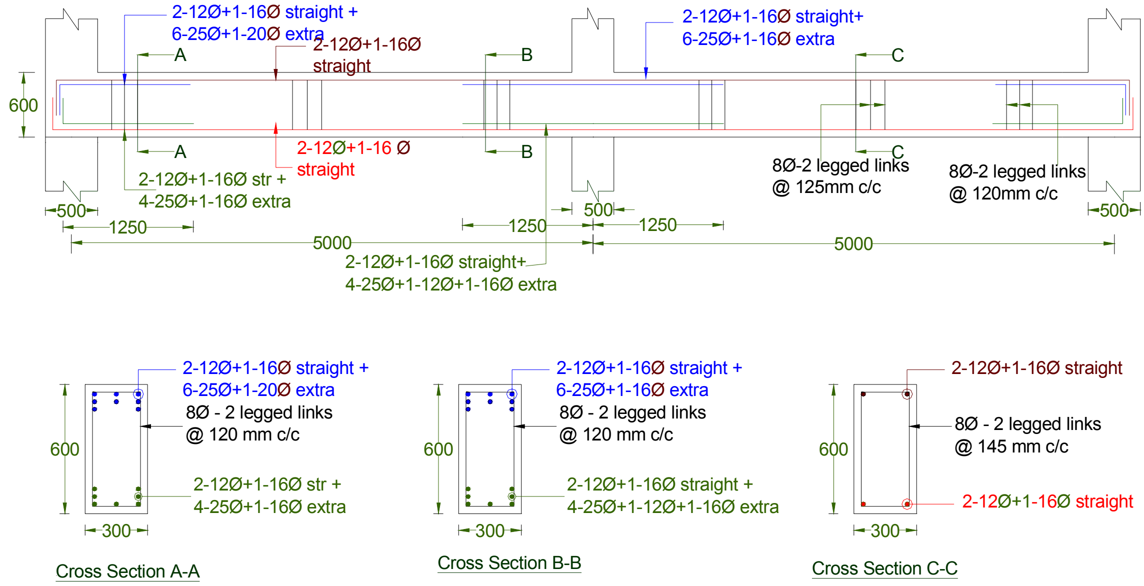


Figure 1.10: Reinforcement details for the beam ABC as per IS 456:2000 (with *R* = 3)

Effect of Finite Size Correction

As mentioned in the problem statement, the effect of finite size joint corrections (i.e., rigid offsets at member ends) has been ignored in the analysis. In case, the designer wishes to take advantage of the finite size joint correction, care shall be taken to model the same in the static analysis. The results with finite size joint widths in the analysis are presented in Table 1.3a. The results without and with finite size corrections can be compared from Tables 1.3 and 1.3a, respectively. However, in the detailed calculations shown in this example, this correction has been ignored.

Table 1.3a Force resultants in the beam AB for various load cases with Finite Size Correction

Load Case	Left end		Center		Right end	
	Shear (kN)	Moment (kN-m)	Shear (kN)	Moment (kN-m)	Shear (kN)	Moment (kN-m)
DL	-48	-29	4	28	55	-45
LL	-14	-10	0	10	16	-13
EQY	83	191	83	8	83	177

Note: The results are rounded of to the next integer value.

Example2 - Beam Design of an RC Frame Building in Seismic Zone II

2 Problem Statement:

The ground plus four storey RC office building of Example-1 (Refer Figures 1.1-1.4) is assumed to be located in seismic zone II on medium soil. The dead load and live loads are the same as in Example-1. However, the earthquake loads are much lower for zone-II. Hence, reduced member sizes are considered as shown in Table 2.1. The design of a beam along grid line 2, as per recommendations of IS13920:1993, is explained.

Solution

Design of Middle Floor Beam

The beam marked ABC in Figure 2.1 for frame 2 (Figure 1.1 of Example 1) is considered for design. Since the beam consists of two symmetrical spans, calculations are performed for one span only.

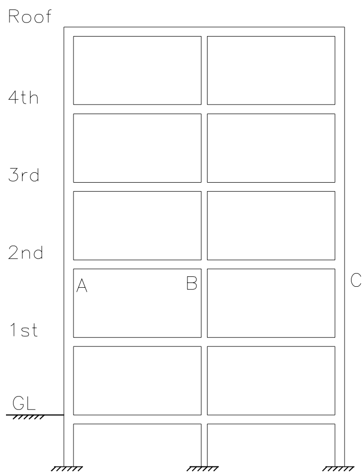


Figure 2.1 Beam ABC

Table 2.1 Schedule of member sizes

Column		Beam	
C1	230 x 500	RB1, FB1	230 x 500
C2	350 x 350	RB2, FB2	230 x 400
C3	300 x 500	PB1	230 x 350
		PB2	300 x 300
Slab Thickness: 125			

Note: All dimensions in mm

2.1 Member Forces

For beam AB, force resultants for various load cases and load combinations have been obtained from computer analysis and are summarized in Table 2.2. Table 2.3 shows force resultants for different load combinations with the maximum values to be used for design being underlined.

Table 2.2 Force resultants in beam AB for different load cases

Load Case	Left end		Centre		Right end	
	V kN	*M* kN-m	*V* kN	*M* kN-m	*V* kN	*M* kN-m
DL	-48	-39	2	29	53	-50
LL	-15	-14	0	10	15	-16
EQY	22	59	22	4	22	-50

Note: V = Shear; M = Moment, The results are rounded of to the next higher integer value.

2.2 Various Checks

2.2.1 Check for Axial Stress

Factored axial force = 0.00 kN

Factored axial stress = 0.0 MPa < 0.10 f_{ck}

Hence, design as flexural member.

(Clause, 6.1.1; IS 13920:1993)

Table 2.3 Force resultants in beam AB for different load combinations

S. No.	Load Combination	Left end		Centre		Right end	
1		Shear (kN)	Moment (kN-m)	Shear (kN)	Moment (kN-m)	Shear (kN)	Moment (kN-m)
2	1.5DL+1.5LL	-95	-80	3	**59**	102	-99
3	1.2(DL+LL*+EQY)	-36	20	29	43	95	-125
4	1.2(DL+LL*-EQY)	-89	-122	-24	33	42	-5
5	1.5(DL+EQY)	-39	30	**36**	50	**113**	**-150**
6	1.5(DL-EQY)	**-105**	**-147**	-30	38	47	0
7	0.9DL+1.5 EQY	-10	**53**	35	32	81	-120
8	0.9DL-1.5 EQY	-76	-124	-31	20	15	**30**

* Appropriate fraction of live load has been used

2.2.2 Check for Member Size

Width of beam, B = 230 mm > 200 mm

Hence, ok.

(Clause 6.1.3; IS 13920:1993)

Depth of beam, D = 500 mm

$$\frac{B}{D} = \frac{230}{500} = 0.46 \quad > 0.3$$

Hence, ok.

(Clause 6.1.2; IS 13920:1993)

Span, L = 5,000 mm

$$\frac{L}{D} = \frac{5{,}000}{500} = 10 \quad > 4$$

Hence, ok.

(Clause 6.1.4 of IS 13920:1993)

2.2.3 Check for Limiting Longitudinal Reinforcement

Effective depth for moderate exposure condition with 16 mm diameter bar in two layers on an average = 500 – 30 – 16 – (16/2) – 8 = 438 mm.

Minimum reinforcement,

$$= \ 0.24\frac{\sqrt{f_{ck}}}{f_y} = \frac{0.24x\sqrt{20}}{415}$$

= 0.26%.

= 0.26 x 230 x 438/100

= 262 mm^2

(Clause 6.2.1(b) of IS13920: 1993)

Maximum reinforcement

= 2.5%

= 2.5 x 230 x 438/100

= 2,518 mm^2

(Clause 6.2.2 of IS 13920:1993)

2.3 Design for Flexure

Table 2.4 shows, in brief, the reinforcement calculations at left end, centre and right end as per IS 13920:1993. Design aid SP: 16 has been used for the purpose. Detailed calculations at left end are given in the following sections. In actual practice, a spread sheet can be used conveniently.

2.3.1 Design for Hogging Moment

M_u = 147 kN-m

$$\frac{M_u}{bd^2} = \frac{147 \times 10^6}{230 \times 438 \times 438} = 3.33$$

Referring to Table-50 of SP: 16

For d'/d = 62/446 = 0.14 and interpolating between d'/d of 0.10 and 0.15, we get

A_{st} at top = 1.132%

= 1.132 x 230 x 438/100 = 1,140 mm^2

> Minimum reinforcement (262 mm^2) < Maximum reinforcement (2,518 mm^2)

A_{sc} at bottom = 0.19 %

But A_{sc} must be at least 50% of A_{st}.

Hence, revise to 1.132 / 2 = 0.566 %

(Clause 6.2.3 of IS 13920:1993)

Hence, A_{sc} at bottom

= 0.566 x 230 x 438 /100

= 571 mm^2

2.3.2 Design for Sagging Moment

M_u = 53 kN-m

The beam is designed as T beam. The limiting capacity of the T-beam assuming $x_u < D_f$ and $x_u < x_{u,max}$ may be calculated as given below.

$$M_u = 0.87\, f_y\, A_{st}\, d(1 - \frac{A_{st}\, f_y}{b_f d\, f_{ck}}) \qquad \text{------- (i)}$$

(Annex G of IS 456: 2000)

Where,

D_f = depth of flange

= 125 mm

x_u = depth of neutral axis

$x_{u,max}$ = limiting value of neutral axis

= 0.48 x d

= 0.48 x 438

= 210 mm

b_w = width of rib

= 230 mm

b_f = width of flange

$= \frac{L_o}{6} + b_w + 6d_f$ or c/c of beams

$= \frac{0.7 \times 5{,}000}{6} + 230 + 6 \times 125$ or 4,000 ,

whichever is less

= 1,563 mm or 4,000 mm

= 1,563 mm (lower of the above)

(Clause 23.1.2 of IS 456: 2000)

Substituting the relevant values in (i) and solving the resulting quadratic equation, we get

A_{st} at bottom = 339 mm^2 > 262 mm^2

< 2,518 mm^2

It is necessary to check the design assumptions before finalizing the reinforcement.

$$x_u = \frac{0.87\, f_y\, A_{st}}{0.36\, f_{ck}\, b_f}$$

$$= \frac{0.87 \times 415 \times 339}{0.36 \times 20 \times 1{,}563} = 10.88 \text{ mm}$$

$< d_f$ ok.

$< x_{u,max}$ i.e. 210 mm ok.

A_{sc} at top = not required.

But A_{sc} must be at least 50 % of A_{st}, hence,

revise to 339 /2 = 170 mm^2

(Clause 6.2.3 of IS 13920: 1993)

A_{st} at bottom = 339 mm^2

= 339 x 230 x 438 /100

= 0.33 % > 0.26 %

< 4%

Hence, ok.

2.3.3 Required reinforcement

Top reinforcement required is the larger of 1,132 mm^2 and 170 mm^2. Hence, provide 1,132 mm^2.

Bottom reinforcement required is the larger of 339 mm^2 and 571 mm^2. Hence, provide 571 mm^2.

2.4 Details of Reinforcement

Table 2.5 show a summary of reinforcement provided at the left end, at center and at the right end of the beam AB.

3-12Φ straight bars are provided throughout the length of the beam at the top and 4-12Φ straight bars are provided throughout at the bottom. 4-16Φ +1-12Φ extra bars at the top and 1-12Φ extra bar at the bottom at the left end are also provided. At the right end, i.e., over the central support, 4-16Φ +1-12Φ extra bars at the top and 2-12Φ extra bottom bars are provided.

At an external joint, as per Clause 6.2.5 of IS 13920:1993, both the top and bottom bars of the beam shall be provided with an anchorage length beyond the inner face of the column equal to the development length in tension + 10 times the bar diameter minus the allowance for 90 degree bend. (Refer Figure 2.2)

Table 2.4 Flexural design for beam AB

Beam AB	**Top reinforcement**		
	Left end	Center	Right end
Hogging moment (kN-m)	-147	-	-150
$-M_u/bd^2$	3.33	-	3.4
A_{st} required at top	**1.132**%	-	**1.163**%
A_{sc} required at bottom	0.19% < 1.132/2 = 0.566% Hence revise to 0.566% (Clause 6.2.3; IS13920: 1993)	-	0.224% < 1.163/2 = 0.582% Hence revise to 0.582% (Clause 6.2.3; IS 13920:1993)
Bottom reinforcement			
Sagging moment (kN-m)	53	58	30
A_{st} at bottom	A_{st} required = 339 mm^2 = **0.33%**	A_{st} required = 371 mm^2 = 0.37% > 0.26 > 1.163/4 = 0.291% ok	A_{st} required = 192 mm^2 =0.16 % < 0.26% <1.163/2 = 0.582% Hence revise to 0.582%. (Clause 6.2.3; IS13920: 1993)
A_{sc} at top	0.33/2 = 0.165% < 0.26% < 1.163/4=0.291% Hence, revise to 0.291%.	0.37 / 2 = 0.185% < 0.26% < 1.163/4 = 0.291 % Hence, revise to 0.291%.	0.582 /2 = 0.291% > 0.26% ok.
	Summary of required reinforcement		
	Top = 1.132% Bottom = 0.566%	Top = 0.291% Bottom = 0.37 %	Top = 1.163 % Bottom = 0.582%

Table 2.5 Summary of reinforcement provided for the beam AB

Beam AB	**Longitudinal reinforcement**		
	Left end	Center	Right end
Top reinforcement	3-12Φ straight + 4-16Φ extra Steel Provided = 1,143 mm^2 i.e. 1.134%	3-12Φ straight Steel Provided = 339 mm^2 i.e. 0.33%	3-12Φ straight + 4-16Φ +1-12Φ) extra Steel Provided = 1,256 mm^2 i.e. 1.246%
Bottom reinforcement	4-12Φ straight + 2-10Φ extra Steel Provided = 609 mm^2 i.e. 0.6%	4-12Φ straight Steel Provided = 452 mm^{2}i.e. 0.44%	4-12Φ str + 2-10Φ extra Steel Provided = 609 mm^2 i.e. 0.6%

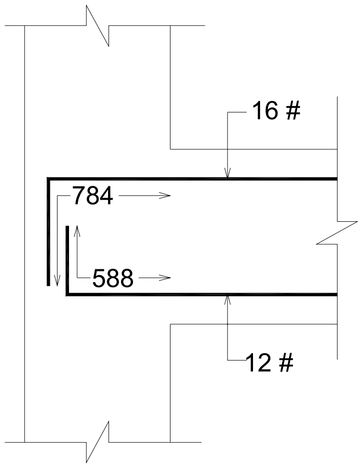


Figure 2.2 Anchorage of beam bars in an external joint

In this case, for Fe 415 steel and M20 grade concrete, from Table 65 of SP: 16,

$l_d = 47\ \Phi + 10\ \Phi - 8\ \Phi = 49\ \Phi$

$= 784$ mm for 16 Φ bar

$= 588$ mm for 12 Φ bar

2.5 Design for Shear

2.5.1 Design Shear Strength of Concrete

Tensile steel provided at left end = 1.134%

Permissible design shear stress of concrete,

$\tau_c = 0.66$ MPa (Table 19 of IS 456:2000)

Design shear strength of concrete

$= \tau_c\, b\, d$

$= 0.66 \times 230 \times 438 / 1{,}000$

$= 66$ kN

Similarly, the design shear strength of concrete at mid-span and at the right end is evaluated as 46 kN and 66 kN, respectively.

2.5.2 Shear Force Due to Plastic Hinge Formation at the ends of the Beam

The additional shear due to formation of plastic hinges at both ends of the beams is evaluated as per clause 6.3 of IS 13920:1993 and is given by

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

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

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

The beam is provided with a steel area of 1,143 mm^2 (i.e., p_t = 1.134%) at top and 609 mm^2 (i.e., p_c = 0.60%) at bottom on the left end of the beam.

For p_t = 1.11% and p_c = 0.60%, referring to Table 50 SP: 16(for p_t = 1.134% or p_c = 0.60% whichever gives lowest value in the table),

$$\frac{M_u^{Ah}}{bd^2} = 3.36$$

Hogging moment capacity at A,

$M_u^{Ah} = 3.36 \times 230 \times 438 \times 438 / 10^6$

$= 149$ kN-m

For calculation of M_u^{As}, the tensile steel p_t = 0.60% and compressive steel p_c = 1.134% is used. The contribution of the compression steel is ignored while calculating the sagging moment capacity as T-beam. Referring to Annex G of IS: 456-2000, sagging moment capacity at A for $x_u < D_f$ and $x_u < x_{u,max}$ may be calculated as given below.

$$M_u^{As} = M_u = 0.87\, f_y\, A_{st}\, d\left(1 - \frac{A_{st}\, f_y}{b_f d\, f_{ck}}\right)$$

$= 94$ kN-m

Similarly, for the right side joint we obtain,

$M_u^{Bh} = 165$ kN-m and $M_u^{Bs} = 94$ kN-m.

Shear is calculated as below:

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

$= \pm 1.4(94 + 165) / 5$

$= \pm 72$kN

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

$= \pm 1.4(149 + 94) / 5$

$= \pm 68$ kN

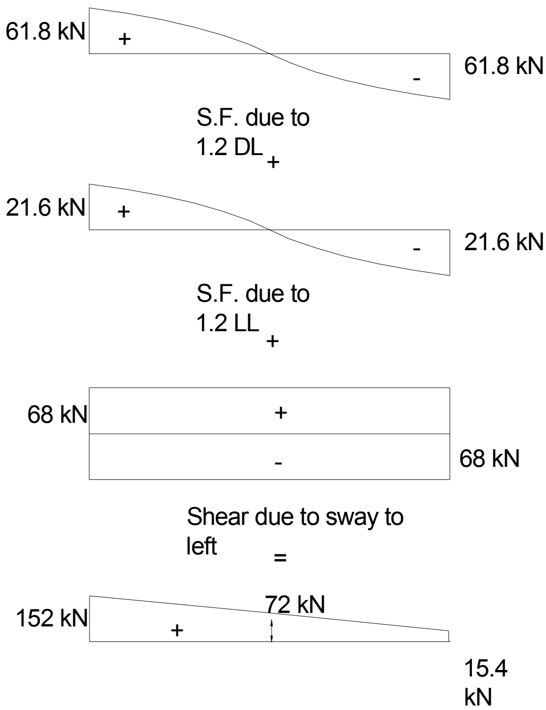


Figure 2.3 Shear diagram due to sway to left

2.6 Design Shear

Referring to the dead and live load diagrams

(Figure 1.4 of Example 1),

DL = Trapezoidal DL+ Brick wall & Self load

= 16.5 x (1 + 5)/2 + 10.575 x 5

= 103 kN

LL = 12 x (1 + 5) / 2 = 36 kN

Shear at left end for sway to right,

$$V_{u,a} = \frac{1.2(DL+LL)}{2} - \frac{1.4(M_u^{As} + M_u^{Bh})}{L}$$

= 1.2 x (103 + 36) /2 - 72

= -11.4 kN

Shear at left end for sway to left,

$$V_{u,a} = \frac{1.2(DL+LL)}{2} + \frac{1.4(M_u^{Ah} + M_u^{Bs})}{L}$$

= 1.2 x (103 + 36) /2 + 68

= 152 kN

Shear at right end for sway to right,

$$V_{u,b} = \frac{1.2(DL+LL)}{2} + \frac{1.4(M_u^{As} + M_u^{Bh})}{L}$$

= 1.2 x (103 + 36)/2 + 72

= 155 kN

Shear at right end for sway to left,

$$V_{u,b} = \frac{1.2(DL+LL)}{2} - \frac{1.4(M_u^{Ah} + M_u^{Bs})}{L}$$

= 1.2 x (103 + 36) /2 - 68

= 15.4 kN

The design shear force shall be the maximum of:

i) Calculated factored shear force as per analysis (Refer Table 2.3)

ii) Shear force due to formation of plastic hinges at both ends of the beam plus due to factored gravity load on the span (as calculated in 2.6.3)

Hence, the design shear force (V_u) will be 152 kN (maximum of 105 kN from analysis and 152 kN corresponding to hinge formation) for the left end of beam and 155 kN (maximum of 113 kN from analysis and 155 kN corresponding to hinge formation) for the right end.

Shear at the mid-span from analysis is 36 kN. However, shear due to formation of plastic hinges at both the ends of the beams will be 72 kN.

The required capacity of shear reinforcement at the left end,

$V_{us} = V_u - V_c$

= 152-66

= 86 kN

Similarly, the required capacity of shear reinforcement at the right end and at mid-span can be calculated as 26 kN and 89 kN, respectively.

Referring to Table 62 of SP: 16, we get the required spacing of 2 legged 8ϕ stirrups as 230 mm centers at left and at the right end. As per Clause 6.3.5 of IS 13920:1993, the spacing of stirrups in the rest of member shall be limited to $d/2$ = 438/2 = 219 mm.

Minimum shear reinforcement as per Clause 26.5.1.6 of IS 456:2000

$S_v = A_{sv} \text{ x } 0.87 f_y /(0.4\, b)$

= 2 x 50 x 0.87 x 415 / (300 x 0.4)

= 300 mm.

< 438 x 0.75 = 328 mm

Hence, ok.

The spacing of minimum stirrups is kept at 300 mm.

Spacing of links over a length of 2*d* at either end of the beam as per Clause 6.3.5 of IS 13920:1993 shall be least of

i) $d/4$ = 438/4= 109 mm

ii) 8 times diameter of smallest bar

= 8 x12 = 96 mm

However, it should not less than 100 mm.

Hence, provide 2 legged 8 ϕ stirrups @100 mm c/c at left and at the right end of the member over a length of $2d$ = 2 x 438 = 876 mm at either end of the beam.

Elsewhere, provide stirrups at 215(< 219 mm) centers.

In case of splicing of main reinforcement, the spacing of links shall be limited to 150 mm centers as per Clause 6.2.6 of IS 13920:1993.

The reinforcement detailing is shown in Figure 2.4.

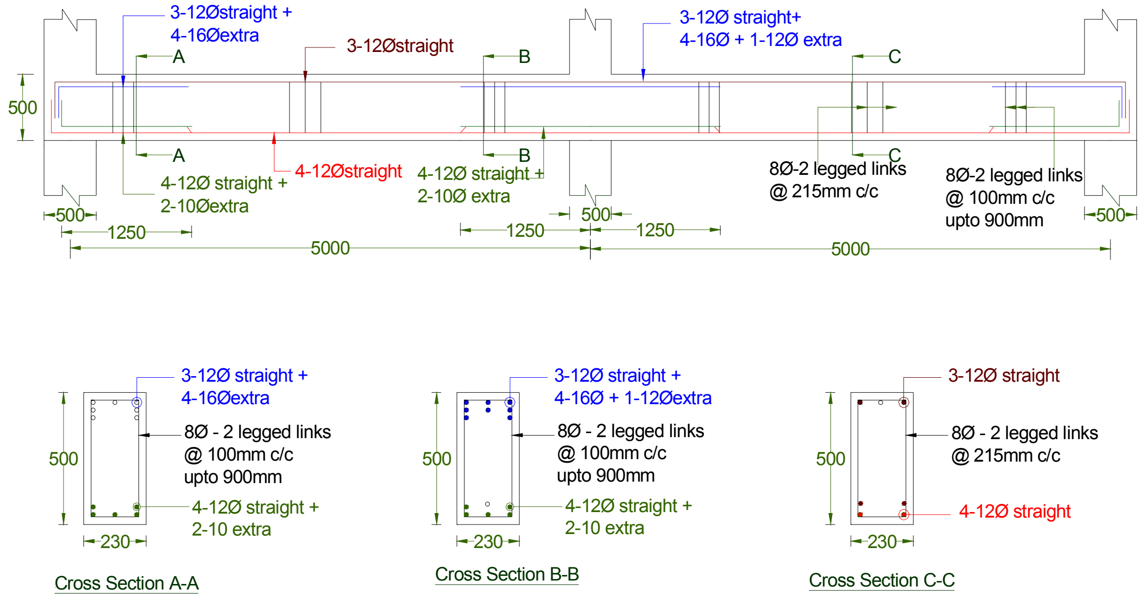


Figure 2.4 Reinforcement details for the beam ABC

2.7 Impact of Ductile Detailing on Bill of Quantities

To compare the impact of ductile detailing (as per IS 13920:1993) on the bill of quantities, the beam has been redesigned as follows:

a) Design and detailing as per IS 456:2000; seismic forces are the same as computed earlier, i.e., with response reduction factor R = 5.0. The reinforcement details are shown in Figure 2.5.

b) Design and detailing as per IS 456:2000; seismic forces are increased by a factor of 5/3 to account for R = 3.0. The reinforcement details are shown in Figure 2.6.

Table 2.6 compares the quantity of reinforcement for the three design cases. While calculating the quantities c/c span is considered.

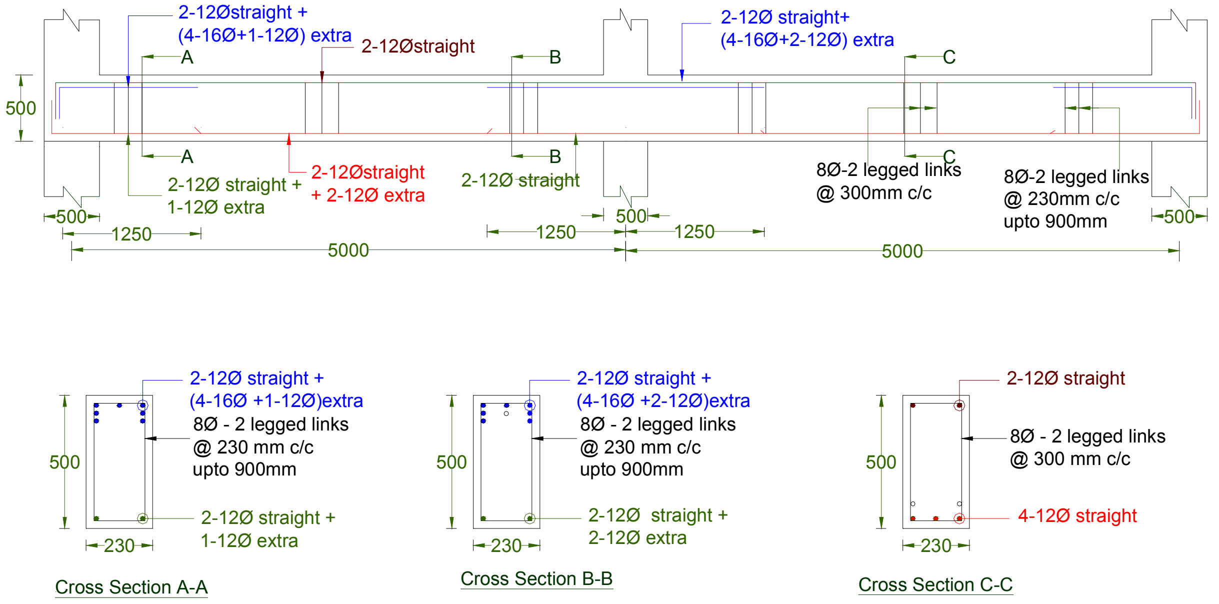


Figure 2.5 Reinforcement detail for the beam ABC as per IS 456:2000 (with *R* = 5.0)

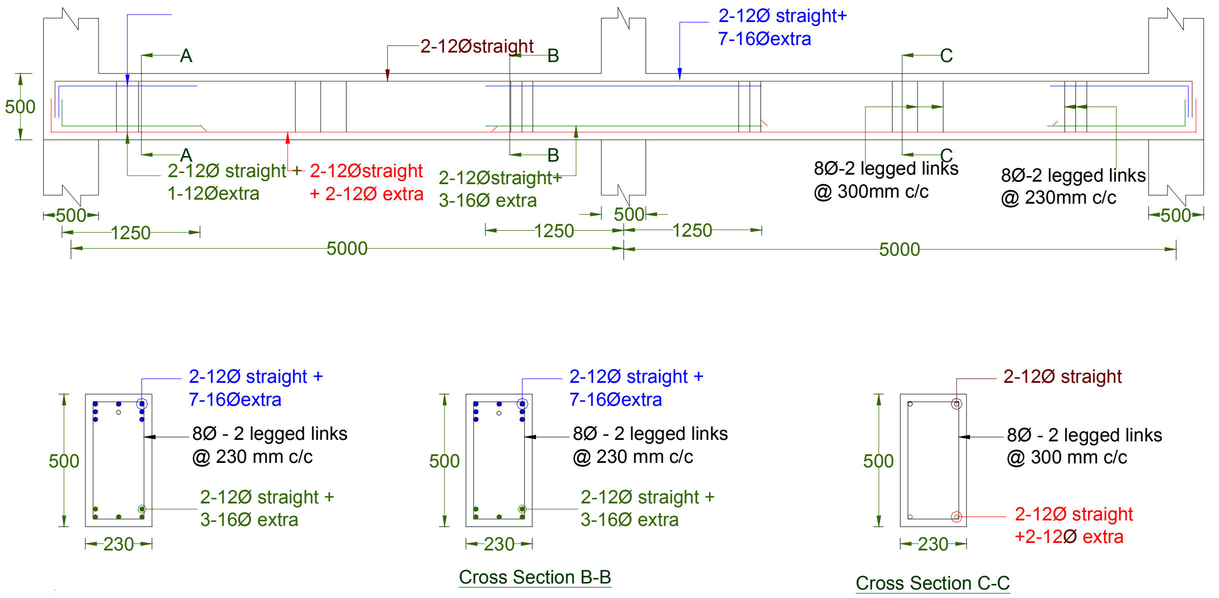


Figure 2.6 Reinforcement detail for the beam ABC as per IS 456:200 (with *R* = 3.0)

Table 2.6 Comparison of bill of quantities of steel in the beam ABC

Description	Detailing as per IS 13920: 1993		Detailing as per IS 456:2000 (Seismic loads with R = 5)		Detailing as per IS 456:200 (Seismic loads with R = 3)	
	Longitudinal	Transverse	Longitudinal	Transverse	Longitudinal	Transverse
Steel required (kg)	52	23	46	13	64	13
Total steel (kg)	75		59		77	
Ratio	1.0		0.79		1.03	

Example 3 - Interior Column Design of an RC Frame Building in Seismic Zone V

3 Problem Statement:

For the ground plus four storey RC office building of Example-1 (Refer Figures 1.1-1.4 of Example 1), design of an interior column element is explained here. The column marked AB in Figure 3.1 for frame 2 is considered for design.

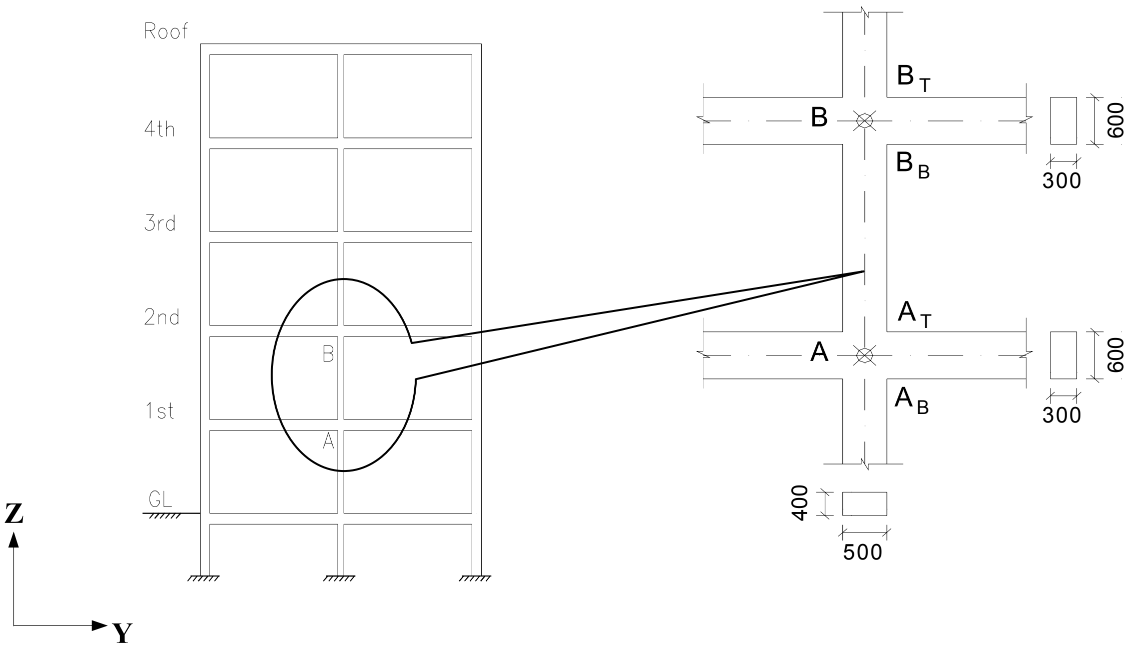


Figure 3.1 Column location in elevation

Solution:

3.1 Load Combinations

Load combinations derived from recommendations of Clause 6.3.1.2 of IS 1893(Part 1): 2002 and given in Table 1.4 of Example-1 are considered for analysis.

3.2 Force Data

For column AB, the force resultants for various load cases and load combinations are shown in Tables 3.1 and 3.2.

In case of earthquake in X direction, column gets a large moment about Y axis and a small moment about X axis due to gravity, minimum eccentricity and torsional effect. Similarly earthquake in Y direction causes a large moment in column about X axis and a small moment about Y axis. Column needs to be designed as a biaxial member for these moments.

Since the column must be designed for earthquake in both X direction and Y direction, all 13 load combinations as shown in Table 1.4 (Example-1) need to be considered. It is necessary to check the column for each combination of loads. Checking the column for all load combinations at all the sections is indeed tedious if carried out by hand. Hence, a computer program is best suited for column design. In the absence of a computer program, one can make a judgment of which two or three load cases out of the thirteen may require the maximum reinforcement and design accordingly.

Referring to Table 3.2, it can be observed that out of the various load combination, one design load combination with earthquake in either (X or Y) direction can be identified, which is likely to be critical. These critical design forces are summarised in Table 3.3. Table 3.4 and Table 3.5

give factors such as $\frac{P_u}{f_{ck}bD}$, $\frac{M_2}{f_{ck}b^2D}$, and $\frac{M_3}{f_{ck}bD^2}$ Using these factors and the charts given in SP: 16, the required maximum reinforcement is calculated and summarised in Table 3.6. The detailed calculations are shown in Section 3.4.

Table 3.1 Force resultants in column AB for different load cases

Load case	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)
DL	-961	1	0	-764	-1	0	-749	1	0	-556	-1	0
LL	-241	0	0	-185	0	0	-185	0	0	-131	1	0
EQx	-22	169	0	-11	-169	0	-11	173	0	-4	-148	0
EQy	0	0	-198	0	0	191	0	0	-194	0	0	166

Table 3.2 Force resultants in column AB for different load combinations

	A_B			A_T			B_B			B_T		
Load Combinations	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)	**-1803**	2	0	**-1424**	-2	0	**-1401**	2	0	**-1031**	0	0
1.2(DL+LL+EQX)	-1252	204	0	-986	-204	0	-968	209	0	-711	-179	0
1.2(DL+LL-EQX)	-1199	-202	0	-959	202	0	-941	-206	0	-702	177	0
1.2(DL+LL+EQY)	-1226	1	-238	-972	-1	229	-954	1	-233	-707	-1	199
1.2(DL+LL-EQY)	-1226	1	238	-972	-1	-229	-954	1	233	-707	-1	-199
1.5(DL+EQX)	**-1475**	**255**	0	**-1163**	**-255**	0	**-1140**	**261**	0	**-840**	**-224**	0
1.5(DL-EQX)	-1409	-252	0	-1130	252	0	-1107	-258	0	-828	221	0
1.5(DL+EQY)	**-1442**	2	**-297**	**-1146**	-2	**287**	**-1124**	2	**-291**	**-834**	-2	**249**
1.5(DL-EQY)	-1442	2	297	-1146	-2	-287	-1124	2	291	-834	-2	-249
0.9DL + 1.5 EQX	-898	254	0	-704	-254	0	-691	260	0	-506	-223	0
0.9DL - 1.5 EQX	-832	-253	0	-671	253	0	**-658**	-259	0	-494	221	0
0.9DL + 1.5 EQY	-865	1	-297	-688	-1	287	-674	1	-291	-500	-1	249
0.9DL - 1.5 EQY	-865	1	297	-688	-1	-287	-674	1	291	-500	-1	-249

3.3 Design Checks

3.3.1 Check for Axial Stress

Lowest factored axial force = 658 kN

(Lowest at A_t or B_b among all load combination is considered)

Factored axial stress = 6,58,000 / (400 x 500)

= 3.29 MPa > 0.10 f_{ck}

Hence, design as a column member.

(Clause 7.1.1; IS 13920:1993)

3.3.2 Check for member size

Width of column, B = 400 mm > 300 mm

Hence, ok

(Clause 7.1.2; IS 13920:1993)

Depth of column, D = 500 mm

$$\frac{B}{D} = \frac{400}{500} = 0.8 \quad > 0.4\text{, hence ok}$$

(Clause 7.1.3; IS 13920:1993)

Span, L = 3,000 mm

The effective length of column can be calculated using Annex E of IS 456: 2000. In this example 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.

$$\frac{L}{D} = \frac{(3000 - 500) \times 0.85}{400} = 5.31 < 12,$$

i.e., Short Column. Hence ok.

(Clause 25.1.2 of IS 456: 2000)

In case of slender column, additional moment due to P-δ effect needs to be considered.

Minimum dimension of column = 400 mm
≥ 15 times the largest diameter of beam longitudinal reinforcement = 15 x 20 = 300 ok
(Clause 7.1.2 of proposed draft IS 13920)

3.3.3 Check for Limiting Longitudinal Reinforcement

Minimum reinforcement,

= 0.8 %.

= 0.8 x 400 x 500/100

= 1,600 mm^2

(Clause 26.5.3.1 of IS 456: 2000)

Maximum reinforcement = 4%

(Limited from practical considerations)

= 4 x 400 x 500/100

= 8,000 mm^2

(Clause 26.5.3.1 of IS 456: 2000)

3.4 Design of Column

3.4.1 Sample Calculation for Column Reinforcement at A_B End

First approximate design is done and finally it is checked for all force combinations.

(a) Approximate Design

In this case, the moment about one axis dominates and hence the column is designed as an uniaxially loaded column. The column is oriented in such a way that depth of column is 400 mm for X direction earthquake and 500 mm for Y direction earthquake force.

Design for Earthquake in X-direction

P_u = 1,475 kN

M_{u2} = 255 kN-m

$$\frac{P_u}{f_{ck} bD} = \frac{1475 \times 10^3}{20 \times 400 \times 500} = 0.37$$

$$\frac{M_{u2}}{f_{ck} bD^2} = \frac{255 \times 10^6}{20 \times 500 \times 400 \times 400} = 0.16$$

Referring to Charts 44 and 45 of SP16
For d'/D = (40 + 25 / 2) / 400 = 0.13, we get p/f_{ck} = 0.14

Design for Earthquake in Y-direction

P_u = 1,442 kN

M_{u2} = 297 kN-m

$$\frac{P_u}{f_{ck} bD} = \frac{1{,}442 \times 10^3}{20 \times 400 \times 500} = 0.36$$

$$\frac{M_{u2}}{f_{ck} bD^2} = \frac{297 \times 10^6}{20 \times 400 \times 500 \times 500} = 0.15$$

Referring to Charts 44 of SP16
For d'/D = (40 + 25 / 2) /500 = 0.105, we get p/f_{ck} = 0.11

Longitudinal Steel

The required steel will be governed by the higher of the above two values and hence, take p/f_{ck} = 0.14.

Required steel = (0.14 x 20) %
= 2.8 %
= 2.8 x 400 x 500 /100
= 5,600 mm^2

Provide 10-25Φ + 4-16Φ bars with total

A_{sc} provided = 5,714 mm^2

i.e., 5,714 x100 /(400 x 500) = 2.85%.

Hence, p/f_{ck} provided = 2.85/20 = 0.143

(b) 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 = 400 mm

P_u = 1,475 kN

M_{u2} = 255 kN-m

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

(Clause 25.4 of IS 456:2000)

= ((3,000-500) / 500) + (400 / 30)

= 18.33 mm < 20 mm

Hence, design eccentricity = 20 mm

M_{u3} = 1,475 x 0.02 = 29.5 kN-m

For $\frac{P_u}{f_{ck}bD} = 0.37$ and p/f_{ck}= 0.143, and referring to Charts 44 and 45 of SP: 16, we get

$$\frac{M_u}{f_{ck}bD^2} = 0.175$$

$$M_{u2,1} = 0.175 \times 20 \times 400 \times 400 \times 500 /(1 \times 10^6)$$

= 280 kN-m

$$M_{u3,1} = 0.175 \times 20 \times 400 \times 500 \times 500 /(1 \times 10^6)$$

= 350 kN-m

$P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}$

(Clause 39.6 of IS 456:2000)

= $0.45 f_{ck} A_g + (0.75 f_y - 0.45 f_{ck}) A_{sc}$

= 0.45 x 20 x 400 x 500 + (0.75 x 415 – 0.45 x 20) x 5,714

= 3,527 kN

P_u/P_{uz} = 1,475 /3,527 = 0.42

The constant α_n which depends on factored axial compression resistance P_{uz} is evaluated as

$$\alpha_n = 1.0 + \frac{0.42 - 0.2}{0.8 - 0.2}(2.0 - 1.0) = 1.367$$

Using the interaction formula of clause 39.6 of IS 456: 2000)

$$\left[\frac{M_{u2}}{M_{u2,1}}\right]^{\alpha_n} + \left[\frac{M_{u3}}{M_{u3,1}}\right]^{\alpha_n} = \left[\frac{255}{280}\right]^{1.367} + \left[\frac{29.5}{350}\right]^{1.367}$$

= 0.88 +0.04

= 0.92 < 1.00

Hence, ok

Checking for Critical Combination with Earthquake in Y Direction (Transverse direction)

Width = 400 mm; Depth = 500 mm

P_u = 1,442 kN

M_{u3} = 297 kN-m

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

= ((3,000-600)/500) + (500 / 30)

= 21.46 mm > 20 mm

M_{u2} = 1,442 x 0.02146 = 31 kN-m

For $\frac{P_u}{f_{ck}bD} = 0.355$ and p/f_{ck}= 0.143,

Referring to Chart 44 of SP: 16, we get

$$\frac{M_{u2,1}}{f_{ck}bD^2} = 0.18$$

$$M_{u2,1} = 0.18 \times 20 \times 400 \times 400 \times 500 / 1 \times 10^6$$

= 288 kN-m

$$M_{u3,1} = 0.18 \times 20 \times 400 \times 500 \times 500 / 1 \times 10^6$$

= 360 kN-m

P_{uz} = 3,527 kN

α_n = 1.35

Using the interaction formula

$$\left[\frac{M_{u2}}{M_{u2,1}}\right]^{\alpha_n} + \left[\frac{M_{u3}}{M_{u3,1}}\right]^{\alpha_n} = \left[\frac{31}{288}\right]^{1..367} + \left[\frac{297}{360}\right]^{1.367}$$

= 0.0473 +0.7687

= 0.816 <1.00

Hence, ok

3.5 Details of Longitudinal Reinforcement

Similar to the sample calculations shown in Section 3.4.1, the steel required at A_T, B_B and B_T is calculated. The Tables 3.4 and 3.5 show brief calculations at A_B, A_T, B_B and B_T locations. The column at joint A should have higher of the reinforcement required at A_B and A_T, and hence 2.8% steel is needed. Similarly, higher of the reinforcement required at B_B and B_T, i.e. 2.4% is needed in the column at joint B.

Figure 3.2 shows the reinforcement in the column along with the steel provided in the transverse and longitudinal beams.

Table -3.3 Critical forces for design of column AB

Load Combination	A_B			A_T			B_B			B_T		
	P	M_2	M_3	P	M_2	M_3	P	M_2	M_3	P	M_2	M_3
Gravity	-1,803	2	0	-1,424	-2	0	-1,401	2	0	-1,031	0	0
Critical comb with EQX	-1,475	255	0	-1,163	-255	0	-1,140	261	0	-840	-224	0
Critical comb with EQY	-1,442	2	-297	-1,146	-2	287	-1,124	2	-291	-834	-2	249

Table- 3.4 Design of column AB for earthquake in X direction

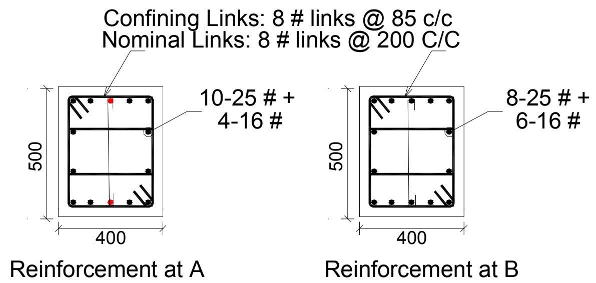
Load Comb	A_B			A_T			B_B			B_T		
	$\frac{P_u}{f_{ck}bD}$	$\frac{M_2}{f_{ck}b^2D}$	p	$\frac{P_u}{f_{ck}bD}$	$\frac{M_2}{f_{ck}b^2D}$	p	$\frac{P_u}{f_{ck}bD}$	$\frac{M_2}{f_{ck}b^2D}$	p	$\frac{P_u}{f_{ck}bD}$	$\frac{M_2}{f_{ck}b^2D}$	p
Gravity	0.45	0.00	0.8	-0.36	0.00	0.8	-0.35	0.00	0.8	0.26	0.00	0.8
Critical comb with EQX	0.37	0.16	2.8	0.29	0.16	2.4	0.29	0.16	2.4	0.21	0.14	2.0

Table- 3.5 Design of column AB for earthquake in Y direction

Load Comb	A_B			A_T			B_B			B_T		
	$\frac{P_u}{f_{ck}bD}$	$\frac{M_3}{f_{ck}bD^2}$	p	$\frac{P_u}{f_{ck}bD}$	$\frac{M_3}{f_{ck}bD^2}$	p	$\frac{P_u}{f_{ck}bD}$	$\frac{M_3}{f_{ck}bD^2}$	p	$\frac{P_u}{f_{ck}bD}$	$\frac{M_3}{f_{ck}bD^2}$	p
Critical comb with EQY	0.36	0.145	2.8	0.29	0.133	1.8	0.28	0.1455	2.2	0.21	0.124	1.6

Note: b = 400 mm and D = 500mm

Table 3.6 Summary of reinforcement for column AB

Column AB	Longitudinal Reinforcement	
Reinforcement at A	10-25Φ +4-16Φ Steel provided = 5,714 mm^2 i.e., 2.86%	
Reinforcement at B	8-25Φ+ 6-16Φ Steel provided= 5,134 mm^2 i.e., 2.57%	

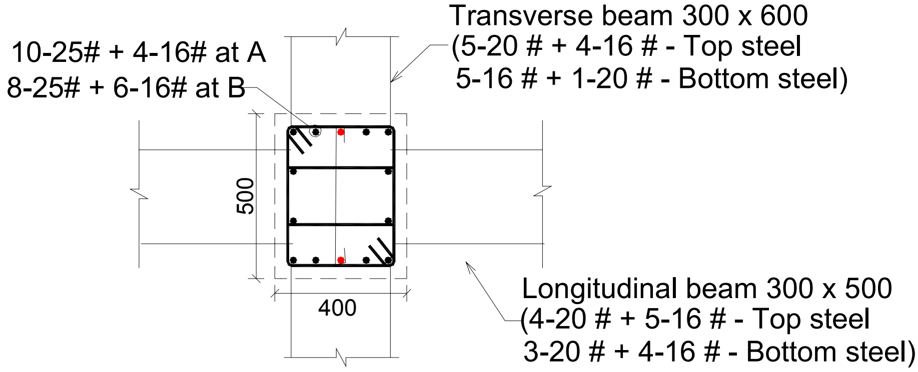


Figure 3.2 Reinforcement details of longitudinal and transverse beam

Table - 3.7 Shear forces in column AB for different load combinations

	A_B		A_T		B_B		B_T	
Load Combination	EQX (kN)	EQY (kN)	EQX (kN)	EQY (kN)	EQX (kN)	EQY (kN)	EQX (kN)	EQY (kN)
1.5(DL+LL)	0	-1	0	0	0	0	0	0
1.2(DL+LL+EQX)	0	-133	0	-137	0	-137	0	-122
1.2(DL+LL-EQX)	0	132	0	136	0	136	0	121
1.2(DL+LL+EQY)	149	0	154	0	154	0	136	0
1.2(DL+LL-EQY)	-149	0	-154	0	-154	0	-136	0
1.5(DL+EQX)	0	-167	0	**-171**	0	-171	0	-153
1.5(DL-EQX)	0	166	0	170	0	170	0	152
1.5(DL+EQY)	186	-1	192	-1	192	-1	171	-1
1.5(DL-EQY)	-186	-1	-192	-1	-192	-1	-171	-1
0.9DL + 1.5 EQX	0	-167	0	-171	0	-171	0	-153
0.9DL - 1.5 EQX	0	166	0	170	0	170	0	152
0.9DL + 1.5 EQY	186	0	192	0	192	0	171	0
0.9DL - 1.5 EQY	-186	0	**-192**	0	-192	0	-171	0

3.6 Design for Shear

3.6.1 Shear Capacity of Column

Assuming 50% steel provided as tensile steel to be on conservative side, A_{st} = 2.86% / 2 = 1.43%

Permissible shear stress τ_c = 0.70 Mpa

(Table 19 of IS 456: 2000)

Considering lowest P_u = 658 kN, we get

Multiplying factor = $\delta = 1 + \frac{3P_u}{A_g f_{ck}}$ =1.49 < 1.5

(Clause 40.2.2 of IS 456: 2000)

τ_c = 0.70 x 1.49 = 1.043 MPa

Effective depth in X direction = 400-40-25/2 = 347.5 mm

V_c = 1.043 x 500 x 347.5 /1,000 = 181 kN

Effective depth in Y direction = 500-40-25/2 = 447.5 mm

V_c = 1.043 x 400 x 447.5 /1,000 = 187 kN

3.6.2 Shear As Per Analysis

As per Table 3.7, the maximum factored shear force in X and Y direction is 192 and 171 kN, respectively.

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

3.6.3.1 Earthquake in X-Direction

The longitudinal beam of size 300 x 500 is reinforced with 4-20Φ extra + 5-16Φ str (2,261 mm^2, i.e., 1.74%) at top and 3-20Φ extra + 4-16Φ str (1,746 mm^2, i.e., 1.34%) at bottom. The hogging and sagging moment capacities are evaluated as 288 kN-m and 221 kN-m, respectively.

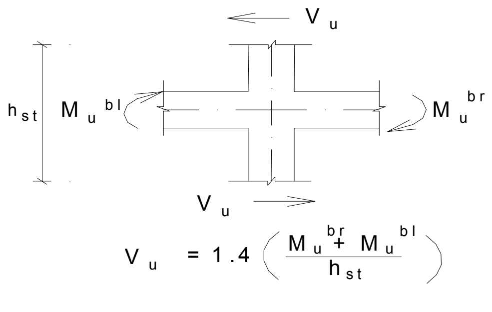


Figure 3.3 Column shear due to plastic hinge formation in beams

Referring to Figure 3.3, the shear force corresponding to plastic hinge formation in the longitudinal beam is evaluated as:

$$V_u = \frac{1.4\ (M_u^{bl} + M_u^{br})}{h_{st}}$$

= 1.4 x (288 +221) /3

= 237 kN

3.6.3.2 Earthquake in Y-Direction

The transverse beam of size 300 x 600 is reinforced with 3-16Φ str + 5-20Φ + 1-16Φ extra (2,374 mm^2, i.e., 1.485%) at top and 3-16Φ str + 3-20Φ extra (1545 mm^2, i.e., 0.978%) at bottom. The hogging and sagging moment capacity is evaluated as 377 kN-m and 246 kN-m, respectively.

Referring to Figure 3.3, the shear force corresponding to plastic hinge formation in the transverse beam is

$$V_u = \frac{1.4\ (M_u^{bl} + M_u^{br})}{h_{st}}$$

$$= \frac{1.4 \times (377 + 246)}{3}$$

= 291 kN

3.6.4 Design Shear

The design shear force for the column shall be the higher of the calculated factored shear force as per analysis (Table 3.7) and the shear force due to plastic hinge formation in either of the transverse or the longitudinal beam.

(Clause7.3.4; IS 13920: 1993)

The design shear in X direction is 237 kN which is the higher of 192 kN and 237 kN. Similarly, the design shear in Y direction is 291 kN which is the higher of 171 kN and 291 kN.

3.7 Details of Transverse Reinforcement

3.7.1 Design of Links in X Direction

V_s = 237 – 181 = 56 kN .

Spacing of 4 Legged 8 Φ Links

$$= \frac{4 \times 50 \times 0.87 \times 415 \times 347.5}{56{,}000} = 448 \text{ mm}$$

3.7.2 Design of Links in Y Direction

V_s = 287 – 187 = 100 kN

Spacing of 3 legged 8 Φ Links

$$= \frac{3 \times 50 \times 0.87 \times 415 \times 447.5}{1,00,000} = 243 \text{ mm}$$

3.7.3 Nominal Links

The spacing of hoops shall not exceed half the least lateral dimension of the column i.e., 400/ 2 = 200 mm. (Clause 7.3.3; IS 13920:1993)

Provide 8 Φ links @ 200 c/c in mid-height portion of the column.

3.7.4 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

(Clause 7.4.8 of IS 13920:1993).

$$A_{sh} = \frac{0.18 \times S \times h \times f_{ck}}{f_y}\left(\frac{A_g}{A_k} - 1\right)$$

h = longer dimension of the rectangular link measured to its outer face

= ((500 – 40 – 40 – 25) /3 + (8 x 2)) + 25)

= 172 mm, or

((400 – 40 – 40- 25)/2 +(8 x 2) +25) =188.5 mm,

Whichever is higher, i.e,. h = 188.5 mm.

A_g = 400 x 500 = 2,00,000 mm^2

A_k = (400- 2 x 40 +2 x 8) x (500- 2 x 40 +2 x 8)

= 336 x 436

= 1,46,496 mm^2

Assuming 8Φ stirrup, A_{sh} = 50 mm^2

$$50 = \frac{0.18 \times S \times 188.5 \times 20}{415}\left(\frac{2,00,000}{1,46,496} - 1\right)$$

Substituting we get S = 84 mm.

Link spacing for confining zone shall not exceed:

(a) ¼ of minimum column dimension i.e,

400 / 4 =100 mm

(b) But need not be less than 75 mm nor more than 100 mm. (Clause 7.4.6 of IS 13920:1993).

Provide 8Φ confining links @ 80 c/c for a distance l_o (Refer Figure 3.4), which shall not be less than:

i) Larger lateral dimension = 500 mm

ii) 1/6 of clear span = (3000 – 500) / 6 = 417 mm

iii) 450 mm

(Clause 7.4.1 of IS 13920:1993)

Provide confining reinforcement for a distance of l_o = 500 mm on either side of the joint. [Refer Figure 3.4]

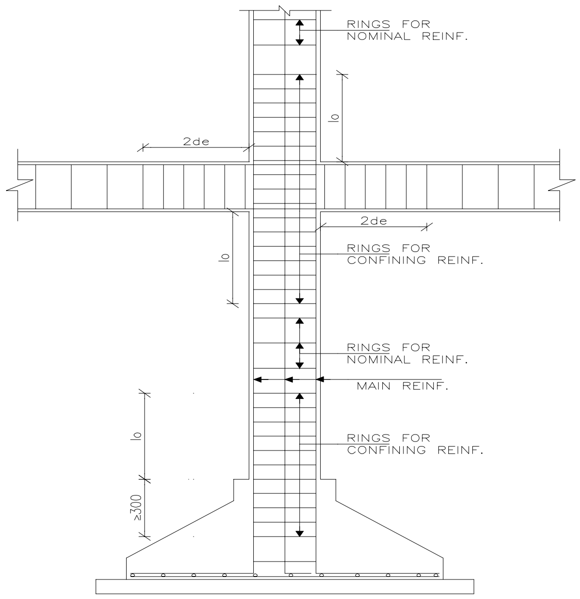


Figure 3.4 Reinforcement details for column

The comparisons of steel quantities are shown in Table 3.8 for various detailing options.

Table 3.8 Comparison of bill of quantities of steel in column

Description	Detailing as per IS 13920: 1993 (Seismic loads as per R = 5)	Detailing as per IS 456: 2000 (Seismic loads as per R = 5)	Detailing as per IS 456: 2000 (Seismic loads as per R = 3)
Links (kg)	25	14	Column needs to be redesigned.
Main steel (kg)	128	128	

CAUTION

Note, however, that the column designed above has not been checked for requirements related to the joint region, which are being incorporated in the new edition of IS 13920. The applications of these provisions are illustrated in Examples 5-8 and may require modifications in column size and /or longitudinal reinforcement.

Example 4 - Exterior Column Design of an RC Frame Building in Seismic Zone V

4 Problem Statement:

For the ground plus four storey RC office building of Example-1 (Refer Figures 1.1-1.4 of Example 1), design of an exterior column element is explained here. The column marked AB in Figure 4.1 for frame 2 is considered for design.

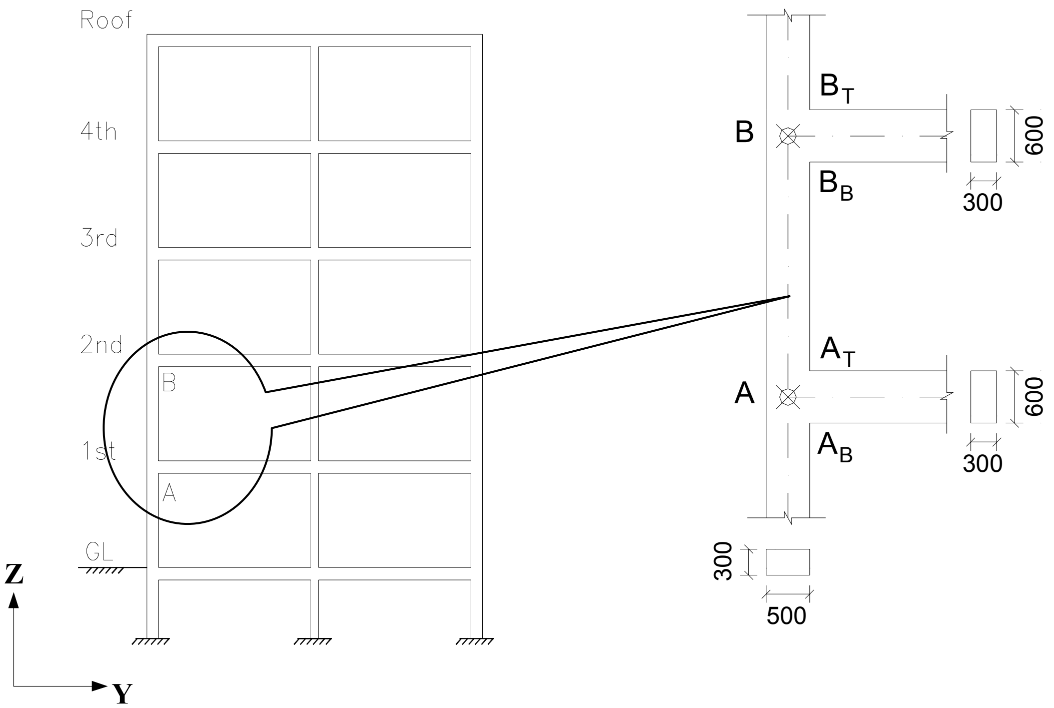


Figure 4.1 Column location in elevation

Solution:

4.1 Load Combinations

Load combinations derived from recommendations of Clause 6.3.1.2 of IS 1893(Part 1): 2002 and given in Table 1.4 of Example-1 are considered for analysis.

4.2 Force Data

For column AB, the force resultants for various load cases and load combinations are shown in Tables 4.1 and 4.2.

In case of earthquake in X direction, column gets a large moment about Y axis and a small moment about X axis due to gravity, minimum eccentricity and torsional effect. Similarly earthquake in Y direction causes a large moment in column about X axis and a small moment about Y axis. Column needs to be designed as a biaxial member for these moments.

Since the column must be designed for earthquake in both X-direction and Y-direction, all 13 load combinations as shown in Table 1.4 (Example-1) need to be considered. It is necessary to check the column for each combination of loads. Checking the column for all load combinations at all the sections is indeed tedious if carried out by hand. Hence, a computer program is best suited for column design. In the absence of a computer program, one can make a judgment of which two or three load cases out of the thirteen may require the maximum reinforcement and design accordingly.

Referring to Table 4.2, it can be observed that out of the various load combination, one design load combination with earthquake in either (X or Y) direction can be identified, which is likely to be critical. These design forces are summarised in Table 4.3. Table 4.4 and Table 4.5 give factors such as $\frac{P_u}{f_{ck}bD}$, $\frac{M_2}{f_{ck}b^2D}$, and $\frac{M_3}{f_{ck}bD^2}$.

Using these factors and the charts given in SP: 16, the required maximum reinforcement is calculated the same being summarised in Table 4.6. The detailed calculations are shown in Section 4.4.

Table 4.1 Force resultants in column AB for different load cases

Load Case	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)
DL	-643	0	15	-513	1	-22	-502	-1	21	-372	1	-20
LL	-119	0	5	-93	0	-7	-93	0	7	-66	0	-7
EQx	-50	108	0	-29	-112	0	-29	112	0	-14	-97	0
EQy	270	0	-112	191	0	97	191	0	-104	119	0	86

Table 4.2 Force resultants in column AB for different load combinations

Load Combinations	A_B			A_T			B_B			B_T		
	P kN	M_2 kN-m	M_3 kN-m	P kN	M_2 kN-m	M_3 kN-m	P kN	M_2 kN-m	M_3 kN-m	P kN	M_2 kN-m	M_3 kN-m
1.5(DL+LL)	-1143	0	30	-909	2	-44	-893	-2	42	-657	2	-41
1.2(DL+LL+EQX)	-867	130	20	-678	-133	-29	-665	133	27	-483	-115	-26
1.2(DL+LL-EQX)	-747	-130	20	-609	136	-29	-596	-136	27	-449	118	-26
1.2(DL+LL+EQY)	-483	0	-115	-414	1	88	-401	-1	-98	-323	1	77
1.2(DL+LL-EQY)	-1131	0	154	-873	1	-145	-860	-1	152	-609	1	-129
1.5(DL+EQX)	**<u>-1040</u>**	**<u>162</u>**	23	**<u>-813</u>**	**<u>-167</u>**	-33	**<u>-797</u>**	**<u>167</u>**	32	**<u>-579</u>**	**<u>-144</u>**	-30
1.5(DL-EQX)	-890	-162	23	-726	170	-33	-710	-170	32	-537	147	-30
1.5(DL+EQY)	-560	0	-146	-483	2	113	-467	-2	-125	-380	2	99
1.5(DL-EQY)	**<u>-1370</u>**	0	**<u>191</u>**	**<u>-1056</u>**	2	**<u>-179</u>**	**<u>-1040</u>**	-2	**<u>188</u>**	**<u>-737</u>**	2	**<u>-159</u>**
0.9DL + 1.5 EQX	-654	162	14	-505	-167	-20	-495	167	19	-356	-145	-18
0.9DL - 1.5 EQX	-504	-162	14	-418	169	-20	-408	-169	19	-314	146	-18
0.9DL + 1.5 EQY	-174	0	-155	-175	1	126	-165	-1	-137	-156	1	111
0.9DL - 1.5 EQY	-984	0	182	-748	1	-165	-738	-1	175	-513	1	-147

4.3 Design Checks

4.3.1 Check for Axial Stress

Factored axial force = 166 kN

(Lowest at A_t or B_b among all load combination is considered)

Factored axial stress = 1,66,000 /300 x 500

= 1.10 MPa < 0.10 f_{ck}

Hence, for the load combination 0.9DL + 1.5 EQY the member needs to be checked as flexural member. For all other load combinations design is done as a Column member.

(Clause 7.1.1; IS 13920:1993)

4.3.2 Check for Member Size

Width of column, B = 300 mm ≥ 300 hence, ok.

(Clause 7.1.2; IS 13920:1993)

Depth of column, D = 500 mm

$$\frac{B}{D} = \frac{300}{500} = 0.6 \quad > 0.4, \text{ hence ok.}$$

(Clause 7.1.3; IS 13920:1993)

Span, L = 3,000 mm

The effective length of column can be calculated using Annex E of IS 456: 2000. In this example 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.

$$\frac{L}{D} = \frac{(3000-500)\times 0.85}{300} = 7.1 \ < 12, \text{ i.e.,}$$

Short column.

Hence, ok. (Clause 25.1.2 of IS 456: 2000)

In case of slender column, additional moment due to P-δ effect needs to be considered.

Minimum Dimension of Column = 300 mm
≥ 15 times the largest diameter of beam longitudinal reinforcement = 15 x 20 = 300 ok.
(Clause 7.1.2 of proposed draft IS 13920)

4.3.3 Check for Reinforcement

Minimum reinforcement,

= 0.8 %.

= 0.8 x 300 x 500 /100

= 1,200 mm^2

(Clause 26.5.3.1 of IS 456: 2000)

Maximum reinforcement = 4%

(Limited from practical considerations)

= 4 x 300 x 500 / 100

= 6,000 mm^2

(Clause 26.5.3.1 of IS 456: 2000)

4.4 Design of Column

4.4.1 Sample Calculation for Column Reinforcement at A_B End

First approximate design is done and finally it is checked for all force combinations.

(a) Approximate Design

In this case, the moment about one axis dominates and hence, the column is designed as an uniaxially loaded column in that direction. The column is oriented in such a way that depth of column is 300 mm for X direction earthquake and 500 mm for Y direction earthquake force.

Design for Earthquake in X-Direction

P_u = 1,040 kN

M_{u2} = 162 kN-m

$$\frac{P_u}{f_{ck}bD} = \frac{1040\times10^3}{20\times300\times500} = 0.347$$

$$\frac{M_{u2}}{f_{ck}bD^2} = \frac{162\times10^6}{20\times500\times300\times300} = 0.18$$

Referring to Charts 45 and 46 of SP16

For d'/D = (40 +25 /2)/300 = 0.175, we get

p/f_{ck} = 0.185.

Design of Earthquake in Y-Direction

P_u = 1,370 kN

M_{u2} = 191kN-m

$$\frac{P_u}{fckbD} = \frac{1370\times10^3}{20\times300\times500} = 0.456$$

$$\frac{M_{u2}}{f_{ck}bD^2} = \frac{191\times10^6}{20\times300\times500\times500} = 0.127$$

Referring to Charts 44 of SP16
For d'/D = (40 +25 / 2)/500 = 0.105, we get

p/f_{ck} = 0.12

Longitudinal Steel

The required steel will be governed by the higher of the above two values and hence p/f_{ck} = 0.185,

Required steel = 0.185 x 20% = 3.7 %

= 3.7 x 300 x 500 /100

= 5,550 mm^2

Provide 12-25Φ bars with total A_{sc} = 5,892 mm^2

i.e., 5,892 x 100 / (300 x 500) = 3.92%.

Hence p/f_{ck} provided = 3.92/20 = 0.20 > 0.185

Hence, ok.

(b) Checking of Section

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

Checking for Critical Combination with Earthquake in X Direction (Longitudinal direction)

Width = 500 mm; Depth = 300 mm

P_u = 1,040 kN

M_{u2} = 162 kN-m

Eccentricity, e_3 = clear height of column /500 + lateral dimension / 30

= ((3,000-500) / 500) + (300 /30)

= 15 mm > 20 mm

M_{u3} = 1,040 x 0.02 = 20.8 kN-m

For $\frac{P_u}{f_{ck}bD} = 0.347$ and p/f_{ck}= 0.20**,** and referring to Charts 44-45 of SP: 16 and we get

$$\frac{M_u}{f_{ck}bD^2} = 0.2$$

$$M_{u2,1} = 0.2 \times 20 \times 300 \times 300 \times 500 /(1 \times 10^6)$$

= 180 kN-m.

$$M_{u3,1} = 0.2 \times 20 \times 300 \times 500 \times 500 /(1 \times 10^6)$$

= 300 kN-m.

$P_{uz} = 0.45 f_{ck}A_g + (0.75f_y - 0.45f_{ck}) A_{sc}$

(Clause 39.6 of IS 456:2000)

= 0.45 x 20 x 300 x 500 +

(0.75 x 415 - 0.45 x 20) x 5,892

= 3,130 kN

P_u/P_{uz} = 1,040 / 3,130 = 0.33

The constant α_n which depends on factored axial compression resistance P_{uz} is evaluated as

$$\alpha_n = 1.0 + \frac{0.33 - 0.2}{0.8 - 0.2}(2.0 - 1.0) = 1.216$$

Using the interaction formula of Clause 39.6, IS 456: 2000),

$$\left[\frac{M_{u2}}{M_{u2,1}}\right]^{\alpha_n} + \left[\frac{M_{u3}}{M_{u3,1}}\right]^{\alpha_n} = \left[\frac{162}{180}\right]^{1.216} + \left[\frac{20.8}{300}\right]^{1.216}$$

= 0.88 + 0.039

= 0.92 < 1.00

Hence, ok.

Checking for Critical Combination with Earthquake in Y Direction (Transverse direction)

Width = 300 mm; Depth = 500 mm;

P_u = 1,370 kN

M_{u3} = 191 kN-m

Eccentricity,e_3 = clear height of column /500 + lateral dimension /30

= ((3,000 – 600)/500) + (500 / 30)

= 21.46 mm > 20 mm

Hence, minimum eccentricity = 20 mm.

M_{u2} = 1,370 x 0.02146 = 29.4 kN-m

For $\frac{P_u}{f_{ck}bD} = 0.456$ and p/f_{ck} = 0.20**,** Referring to Chart 44 of SP: 16, we get

$$\frac{M_{u2,1}}{f_{ck}bD^2} = 0.18$$

$$M_{u3,1} = 0.18 \times 20 \times 500 \times 500 \times 300 / 1 \times 10^6$$

= 270 kN-m

$$M_{u2,1} = 0.18 \times 20 \times 300 \times 300 \times 500 / 1 \times 10^6$$

= 162 kN-m

P_{uz} = 3,130 kNα_n = 1.28

Using the interaction formula

$$\left[\frac{M_{u2}}{M_{u2,1}}\right]^{\alpha_n} + \left[\frac{M_{u3}}{M_{u3,1}}\right]^{\alpha_n} = \left[\frac{29.4}{162}\right]^{1.216} + \left[\frac{191}{270}\right]^{1.216}$$

$$= 0.12 + 0.66$$

$$= 0.78 < 1.00 \quad \text{ok.}$$

4.5 Details of Longitudinal Reinforcement

Similar to the sample calculations shown in Section 4.4.1, the steel required at A_B , A_T, B_B and B_T is calculated.

The Tables 4.4 and 4.5 show brief calculations at A_B, A_T, B_B and B_T locations. The column at the joint A should have the higher of the reinforcement required at A_B and A_T, and hence, 3.7% steel is needed. Similarly higher of the reinforcement required at B_B and B_T, i.e., 3.5% is needed in the column at the joint B.

Figure 4.2 shows the reinforcement in the column along with steel provided in the transverse and longitudinal beams.

Table -4.3 Critical forces for design of column AB

Load Combination	A_B			A_T			B_B			B_T		
	P	M_2	M_3	P	M_2	M_3	P	M_2	M_3	P	M_2	M_3
Gravity	-1,143	0	30	-909	2	-44	-893	-2	42	-657	2	-41
Critical comb with EQX	-1,040	162	22	-813	-167	-33	-797	167	32	-579	-144	-30
Critical comb with EQY	-1,370	0	191	-1,056	2	-179	-1,040	-2	188	-737	2	-159

Table -4.4 Design of column AB for earthquake in X-Direction

Load Comb	A_B			A_T			B_B			B_T		
	$\frac{P_u}{f_{ck}bD}$	$\frac{M_2}{f_{ck}b^2D}$	p	$\frac{P_u}{f_{ck}bD}$	$\frac{M_2}{f_{ck}b^2D}$	p	$\frac{P_u}{f_{ck}bD}$	$\frac{M_2}{f_{ck}b^2D}$	p	$\frac{P_u}{f_{ck}bD}$	$\frac{M_2}{f_{ck}b^2D}$	p
Gravity	0.39	-	0.80	0.30	0.03	0.8	0.3	0.03	0.8	0.22	0.03	0.8
Critical comb with EQX	0.347	0.178	3.7	0.27	0.18	3.2	0.28	0.185	3.5	0.19	0.16	2.8

Table - 4.5 Design of column AB for earthquake in Y-Direction

Load Comb.	A_B			A_T			B_B			B_T		
	$\frac{P_u}{f_{ck}bD}$	$\frac{M_3}{f_{ck}bD^2}$	p	$\frac{P_u}{f_{ck}bD}$	$\frac{M_3}{f_{ck}bD^2}$	p	$\frac{P_u}{f_{ck}bD}$	$\frac{M_3}{f_{ck}bD^2}$	p	$\frac{P_u}{f_{ck}bD}$	$\frac{M_3}{f_{ck}bD^2}$	p
Critical comb with EQY	0.46	0.13	2.4	0.35	0.12	1.6	0.35	0.13	2.0	0.24	0.11	1.2

Table - 4.6 Summary of reinforcement for column AB

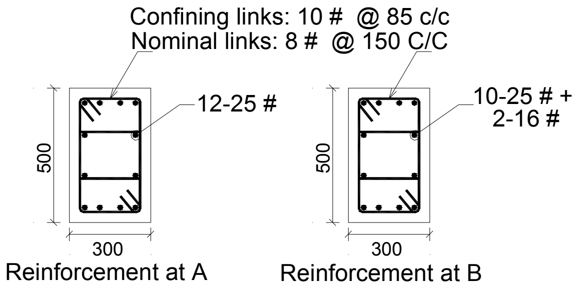
Column AB	Longitudinal Reinforcement	Reinforcement Details
Reinforcement at A	12-25Φ Bars A_{sc} = 5,892 mm^2 i.e., 3.93%.	
Reinforcement at B	10-25Φ+ 2-16Φ Steel provided= 5,360 mm^2 i.e., 3.57%	

Table -4.7 Tabulation of shear forces in column AB for different load combinations

	A_B		A_T		B_B		B_T	
Load Combination	EQX (kN)	EQY (kN)	EQX (kN)	EQY (kN)	EQX (kN)	EQY (kN)	EQX (kN)	EQY (kN)
1.5(DL+LL)	-14	0	-29	1	-29	1	-27	2
1.2(DL+LL+EQX)	-9	-83	-18	-89	-18	-89	-17	-79
1.2(DL+LL-EQX)	-9	83	-19	90	-19	90	-18	81
1.2(DL+LL+EQY)	78	0	62	0	62	0	56	0
1.2(DL+LL-EQY)	-97	0	-99	1	-99	1	-92	2
1.5(DL+EQX)	-10	-104	-21	-111	-21	-111	-20	-98
1.5(DL-EQX)	-11	104	-22	**112**	-22	112	-21	101
1.5(DL+EQY)	99	0	80	0	80	0	72	0
1.5(DL-EQY)	-120	0	**-123**	1	-123	1	-113	2
0.9DL + 1.5 EQX	-6	-104	-12	-111	-12	-111	-11	-99
0.9DL - 1.5 EQX	-7	104	-14	112	-14	112	-13	100
0.9DL + 1.5 EQY	103	0	88	0	88	0	80	0
0.9DL - 1.5 EQY	-116	0	-114	1	-114	1	-105	2

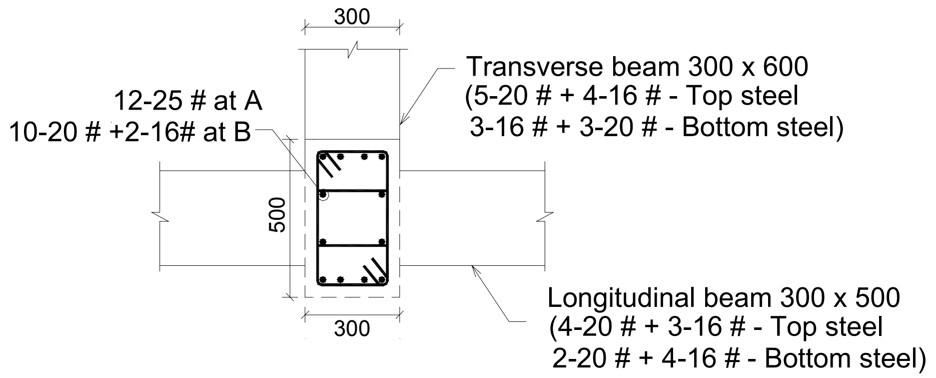


Figure 4.2 Reinforcement details of longitudinal and transverse beam

4.6 Design for Shear

4.6.1 Shear Capacity of Column

Assuming 50% steel provided as tensile steel to be on conservative side, A_{st} = 3.92% / 2 = 1.96%

Permissible shear stress τ_c = 0.79 Mpa

(Table 19 of IS 456: 2000)

Considering lowest P_u = 166 kN,

$$\delta = 1 + \frac{3P_u}{A_g f_{ck}} = 1.167 < 1.5$$

(Clause 40.2.2 of IS 456: 2000)

τ_c = 0.79 x 1.167 = 0.92 MPa

Effective depth in X direction = 300-40-25/2 = 247.5 mm

V_c = 0.92 x 500 x 247.5 /1,000 = 114 kN

Effective depth in Y direction = 500-40-25/2 = 447.5 mm

V_c = 0.92 x 300 x 447.5 /1,000 = 123 kN

4.6.2 Shear As Per Analysis

The maximum factored shear force in X and Y direction is 123 and 112 kN respectively. (Refer Table 4.7)

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

4.6.3.1 Earthquake in X-Direction

The longitudinal beam of size 300 x 500 is reinforced with 4-20Φ extra +3-16Φ str (1859 mm^2, i.e., 1.43%) at top and 2-20Φ extra + 4-16Φ str (1432 mm^2, i.e., 1.10%) at bottom. The hogging and sagging moment capacities are evaluated as 238 kN-m and 180 kN-m, respectively.

Referring to Figure 4.3, the shear force corresponding to plastic hinge is evaluated as:

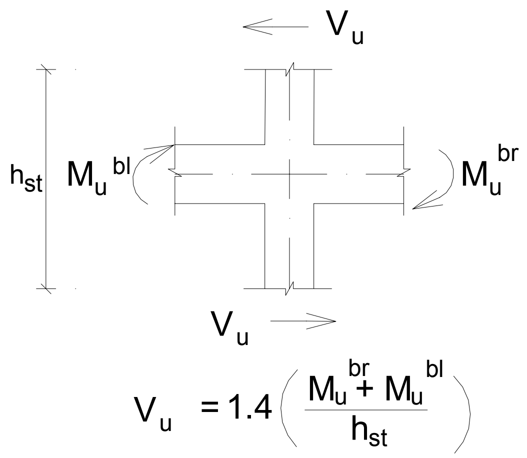


Figure 4.3 Column shear due to plastic hinge formation in longitudinal beams

$$V_u = \frac{1.4\ (M_u^{bl} + M_u^{br})}{h_{st}}$$

= 1.4 x (238 +180) /3

= 195 kN

4.6.3.2 Earthquake in Y-Direction

The transverse beam of size 300 x 600 is reinforced with 3-16Φ str + 5-20Φ extra + 1-16Φ extra (2,374 mm^2, i.e., 1.487 %) at top and 3-16Φ str + 3-20Φ extra (1,545 mm^2, i.e., 0.978%) at bottom. The hogging and sagging moment capacities are evaluated as 377 kN-m and 286 kN-m, respectively.

Referring to Figure 4.4, the shear force corresponding to plastic hinge formation in the transverse beam is

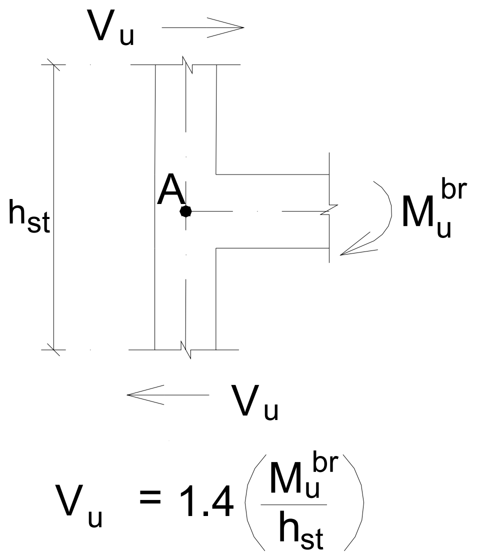


Figure 4.4 Column shear due to plastic hinge formation in transverse beams

$$V_u = \frac{1.4\,(M_u^{br})}{h_{st}}$$

$$= \frac{1.4 \times (377)}{3}$$

= 176 kN.

4.6.4 Design Shear

The design shear force for the column shall be the higher of the calculated factored shear force as per analysis (Table 4.7) and the shear force due to plastic hinge formation in either of the transverse or longitudinal beams.

(Clause7.3.4; IS 13920: 1993)

From Section 4.6.3 above, the design shear in X direction is 195 kN which is the higher of 112 kN and 195 kN. Similarly the design shear in Y direction is 176 kN, which is the higher of 123 kN and 176 kN.

4.7 Details of Transverse Reinforcement

4.7.1 Design of Links in X Direction

V_s = 195 – 114 = 81 kN

Spacing of 4 legged 8 Φ Links

= 4 x 50 x 0.87 x 415 x 247.5 /81,000

= 221 mm

4.7.2 Design of Links in Y Direction

V_s = 176 – 123 = 53 kN

Spacing of 2 Legged 8 Φ Links

= 2 x 50 x 0.87 x 415 x 447.5 /53,000

= 305 mm

4.7.3 Nominal Links

The spacing of hoops shall not exceed half the least lateral dimension of the column, i.e., 300/ 2 = 150 mm.

(Clause 7.3.3 of IS 13920: 1993)

Provide 8 Φ links @ 150 c/c in mid-height portion of column.

4.7.4 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

$$A_{sh} = \frac{0.18 \times S \times h \times f_{ck}}{f_y} \left(\frac{A_g}{A_k} - 1 \right)$$

(Clause 7.4.8 of IS 13920: 1993).

Assuming h = longer dimension of the rectangular link measured to its outer face

= ((500- 40 – 40- 25)/3 + 25 +10 +10 =177 mm or 300 - 40 – 40 +10 +10 = 240 mm, whichever is higher, i.e., h = 240.

A_g = 300 x 500 = 1,50,000 mm^2

A_k = (300 –2 x 40 +2 x10) x (500-2 x 40 + 2 x 10)

= 240 x 440

= 1,05,600 mm^2

Assuming 10Φ stirrup, A_{sh} = 78.54 mm^2

$$78.54 = \frac{0.18 \times S \times 240 \times 20}{415} \left(\frac{1{,}50{,}000}{1{,}05{,}600} - 1 \right)$$

Substituting we get S = 90 mm.

Link spacing for confining zone shall not exceed:

¼ of minimum column dimension i.e,

300 / 4 = 75 mm

But need not be less than 75 mm nor more than 100 mm.

(Clause 7.4.6 of IS 13920: 1993).

Provide 10Φ confining links @ 75 c/c for a distance L_o (Refer figure 4.5), which shall not be less than:

a. Larger lateral dimension = 500 mm

b.1/6 of clear span = (3,000 –500)/6 = 417 mm

c. 450 mm

(Clause 7.4.1 of IS 13920: 1993).

Provide confining reinforcement for a distance of L_o = 500 mm on either side of the joint. (Refer Figure 4.5)

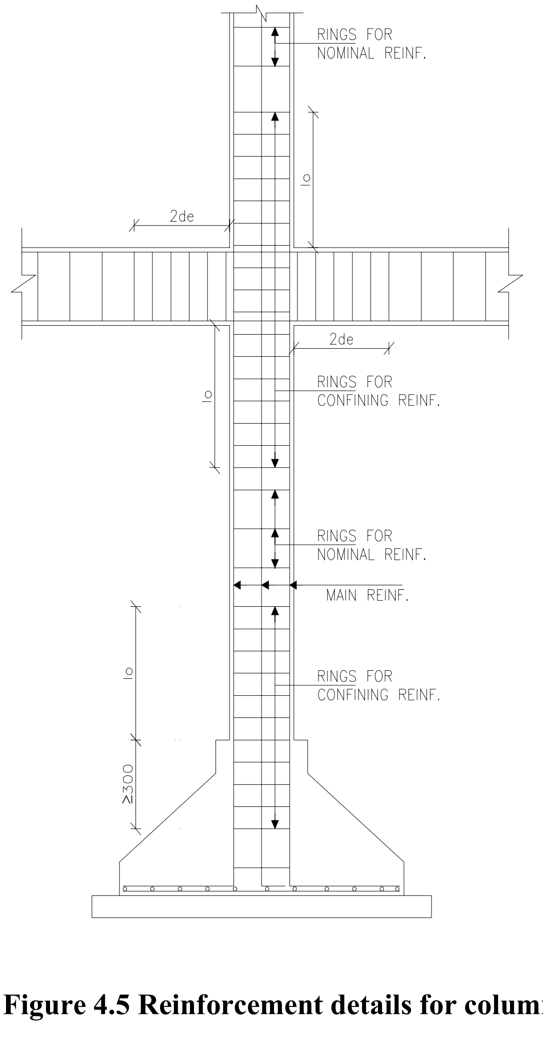


Figure 4.5 Reinforcement details for column

4.8 Check as Flexural Member for Load Comb 0.9 DL + 1.5 EQY

Factored moment = 156 kN-m

Effective depth = 500 – 40 - 25/2 = 447.5mm

$$\frac{M_u}{bd^2} = \frac{156 \times 10^6}{300 \times 447.5^2} = 2.60$$

Referring to Table 2 of SP: 16, we get

A_{st} = 0.883%

= 0.883 x 300 x 447.5 / 100

= 1,185 mm^2

A_{st} provided on one face = 3 – 25Φ

= 3 x 491

= 1,473 mm^2 >1,185 mm^2

Hence, ok.

CAUTION

Note, however, that the column designed above has not been checked for requirements related to the joint region, which are being incorporated in the new edition of IS 13920. These provisions are illustrated in Examples 5-8 and may require modifications in column size and / or longitudinal reinforcement.

Table 4.8 Comparison of bill of quantities of steel in column

Description	Detailing as per IS 13920: 1993 (Seismic loads as per R = 5)	Detailing as per IS 456: 2000 (Seismic loads as per R = 5)	Detailing as per IS 456: 2000 (Seismic loads as per R = 3)
Links (kg)	25	10	Column needs to be redesigned.
Main steel (kg)	132	132	

Example 5 – Interior Beam – Column Joint Design for Zone -V

5. Problem Statement:

Detailed design as per draft revision of IS 13920:1993 of an interior joint in an intermediate RC frame is explained for the ground plus four storey RC office building of Example-1 (Refer Figures 1.1-1.4)

Solution:

5.1. Preliminary Data

The joint of column marked in Figure 5.1 for Frame 2 is considered for design. The plan of the building and the sectional elevation of a typical RC frame is shown in Figures 5.1 and 5.2.

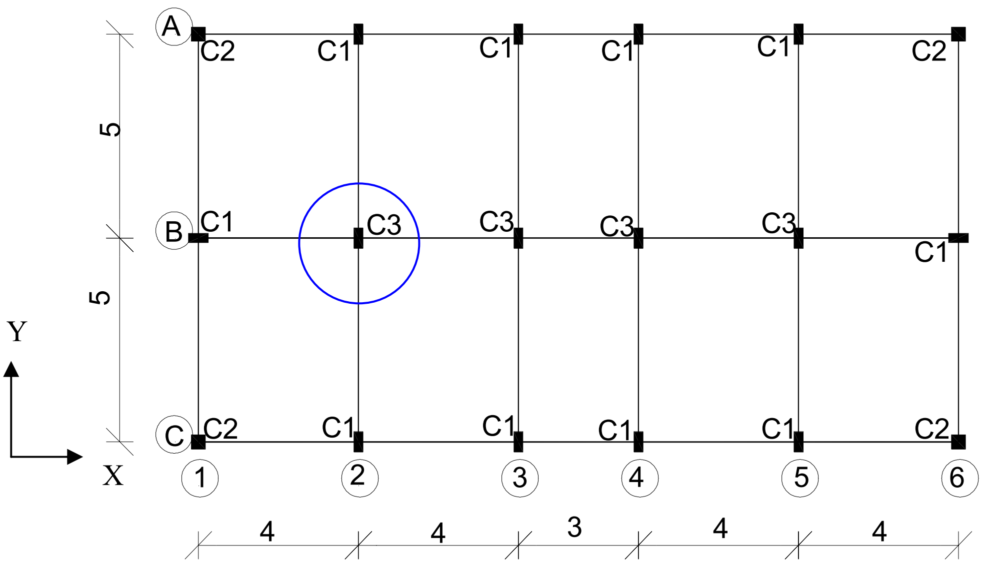


Figure 5. 1 Plan of building (All dimensions in meters)

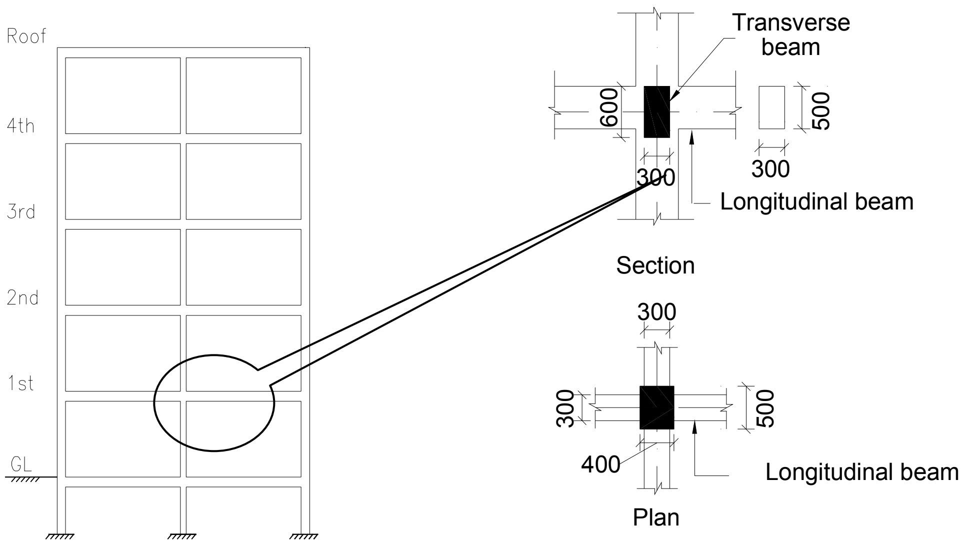


Figure 5.2 Column location in elevation

5.2. Design Data

The details of the column and beam reinforcement meeting at the joint are shown in Figure 5.3.

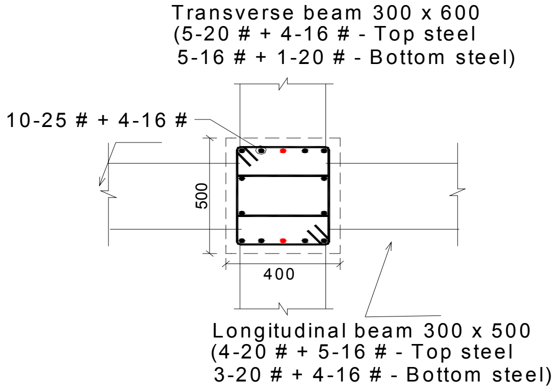


Figure 5.3 Reinforcement details for column and beams.

The transverse beam of size 300 x 600 is reinforced with 5-20Φ + 4-16Φ (2,374 mm^2, i.e., 1.487%) at top and 1-20Φ + 5-16Φ (1,319 mm^2, i.e., 0.83%) at bottom. The hogging and sagging moment capacity is evaluated as 377 kN-m and 246 kN-m, respectively.

The longitudinal beam of size 300 x 500 is reinforced with 4-20Φ + 5-16Φ (2,261 mm^2, i.e. 1.74%) at top and 3-20Φ + 4-16Φ (1,746 mm^2 i.e. 1.34%) at bottom. The hogging and sagging moment capacity is evaluated as 288 kN-m and 221 kN-m, respectively.

5.3 Check for Earthquake in Y-Direction

5.4.1 Joint Shear

The joint shear equilibrium is shown in Figure 5.4

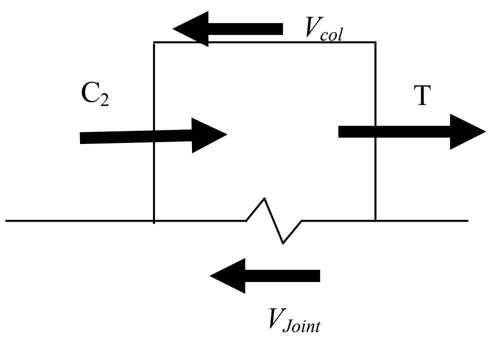


Figure 5.4 Joint shear

Column Shear

The column shear is as explained below. (Refer Figures 5.5 and 5.6 for sway to right and left conditions respectively).

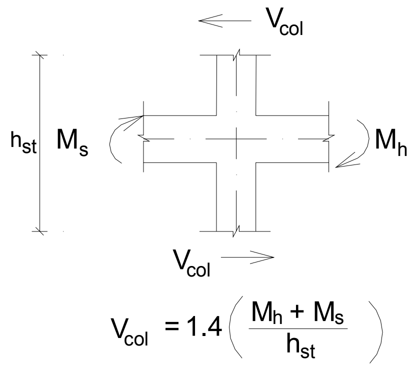


Figure 5.5 Column with sway to right

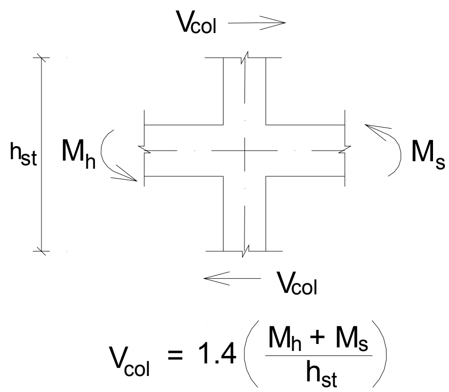


Figure 5.6 Column with sway to left

For both the above cases,

$$V_{col} = 1.4\left(\frac{M_s + M_h}{h_{st}}\right)$$

$$= 1.4 \times \left(\frac{377 + 246}{3}\right)$$

$$= 291 \text{ kN}$$

Force Developed in Beam Reinforcement

Figures 5.7 and 5.8 show the development of forces in the joint due to beam reinforcement, for sway to right and left, respectively.

Force developed in the top bars

$T_1 = A_{st}$ x 1.25 x f_y

= 2,374 x 1.25 x 415 /1,000

= 1,231 kN = 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]

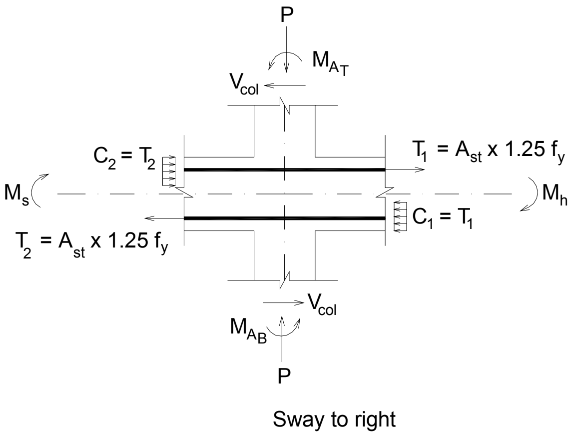


Figure 5.7 Free body diagram of the joint

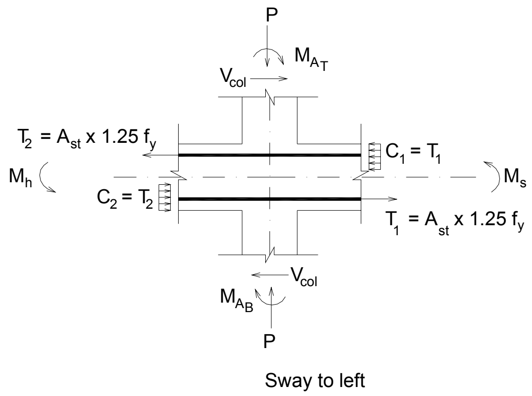


Figure 5.8 Free body diagram of the joint

Force developed in the bottom bars

$T_2 = A_{st} \times 1.25 \times f_y$

$= 1{,}319 \times 1.25 \times 415 /1{,}000$

$= 684$ kN $= C_2$

Referring to Figure 5.4,

$V_{Joint} = T_1 + C_2 - V_{col}$

$= 1{,}231 + 684 - 291$

$= 1{,}624$ kN

Maximum value of T_1 and minimum value of V_{col} are used in the above equation.

5.4.2 Check for Joint Shear Strength

The effective width provisions for joints are shown in Figure 5.9. The calculation of the effective width of the joint and the design shear strength of the joint is based on the draft revision of IS 13920:1993.

The effective width of the joint is lesser of the

i) $b_j = b_b + 0.5 \times h$

ii) $b_j = b_c$

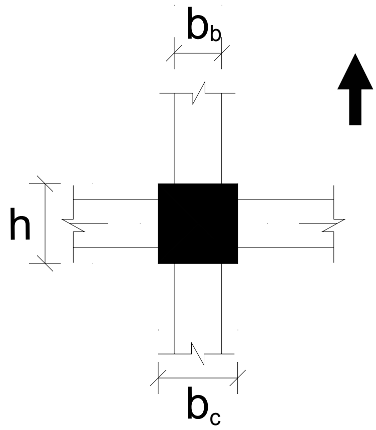


Figure 5.9 Effective width for joint

$b_j = b_b + h/2$

$= 300 + 500 /2$

$= 550$ mm

Or $b_j = b_c = 400$ mm

Take effective width as 400 mm.

h = full depth of column

$= 500$ mm

Effective shear area of the joint $= A_c = b_j\ h$

Shear strength $= 1.2 \sqrt{f_{ck}}\ A_c$

Shear strength of joint confined on two opposite faces, as per Clause 8.1.3 of draft revision of IS 13920:1993

$= 1.2 \times \sqrt{20} \times 400 \times 500 /1{,}000$

$= 1{,}073$ kN $< 1{,}624$kN

Hence, not Safe.

5.4.3 Check for Flexural Strength Ratio

The hogging and sagging moment capacities of the transverse beam are as 377 kN-m and 246 kN - m, respectively.

The column is reinforced with 10 - 25Φ +

4 - 16Φ bars with total $A_{sc} = 5{,}714$ mm^2 i.e.

5,714 x 100 / (400 x 500) = 2.852%.

p/f_{ck} = 2.852 / 20 = 0.1426

It is conservative here to calculate the moment capacity of column with zero axial loads. In actual practice it is desirable to take minimum $\frac{M_u}{f_{ck} b D^2}$ corresponding to actual $\frac{P_u}{f_{ck} b D}$ obtained from different load combinations. Referring to chart 44 of SP: 16, corresponding to $\frac{P_u}{f_{ck} b D}$ = 0 .00 at A_B to be on the conservative side, for p/f_{ck} = 0.143 and d'/D = (40 + 25 /2) / 500 = 0.105, we get

$$\frac{M_u}{f_{ck} b D^2} = 0.19.$$

M_u = 0.19 x 20 x 400 x 500 x 500 / 1x10^6

= 380 kN-m

Referring to Figure 5.10, the joint is checked for strong column - weak beam.

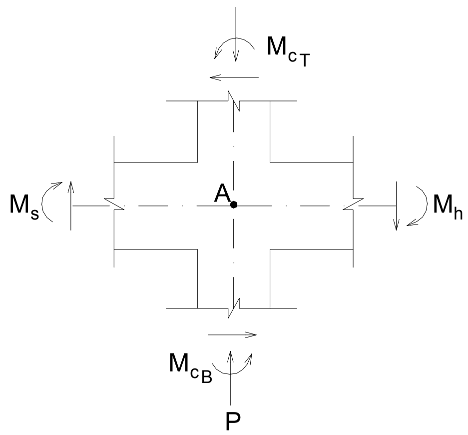


Figure 5.10 Check for strong column - weak beam condition

$\sum M_c$ = 380+380 = 760 kN-m

$\sum M_b$ = 377 + 246 = 623 kN-m

The ratio of $\frac{\sum M_c}{\sum M_b}$ = 760 /623 = 1.22 > 1.1

Hence, requirement of strong column-weak beam condition as per proposed draft IS 13920 is satisfied.

(Clause 7.2.1 of IS 13920 proposed draft)

5.4 Check for Earthquake in X Direction

5.4.1 Joint Shear

The joint shear equilibrium is shown in Figure 5.4.

Shear due to formation of plastic hinge in beams

Referring to Figures 5.5 and 5.6, for both the cases,

$$V_{col} = 1.4 \left(\frac{M_s + M_h}{h_{st}} \right)$$

$$= 1.4 \times \left(\frac{288 + 221}{3} \right)$$

= 238 kN

Force Developed in Beam Reinforcement

Referring to Figures 5.7 and 5.8, we get,

$T_1 = A_{st}$ x 1.25 x f_y

= 2,261 x 1.25 x 415/1,000

= 1,173 kN = C_1

$T_2 = A_{st}$ x 1.25 x f_y

= 1,746 x 1.25 x 415/1,000

= 905 kN = C_2

The joint shear is evaluated considering maximum T_1 and minimum V_{col}.

$V_{Joint} = T_1 + C_2 - V_{col}$

= 1,173 + 905-238

= 1,840 kN

5.4.2 Check for joint shear strength

$b_j = b_b$ + h/2

= 300 + 400 /2 = 500

or $b_j = b_c$ = 500 mm

Adopt lesser of the two values i.e. b_j = 500 mm

h = Depth of column or full depth of beam

= 400 mm

Shear strength of joint not confined as per Clause 8.1.3 of draft revision (b_c > ¾ b_b on two opposite faces) of IS 13920:1993

Shear strength = 1.0 $\sqrt{f_{ck}}\ A_c$

= 1.0 x $\sqrt{20}$ x 500 x 400 /1000

= 894 kN < 1,840 kN

Hence not safe.

(Clause 8.3 of IS 13920 proposed draft)

5.4.3 Check for flexural strength ratio

The limiting hogging and sagging moments capacity of the longitudinal beam is 288 kN-m and 221 kN-m, respectively.

It is conservative here to calculate moment capacity of column with zero axial loads. In actual practice it is desirable to take minimum $\frac{M_u}{f_{ck}bD^2}$ corresponding to actual $\frac{P_u}{f_{ck}bD}$ obtained from different load combinations. Referring to chart 44, corresponding to $\frac{P_u}{f_{ck}bD}$ = 0.00 at A_B, for p/f_{ck}= 0.1426 and d'/D = (40 + 25/2) /400 = 0.13125, we get $\frac{M_u}{f_{ck}bD^2}$ = 0.178.

M_u= 0.178 x 20 x 400 x 400 x 500 /1x 10^6 = 284 kN-m

$\sum M_c$ = 284+284 = 568 kN-m

$\sum M_b$ = 288+221 = 509 kN-m

The ratio of $\frac{\sum M_c}{\sum M_b}$ = 568/509 = 1.11 > 1.1

Hence, requirement of strong column-weak beam condition is satisfied.

(Clause 7.2.1 of proposed draft IS13920)

5.5 Revision

As can be seen from the checks in section 5.3.2, and 5.4.2, the joint is not safe. In such cases the following three alternatives can be tried.

i) 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.

ii) 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.

iii) 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 column from 400 x 500 to 600 x 600 and longitudinal beam size from 300x500 to 300x600. Member forces are taken as calculated earlier without reanalysis of the structure. In practice the structure may be reanalyzed.

The redesigned longitudinal beam of size 300 x 600 is reinforced with 6-20Φ (1,884 mm2, i.e. 1.18%) at top and 2-20Φ + 3-16Φ (1,231 mm2 i.e. 0.77%) at bottom. The hogging and sagging moment capacity is evaluated as 293 kN-m and 229 kN-m, respectively.

The column is redesigned to 600 x 600 with 4-20ϕ + 10 -16ϕ bars (3,266 mm^2, 0.9% steel) as main longitudinal reinforcement. This design is made based upon the existing forces without reanalysis.

As per analysis results, the column size now requires 4- 20ϕ + 10-16ϕ bars (3,266 mm^2, 0.9% steel) as main longitudinal steel. The value of p/f_{ck} = 0.90/20 = 0.045.

The $\sum M_c$ required in transverse direction is 623 x 1.1 = 685 kN-m and 1.1 x 522 (i.e. 293+223)= 574 kN-m in longitudinal direction.

Hence required moment capacity for column is M_c = 685/2 = 343 kN-m in Y direction and 574 / 2 = 287 kN-m in X direction.

(Clause 7.2.1 of IS 13920 proposed draft)

Using SP 16 the steel required to get the above moment capacity of column is calculated as 1.1%. Hence revise the main longitudinal steel to 8-20ϕ + 8-16ϕ bars (4,120 mm^2, 1.14% steel). The revised reinforcement details are shown in Figure 5.11.

The above column section will satisfy the flexural strength check.

While redesigning the column few load combinations may give an axial stress less than $0.1 f_{ck}$. The section needs to be checked for flexure for these load combinations.

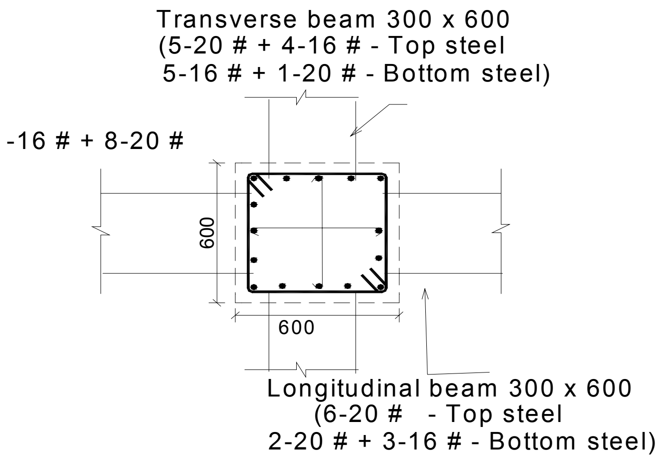


Figure 5.11 Revised reinforcement details for column and beams

Check for Earthquake in Y Direction

$b_j = b_b + h/2$

$= 300 + 600/2$

$= 600$ mm

Or $b_j = b_c = 600$ mm

h = full depth of column

$= 600$ mm

Take $b_j = 600$ mm

Shear strength $= 1.0 \sqrt{f_{ck}}\, A_c$

$= 1.0 \times \sqrt{20} \times 600 \times 600/1{,}000$

$= 1{,}610$ kN $\approx 1{,}624$ kN

Hence, ok.

(Clause 8.3 of IS 13920:1993 proposed draft)

Check for Earthquake in X Direction

Referring to Figures 5.5 and 5.6, for both the cases, shear due to formation of plastic hinge in beams is

$$V_{col} = 1.4\left(\frac{M_s + M_h}{h_{st}}\right)$$

$$= 1.4 \times \left(\frac{293 + 229}{3}\right)$$

$= 244$ kN

Force Developed in Beam Reinforcement

Referring to Figures 5.7 and 5.8, we get,

$T_1 = A_{st}$ x 1.25 x f_y

= 1,884 x 1.25 x 415/1,000

= 978kN = C_1

$T_2 = A_{st}$ x 1.25 x f_y

= 1,231 x 1.25 x 415/1,000

= 638 kN = C_2

The joint shear is evaluated considering maximum T_1 and minimum V_{col}.

$V_{Joint} = T_1 + C_2 - V_{col}$

= 978 + 638-244

= 1,372 kN

$b_j = b_b + h/2$

$= 300 + 600/2$

$= 600$ mm

Or $b_j = b_c = 600$ mm

h = full depth of column

$= 600$ mm

Take $b_j = 600$ mm

Shear strength $= 1.0 \sqrt{f_{ck}}\, A_c$

$= 1.0 \times \sqrt{20} \times 600 \times 600/1{,}000$

$= 1{,}610$ kN $< 1{,}372$ kN

Hence, ok. .

5.6 Confining Links

In this case with the column dimensions revised to 600 x 600, the width of beam is 300 mm, which is less than 3/4 width of column i.e. 3/4 x 600 = 450 mm. Hence, full confining reinforcement is required in the joint.

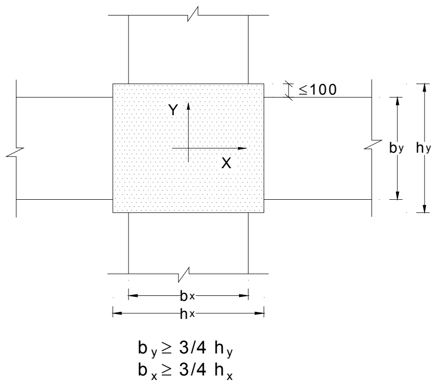


Figure 5.12 Confinement of joint concrete by beams

The spacing of links for the confining zone shall not exceed:

i) ¼ of minimum column dimension i.e,

600 / 4 = 150 mm

ii) But need not be less than 75 mm nor more than 100 mm.

(Clause 7.4.6 of IS 13920:1993)

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

$$A_{sh} = \frac{0.18 \times S \times h \times f_{ck}}{f_y}\left(\frac{A_g}{A_k} - 1\right)$$

(Clause 7.4.8 of IS 13920:1993)

Assuming

h = longer dimension of the rectangular confining measured to its outer face

= (600 - 40 - 40 -20) / 2 +8 x 2 + 20 = 286 mm

A_g = 600 x 600 = 3,60,000 mm^2

A_k = (600-2 x 40 +2 x8) x (600- 2 x 40 +2 x 8)

= 536 x 536

= 2, 87,296 mm^2

A_{sh} = 50 mm^2

$$50 = \frac{0.18 \times S \times 286 \times 20}{f_y}\left(\frac{3{,}60{,}000}{2{,}87{,}296} - 1\right)$$

S = 80 mm

Provide 8Φ confining links @ 80 c/c in the joint.

Example 6 — Exterior Beam-Column Joint Design for Zone V

6. Problem Statement:

Detailed design as per draft revision of IS 13920:1993 of an exterior joint in an intermediate RC frame is explained for the ground plus four storey RC office building of Example-1 (Refer Figures 1.1-1.4)

Solution:

6.1 Preliminary Data

The joint of column marked in Figure 6.1 for Frame 2 is considered for design. The plan of the building and the sectional elevation of a typical RC frame are shown in Figures 6.1 and 6.2.

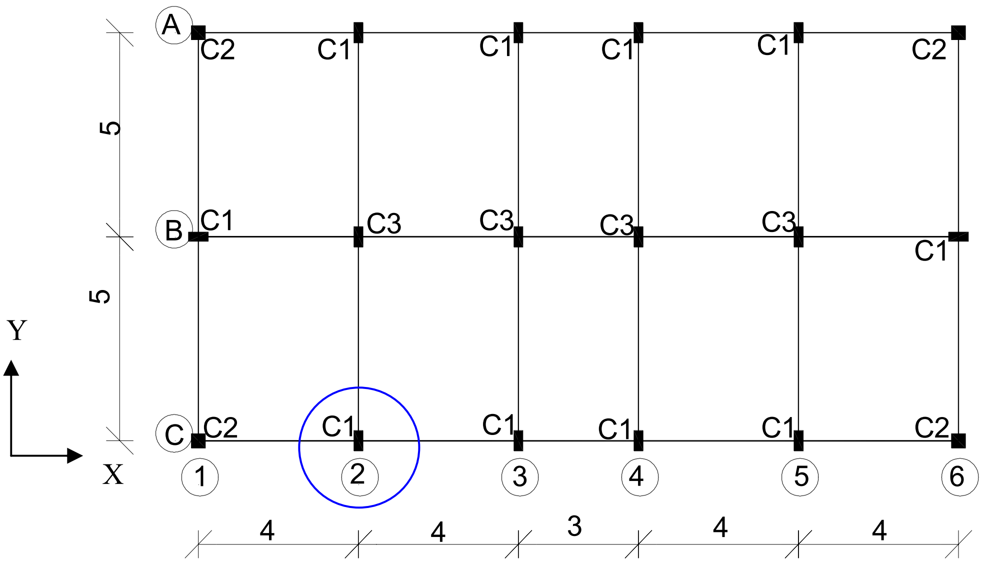


Figure 6. 1 Plan of building (All dimensions in meters)

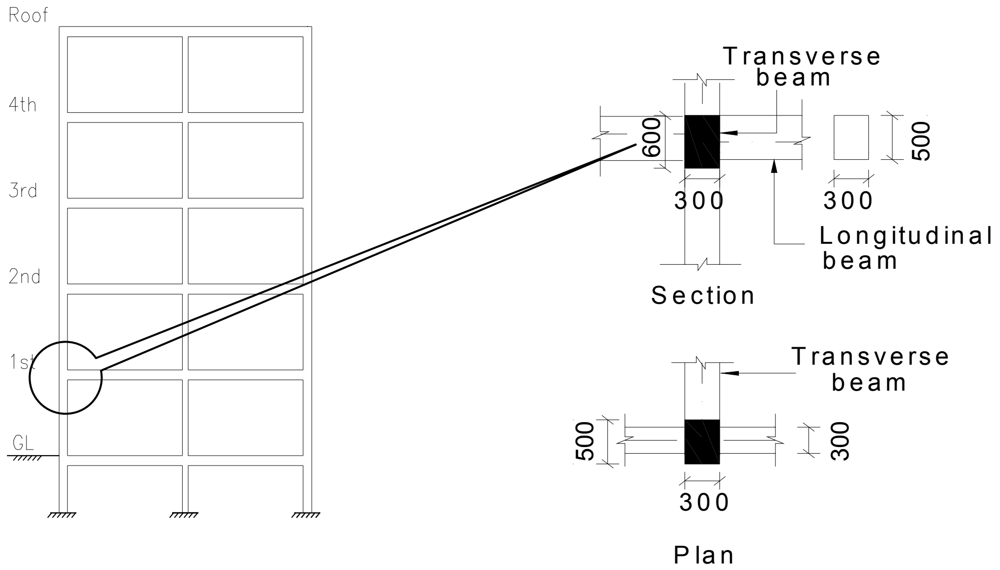


Figure 6-2 Column location in elevation

6.2 Design Data

The details of the column and beam reinforcement meeting at the joint are shown in Figure 6.3.

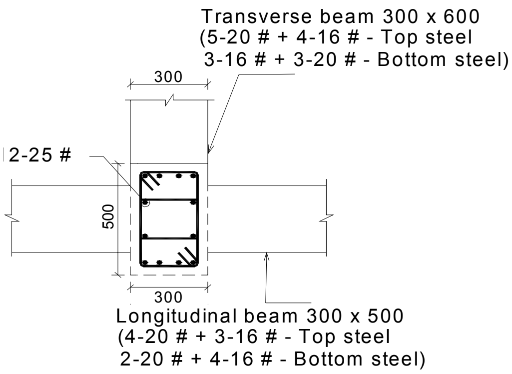


Figure 6.3 Reinforcement details for column and beams.

The transverse beam of size 300 x 600 is reinforced with 5-20Φ + 4-16Φ (2,374 mm^2, i.e. 1.487 %) at top and 3-20Φ + 3-16Φ (1,545 mm^2, i.e., 0.97%) at bottom. The hogging and sagging moment capacity is evaluated as 377 kN-m and 286 kN-m, respectively.

The longitudinal beam of size 300 x 500 is reinforced with 4-20Φ +3-16Φ (1,859 mm^2, i.e., 1.43%) at top and 2-20Φ + 4-16Φ (1,432 mm^2, i.e., 1.10%) at bottom. The hogging and sagging moment capacities are evaluated as 247 kN-m and 180 kN-m respectively.

6.3 Check for Earthquake in Y-Direction

6.3.1 Joint Shear

The joint shear equilibrium is shown in Figure 6.4

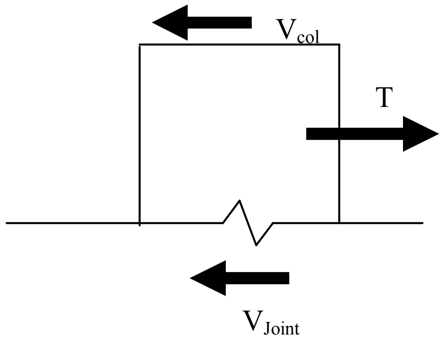


Figure 6.4 Joint shear

Column Shear

The column shear is evaluated as explained below. (Refer Figures 6.5 and 6.6 for sway to right and left, conditions respectively).

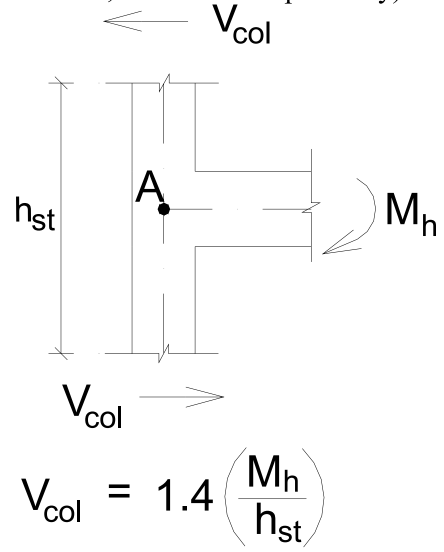


Figure 6.5 Column with sway to right

$$V_{col} = 1.4\left(\frac{M_h}{h_{st}}\right)$$

$$= 1.4 \times \left(\frac{377}{3}\right)$$

= 176 kN

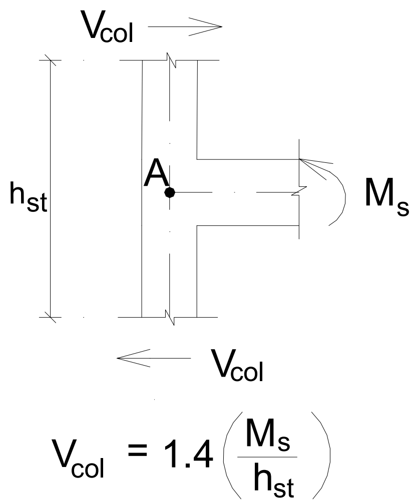


Figure 6.6 Column with sway to left

$$V_{col} = 1.4\left(\frac{M_s}{h_{st}}\right)$$

$$= 1.4 \times \left(\frac{286}{3}\right)$$

= 133 kN

Force Developed in Beam Reinforcement

Figures 6.7 and 6.8 shows the development of forces in the joint due to beam reinforcement, for sway to right and left, respectively.

Force developed in top bars

$T_1 = A_{st}$ x 1.25 x f_y

= 2,374 x1.25 x 415 /1,000

= 1,231 kN

The factor 1.25 is to account for the actual ultimate strength being higher than the actual yield strength (Draft revision of IS 13920:1993)

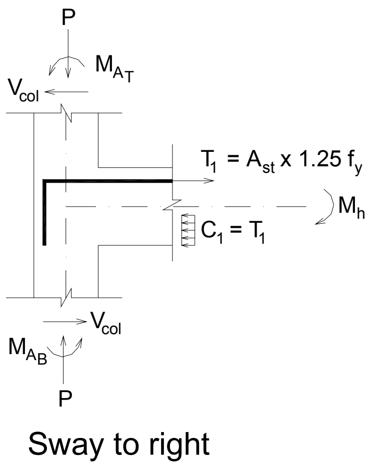


Figure 6.7 Free body diagram of the joint

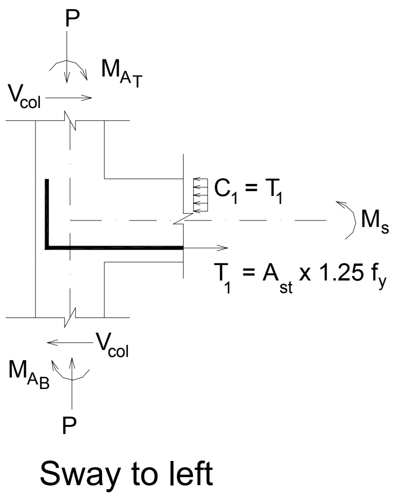


Figure 6.8 Free body diagram of the joint

Force developed in bottom bars

$T_1 = A_{st}$ x 1.25 x f_y

= 1,545 x1.25 x 415 /1,000

= 801 kN.

Referring to Figure 6.4,

$V_{Joint} = T_1 - V_{col}$

= 1,231-176

= 1,055 kN for sway to right.

For sway to left,

= 801 -133 = 668 kN

6.4.2 *Check for Joint Shear Strength*

The effective width provisions for joints are shown in Figure 6.9.

The calculation of the effective width of the joint and the design shear strength of the joint is based on the draft revision of IS 13920:1993

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

i) $b_j = b_b + 0.5 \times h$

ii) $b_j = b_c$

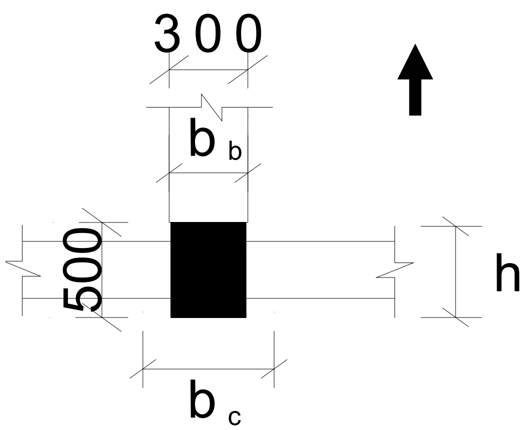


Figure 6.9 Effective width for joint

$b_j = b_b + h/2$

= 300 + 500 /2

= 550 mm

$b_j = b_c$

= 300 mm

Take effective width of joint as 300 mm

h = full depth of column

= 500 mm

Effective area of joint resisting shear = $A_c = b_j h$

Shear strength of joint not confined (b_c < ¾ b_b only on one faces and b_c > ¾ b_b on other two faces) as per Clause 8.1.3 of draft revision of IS 13920:1993

Shear strength = 1.0 $\sqrt{f_{ck}}$ A_c

Shear strength of joint confined on three faces or on two opposite faces, as per draft revision IS 13920:1993

= 1.0 x $\sqrt{20}$ x 300 x 500 /1,000

= 671 kN < 1,055 kN

Hence, not safe.

(Clause 8.3 of IS 13920 proposed draft)

6.4.3 Check for Flexural Strength Ratio

The hogging and sagging moment capacities of transverse beam are 377 kN-m and 286 kN-m, respectively.

Column is reinforced with 10-25Φ + 4-16Φ bars with total A_{sc} = 5,714 mm^2,

i.e., 5,714 x100 / (300 x 500) = 3.8%.

p/f_{ck} = 3.8 /20 = 0.19

It is conservative here to calculate the moment capacity of column with zero axial loads. In actual practice it is desirable to take minimum $\frac{M_u}{f_{ck}bD^2}$ corresponding to actual $\frac{P_u}{f_{ck}bD}$ obtained from different load combinations. Referring to chart 44 of SP: 16, corresponding to $\frac{P_u}{f_{ck}bD}$ = 0.00 at A_B, p/f_{ck} = 0.19 and d'/D = (40 +25 /2) / 500 = 0.105, we get

$\frac{M_u}{f_{ck}bD^2}$ = 0.235.

M_c = 0.235 x 20 x 300 x 500 x 500 / (1x 10^6)

= 353 kN-m

$\sum M_c$ = 353 + 353 = 706 kN-m

$\sum M_b$ = 377 kN-m

(Maximum moment resistance is considered)

As shown in Figure 6.10, the beam-column joint is checked for strong column-weak beam condition.

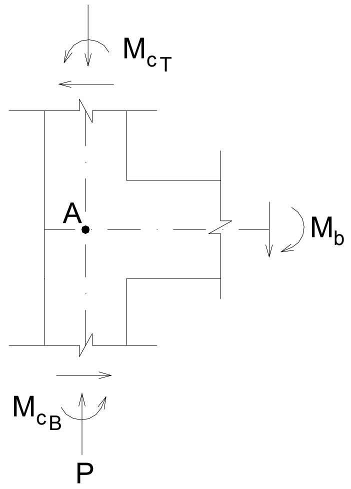


Figure 6.10 Check for strong column - weak beam condition

The ratio of $\frac{\sum M_c}{\sum M_b}$ = 706 / 377 = 1.87 > 1.1

Hence, requirement of strong column-weak beam condition as per draft revision of IS 13920:1993 is satisfied

6.4 Check for Earthquake in X- direction

6.4.1 Joint Shear

The joint shear equilibrium is shown in Figure 6.11.

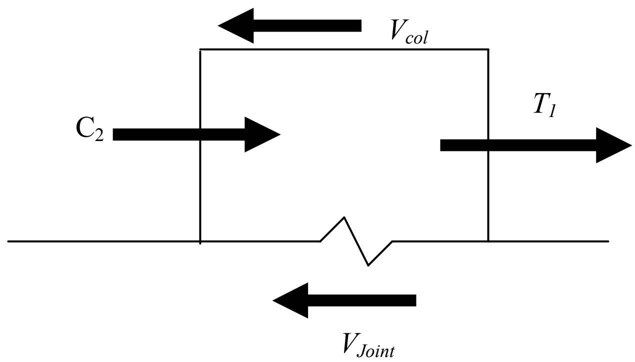


Figure 6.11 Joint shear

Shear due to formation of plastic hinges in beams

The column shear is evaluated as below. Refer Figures 6.12 and 6.13 for sway to right and left, respectively.

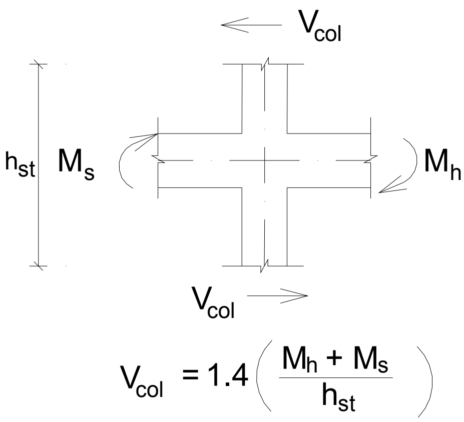


Figure 6.12 Column with sway to right

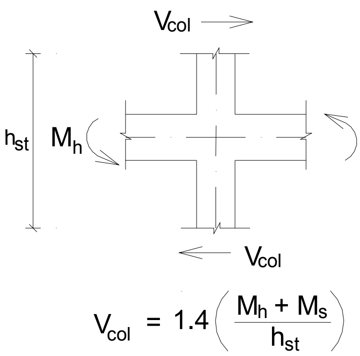


Figure 6.13 Column with sway to left

$$V_{col} = 1.4\left(\frac{M_s + M_h}{h_{st}}\right)$$

$$= 1.4 \times \left(\frac{247 + 180}{3}\right)$$

= 200 kN

Force developed in beam reinforcement

Figures 6.14 and 6.15 show the development of forces in the joint due to beam reinforcement, for sway to right and left, respectively.

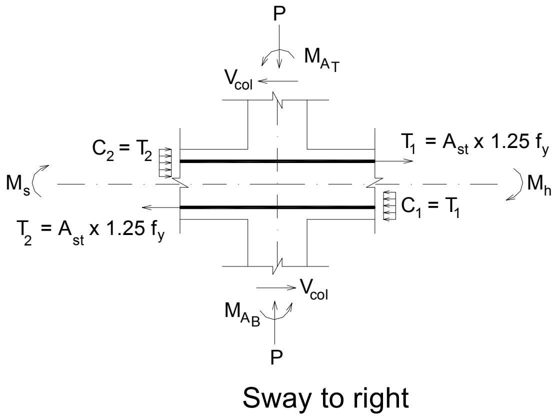


Figure 6.14 Free body diagram of the joint

$T_1 = A_{st}$ x 1.25 x f_y

= 1,859 x1.25 x 415 /1,000

= 964 kN = C_1

$T_2 = A_{st}$ x 1.25 x f_y

= 1,432 x1.25 x 415 /1,000

= 742 kN = C_2

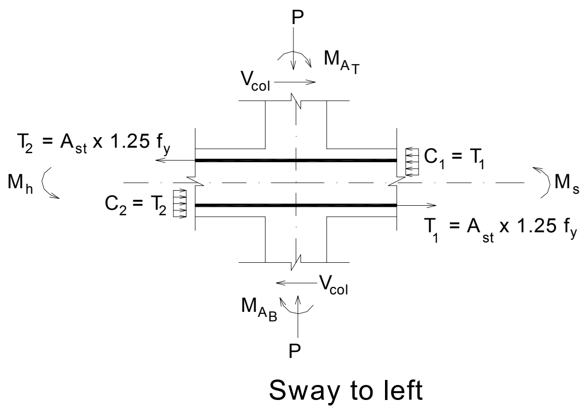


Figure 6.15 Free body diagram of the joint

$T_1 = A_{st}$ x 1.25 x f_y

= 1,432 x 1.25 x 415 /1,000

= 742 kN = C_1

$T_2 = A_{st}$ x 1.25 x f_y

= 1,859 x 1.25 x 415 /1,000

= 964 kN = C_2

Referring to Figure 6.11,

$V_{Joint} = T_1 + C_2 - V_{col}$

= 964 + 742 - 200

= 1,506 kN

6.5.2 Check for Joint Shear Strength

The effective width calculations for the joint are based on Figure 6.15.

$b_j = b_b + h/2$

$= 300 + 300/2$

$= 450$ mm

$b_j = b_c$

$= 500$ mm

Take $b_j = 450$ mm

h = full depth of column

$= 300$ mm

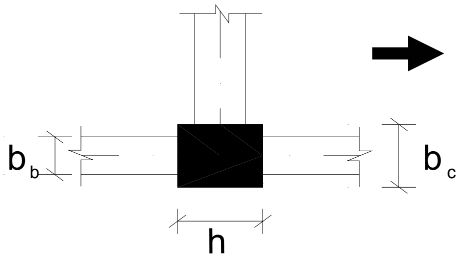


Figure 6.16 Effective width of joint

Shear strength of joint not confined ($b_c > ¾\, b_b$ on both opposite faces) as per Clause 8.1.3 of draft revision of IS 13920:1993

Shear strength = $1.0\sqrt{f_{ck}}\,A_c$

$= 1.0 \times \sqrt{20} \times 450 \times 300/1{,}000$

$= 604$ kN << 1,503 kN

Hence, not safe.

6.5.3 Check for Flexural Strength Ratio

The hogging and sagging moment capacities are evaluated as 247 kN-m and 188 kN-m, respectively.

The limiting moment capacity of the column calculated using SP: 16 is 212 kN-m

As shown in Figure 6.17, the beam-column joint is checked for strong column-weak beam condition.

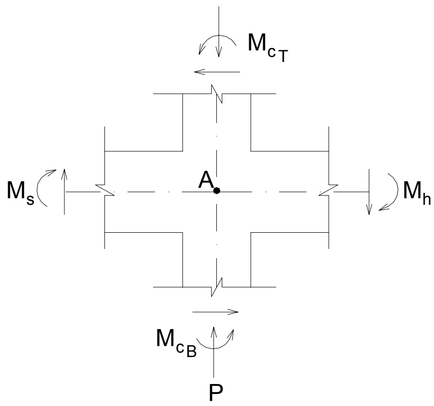


Figure 6.17 Check for strong column-weak beam condition

$\sum M_c = 212 + 212 = 424$ kN-m

$\sum M_b = 247 + 180 = 427$ kN-m

The ratio of $\dfrac{\sum M_c}{\sum M_b} = 424/427 = 0.99 < 1.1$

Strong column-weak beam condition is not satisfied

Hence, not ok.

(Clause 7.2.1 of IS 13920 proposed draft)

6.5 Revision

As can be seen from the checks in sections 6.4.2, 6.5.2 and 6.5.3, the joint is not safe. In such cases the following three alternatives can be tried separately or in combination.

i) 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.

ii) Increase the size of the beam section. If this option is adopted, it is advisable to increase 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 on the beam, increase in beam width can be considered if the difference between the shear strength of joint and joint shear is small.

iii) 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 column size from 300 x 500 to 400 x 500 and the beam depth from 600 mm to 750 mm and 500 mm to 600 mm for the transverse and longitudinal beams respectively. Member forces are taken as calculated earlier without reanalysis of the structure. In practice the structure may be reanalyzed.

The transverse beam is redesigned to 300 x 750 with 4-16ϕ + 3-20 ϕ +1- 12ϕ at top (1,859 mm^2, 0.91% steel) and 3-16ϕ + 2-20ϕ at bottom (1,231 mm^2, 0.60% steel). Using SP: 16, the moment capacity of the beam is calculated as M_h = 371 kN-m and M_s = 297 kN-m, respectively.

Similarly, the longitudinal beam is redesigned to 300 x 750 with 3-16ϕ + 3-16ϕ at top (1,206 mm^2, 0.59% steel) and 3-16ϕ + 1-16ϕ at bottom (804 mm^2, 0.39% steel). Using SP: 16, the moment capacity of the beam section is calculated as M_h = 265 kN-m and M_s = 184 kN-m, respectively.

As per analysis results, the column size now requires 14-16ϕ bars (2,814 mm^2, 1.41% steel) as main longitudinal steel. The value of p/f_{ck} = 1.41/20 = 0.07.

The ΣM_c required in transverse direction is 371 x 1.1 = 408 kN-m and 1.1 (226 + 208) = 477 kN-m in longitudinal direction.

Hence, required moment capacity for column is M_c = 415/2 = 208 kN-m in transverse direction and 449/ 2 = 225 kN-m in longitudinal direction.

(Clause 7.2.1 of IS 13920 proposed draft)

Using SP-16, the steel required to get the above moment capacity of column is calculated as 2%. Hence, revise the main longitudinal steel to 14-20ϕ bars (4396 mm^2, 2.2% steel). The revised reinforcement details are shown in Figure 6.17.

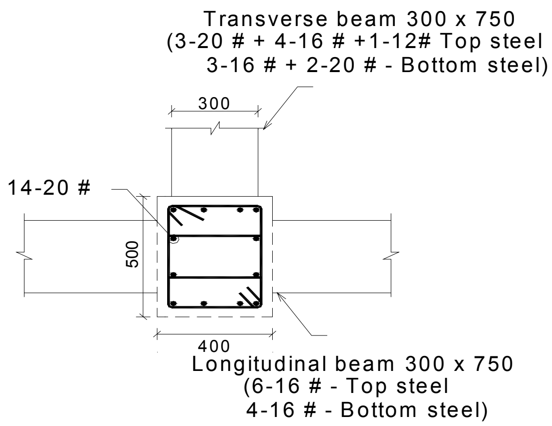


Figure 6.18 Revised reinforcement details for column and beams.

Check for Earthquake in Y Direction

Column sway to right

The column shear is evaluated as below.

$$V_{col} = 1.4\left(\frac{M_s}{h_{st}}\right)$$

$$= 1.4 \times \left(\frac{371}{3}\right)$$

= 173 kN

Max. force developed in the top bars, T_1,

$T_1 = A_{st}$ x 1.25 x f_y

= 1,859 x 1.25 x 415 /1,000

= 964 kN

Joint shear is calculated as

$V_{Joint} = T_1 - V_{col}$

= 964-173

= 791 kN

$b_J = b_b + h/2$

= 300 + 500 /2

= 550 mm

or

$b_J = b_c$

= 400 mm

Take b_J = 400 mm

h = full depth of column

= 500 mm

Shear strength = 1.0 $\sqrt{f_{ck}}$ A_c

= 1.0 x $\sqrt{20}$ x 400 x 500 /1,000

= 894 kN > 791 kN

Hence, ok.

Check for Earthquake in X direction

Column sway to right

The column shear is evaluated as below.

$$V_{col} = 1.4\left(\frac{M_h + M_s}{h_{st}}\right)$$

$$= 1.4 \times \left(\frac{265 + 184}{3} \right)$$

= 314 kN

Max. Force developed in the top bars

$T_1 = A_{st}$ x 1.25 x f_y

= 1,206 x1.25 x 415 /1,000

= 626 kN = C_1

$T_2 = A_{st}$ x 1.25 x f_y

= 804 x1.25 x 415 /1,000

= 403 kN = C_2

$V_{Joint} = T_1 + C_2 - V_{col}$

= 626 + 403 - 314

= 714 kN

$b_J = b_b + h/2$

= 300 + 500 /2

= 550 mm

or

$b_J = b_c$

= 500 mm

Take b_J = 500 mm

h = full depth of column

= 400 mm

Shear strength = 1.0 $\sqrt{f_{ck}}$ A_c

= 1.0 x $\sqrt{20}$ x 500 x 400 /1,000

= 894 kN > 714 kN

Hence, ok.

6.6 Confining Links

The column dimensions have been revised to 400 x 500, and the width of beam is 300 mm, which is less than ¾ of column width (i.e. 3/4 x 500 = 375 mm) in one direction. An offset of (500 - 300)/2 = 100 mm of concrete is exposed on either side of beam As per Clause 8.2 of IS 13920:1993, since the joint is not confined by beams framing into its two vertical faces and also since the width of the longitudinal beam is less than ¾ of the column width, special confining reinforcement is required in the joint.

.

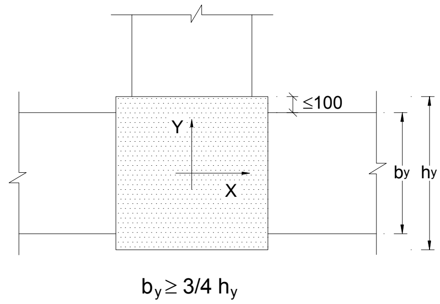


Figure 6.19 Confinement of joint concrete by beams

The spacing of links used as special confining reinforcement shall not exceed:

(i) ¼ of minimum column dimension i.e,

400 / 4 = 100 mm

(ii) But need not be less than 75 mm nor more than 100 mm.

(Clause 7.4.6 of IS 13920:1993)

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

(Clause 7.4.8 of IS 13920:1993)

$$A_{sh} = \frac{0.18\ x\ S \times h \times f_{ck}}{f_y} \left(\frac{A_g}{A_k} - 1 \right)$$

h = longer dimension of the rectangular confining measured to its outer face

= (500 – 40 – 40 – 20) / 2 + 10 x 2 + 20

= 240 mm

A_g = 500 x 400 = 2,00,000 mm^2

A_k = (500-2 x 40 +2 x 10) x (400- 2 x 40 +2 x 10)

= 440 x 340

= 1,49,600 mm^2

Assuming 10 diameter links, A_{sh} = 78.54 mm^2

$$78.54 = \frac{0.18 \times S \times 240 \times 20}{f_y} \left(\frac{2{,}50{,}000}{1{,}49{,}600} - 1 \right)$$

S = 112 mm

Provide 10Φ confining links @ 100 c/c in the joint

Example 7 – Interior Beam-Column Roof Joint Design for Zone-V

7. Problem Statement:

Detailed design as per draft revision of IS 13920:1993 of an interior roof joint in an intermediate RC frame is explained here as per IS 13920 (proposed draft) for the ground plus four storey RC office building of Example-1 (Refer Figures 1.1-1.4)

Solution:

Preliminary Data

The joint of column marked in Figure 7.1 for Frame 2 is considered for design. The plan of the building and the sectional elevation of a typical RC frame are shown in Figures 7.1 and 7.2.

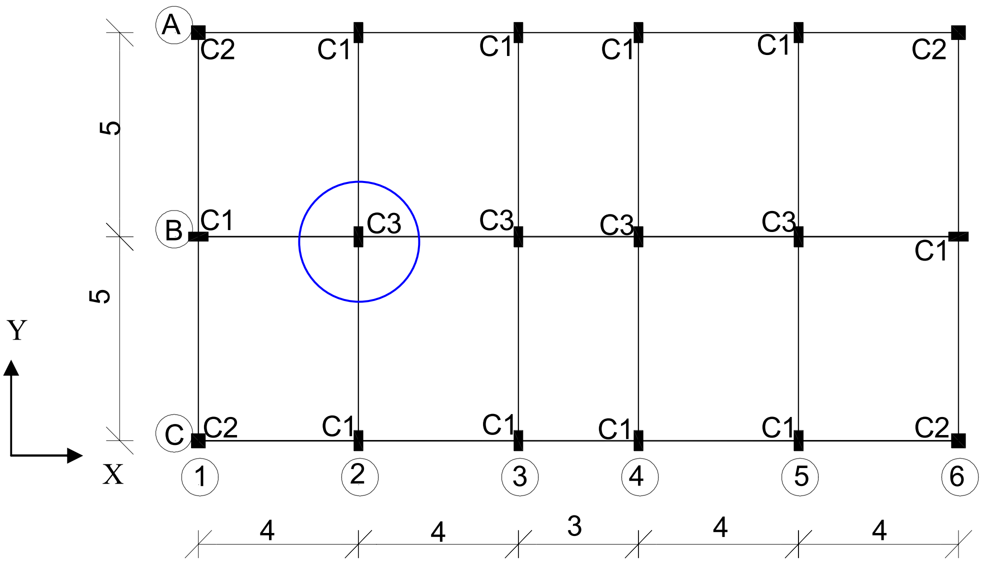


Figure 7. 1 Plan of building (All dimensions in meters)

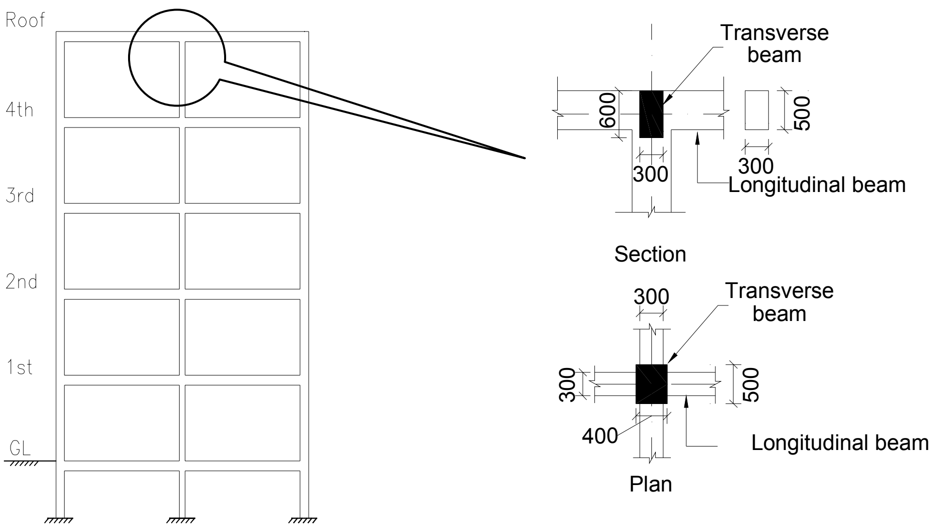


Figure 7.2 Column location in elevation

Design Data

The details of the column and beam reinforcement meeting at the joint are shown in Figure 7.3.

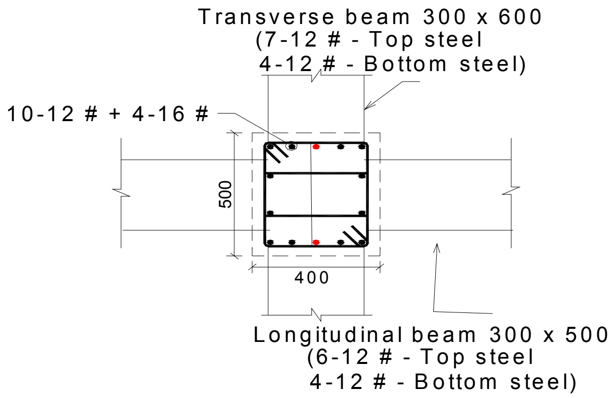


Figure 7.3 Reinforcement details for column and beams.

The transverse beam of size 300 x 600 is reinforced with 7-12Φ (791 mm^2 , i.e., 0.48%) at top and 4-12Φ (452 mm^2 , i.e., 0.27%) at bottom. The hogging and sagging moment capacity is evaluated as 139 kN-m and 83 kN-m, respectively.

The longitudinal beam of size 300 x 500 is reinforced with 6-12Φ (678 mm^2 i.e. 0.510%) at top and 4-12Φ (452 mm^2 i.e. 0.34%) at bottom. The hogging and sagging moment capacity is 105 kN-m and 66 kN-m, respectively.

Check for Earthquake in Y Direction

Joint Shear

Figure 7.4 shows the joint shear equilibrium.

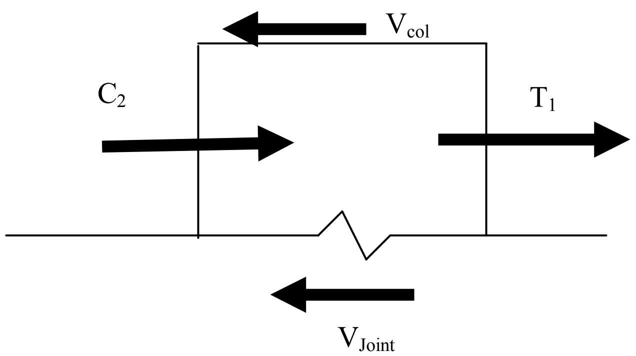


Figure 7.4 Joint shear

Column Shear

The column shear is evaluated as explained below. (Refer Figures 7.5 and 7.6 for sway to right and left condition respectively).

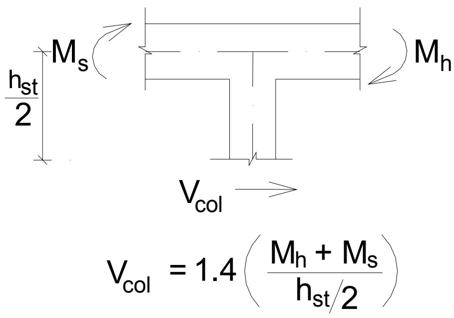


Figure 7.5 Column with sway to right

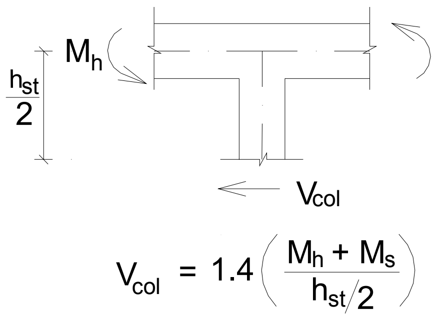


Figure 7.6 Column with sway to left

For both the above cases,

$$V_{col} = 1.4\left(\frac{M_s + M_h}{h_{st}/2}\right)$$

$$= 1.4 \times \left(\frac{139 + 83}{3/2}\right)$$

= 207 kN

Force Developed in Beam Reinforcement

Figures 7.7 and 7.8 show the development of forces in the joint due to beam reinforcement, for sway to right and to left respectively.

Force developed in the top bars

$T_1 = A_{st}$ x 1.25 x f_y

= 791 x 1.25 x 415 /1,000

= 410 kN = C_1

The factor 1.25 is to account for the actual ultimate strength being higher than the actual yield strength (Draft revision IS 13920:1993)

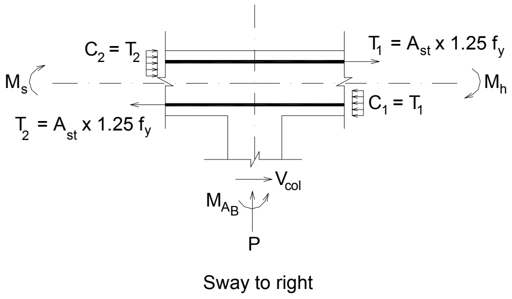


Figure 7.7 Free body diagram of the joint

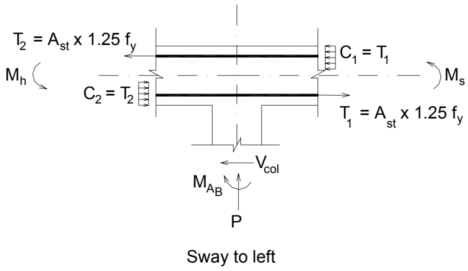


Figure 7.8 Free body diagram of the joint

Force developed in the bottom bars

$T_2 = A_{st}$ x 1.25 x f_y

= 452 x 1.25 x 415 /1,000

= 235 kN = C_2

Referring to Figure 7.4,

$V_{Joint} = T_1 + C_2 - V_{col}$

= 410 + 235-207

= 438 kN

7.3.2 Check for joint shear strength

The effective width provisions for joints are shown in Figure 7.9.

The calculation of the effective width and the design shear strength of the joint is based on the draft revision of IS 13920:1993

The effective width of the joint is the lesser of:

i) $b_j = b_b + 0.5 \times h$

ii) $b_j = b_c$

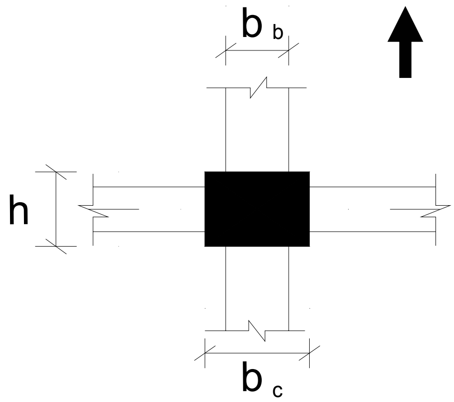


Figure 7.9 Effective widths for joint

$b_j = b_b + \text{h} /2$

= 300 + 500 /2

= 550 mm

h = full depth of column

= 500 mm

$b_j = b_c$

= 400 mm

Take effective width of joint as 400 mm

Effective area of joint resisting shear = $A_c = b_j\ h$

Shear strength of joint confined on two opposite faces, as per Clause 8.1.3 of draft revision of IS 13920:1993

Shear strength = 1.2 $\sqrt{f_{ck}}\ A_c$

= 1.2 x $\sqrt{20}$ x 400 x 500 /1,000

= 1,073 kN > 645 kN

Hence, ok.

7.3.3 Check for flexural strength ratio

The hogging and sagging moment capacity of the transverse beam is evaluated as 139 kN-m and 83 kN-m, respectively.

The column is reinforced with 10 - 12Φ + 4 - 16Φ bars with total A_{sc} = 1,934 mm^2 i.e.

1,934 x 100 / (400 x 500) = 0.967%.

p/f_{ck} = 0.967 / 20 = 0.048

It is conservative here to calculate the moment capacity of the column with zero axial loads. In actual practice it is desirable to take minimum

$\frac{M_u}{f_{ck}bD^2}$ corresponding to actual $\frac{P_u}{f_{ck}bD}$ obtained from different load combinations. Referring to chart 44 of SP: 16, corresponding to $\frac{P_u}{f_{ck}bD} = 0.00$ at A_B, p/f_{ck} = 0.05 and $d'/D = (40 + 25/2) / 500 = 0.105$, we get

$$\frac{M_u}{f_{ck}bD^2} = 0.05.$$

$M_u = 0.05 \times 20 \times 400 \times 500 \times 500 / 1\times10^6$

= 100 kN-m

As per Figure 7.10, the joint is checked for strong column - weak beam condition.

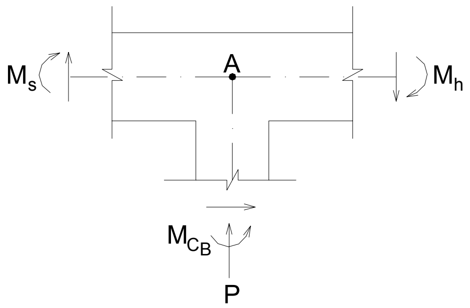


Figure 7.10 Check for strong column - weak beam condition

$\sum M_c = 100$ kN-m

$\sum M_b = 139 + 83 = 222$ kN-m

The ratio of $\frac{\sum M_c}{\sum M_b}$ = 100/222 = 0.45 < 1.1

Hence, requirement of strong column-weak beam condition as per draft revision of IS 13920:1993 is not satisfied.

Check for Earthquake in X Direction

Joint Shear

The joint equilibrium is shown in Figure 7.4.

Shear Due to Plastic Hinge in Beam

Referring to Figure 7.5 and 7.6, for both the cases,

$$V_{col} = 1.4\left(\frac{M_s + M_h}{h_{st}}\right)$$

$$= 1.4 \times \left(\frac{105 + 66}{3/2}\right)$$

= 160 kN

Force Developed in Beam Reinforcement

Referring to Figures 7.7 and 7.8, we get,

$T_1 = A_{st} \times 1.25 \times f_y$

= 678 x1.25 x 415 /1,000

= 352 kN = C_1

$T_2 = A_{st} \times 1.25 \times f_y$

= 452 x1.25 x 415 /1,000

= 235 kN = C_2

The joint shear is evaluated considering maximum T_1 and minimum V_{col}.

$V_{Joint} = T_1 + C_2 - V_{col}$

= 352 + 235 -160

= 427 kN

7.4.3 Check for Joint Shear Strength

The effective width of the joint is evaluated as:

$b_j = b_b + h/2$

= 300 + 400 /2

= 500 mm

h = full depth of column

= 400 mm

$b_j = b_c$

= 500 mm

Take b_j = 500 mm

Shear strength of joint not confined as per Clause 8.1.3 of draft revision of IS 13920:1993

Shear strength = $1.0 \sqrt{f_{ck}}\, A_c$

= 1.0 x $\sqrt{20}$ x 500 x 400 /1,000

= 894 kN > 587 kN

Hence, ok.

(Clause 8.3 of IS 13920 proposed draft)

7.4.4 Check for Flexural Strength Ratio

The hogging and sagging moment capacity of the longitudinal beam is 105 kN-m and 66 kN-m, respectively.

It is conservative here to calculate moment capacity of column with zero axial loads. In actual practice it is desirable to take minimum

$\frac{M_u}{f_{ck}bD^2}$ corresponding to actual $\frac{P_u}{f_{ck}bD}$ obtained from different load combinations. Referring Chart 44 of SP: 16, corresponding to $\frac{P_u}{f_{ck}bD}$ = 0.00 at A_B, p/f_{ck} = 0.05 and d'/D = (40 + 25/2) /400 = 0.13125, we get

$$\frac{M_u}{f_{ck}bD^2} = 0.05$$

M_u = 0.05 x 20 x 400 x 400 x 500 /1x10^6

= 80 kN-m

$\sum M_c$ = 80 kN-m

$\sum M_b$ = 105 + 66 = 171 kN-m

The ratio of $\frac{\sum M_c}{\sum M_b}$ = 80/171 = 0.47 < 1.1

Hence, strong column weak beam condition is not satisfied.

(Clause 7.2.1 of IS 13920 proposed draft)

7.4.5 Re-design of Column

As can be seen from the checks in section 7.4.3 and 7.5.4, the joint is not safe. In such cases it is recommended to either increase the column section or the reinforcement or both so that $\sum M_c$ is increased

It is proposed to increase the reinforcement in the column. Member forces are taken as calculated earlier without reanalysis of the structure. In practice, the structure may be reanalyzed.

The $\sum M_c$ required in the transverse direction is 222 x 1.1 = 244 kN-m and 1.1 x 171 = 188 kN-m in the longitudinal direction.

Hence required moment capacity for the column is M_c = 244 kN-m in the transverse direction and 188 kN-m in the longitudinal direction.

(Clause 7.2.1 of IS 13920 proposed draft)

Using SP: 16, the steel required to get the above moment capacity of the column is calculated as 1.8%. Hence revise the main longitudinal steel to 8-20ϕ+6-16ϕ bars (3,718 mm^2, 1.86% steel). The revised reinforcement details are shown in Figure 7.11.

The redesigned column section is expected to satisfy the flexural strength check.

While redesigning the column, a few load combinations may give axial stresses less than 0.1f_{ck}. The section then needs to be checked for flexure load combinations.

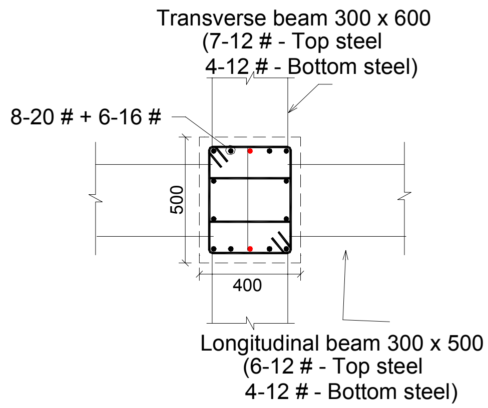


Figure 7.11 Revised reinforcement details for column and beams

Confining Links

In case of an internal joint like the one being designed where beams frame into all vertical faces of the joint, adequate confinement can be assumed, if the beam width is at least 3/4 of the column width and if no more than 100 mm of column offset is exposed on either side of the beams. (Draft revision of IS13920: 1993)

In this case the column dimensions are 400 x 500. The width of the beam is 300 mm, which is less than 3/4 width of column (i.e. 3/4 x 500 = 375 mm). The maximum column offset on either side of the framing beam is (500 - 300) / 2 = 100 mm. Hence, special confining reinforcement as per provisions of Clause 7.4.6 and 7.4.8 of IS 13920: 1993 is required. (Refer Figure 7.12).

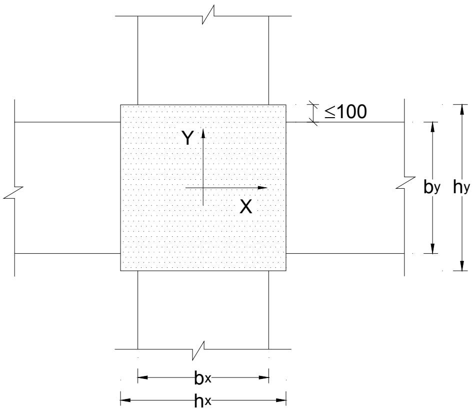
The spacing of hoops used as special confining reinforcement shall not exceed:

(i) ¼ of minimum column dimension i.e,

400 / 4 =100 mm

(ii) But spacing not be less than 75 mm nor more than 100 mm.

(Clause 7.4.6 of IS 13920:1993)



$b_y \geq 3/4\ h_y$
$b_x \geq 3/4\ h_x$

Figure 7.12 Confinement of joint concrete by beams

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

$$A_{sh} = \frac{0.18 \times S \times h \times f_{ck}}{f_y}\left(\frac{A_g}{A_k} - 1\right)$$

(Clause 7.4.8 of IS 13920:1993)

h = longer dimension of the rectangular confining stirrup measured to its outer face

= (500 – 40 – 40 + 12) /2 + 10 x 2 +12

= 236 mm

or

= (400 - 40 – 40 - 12) / 2 + 10 x 2 +12

= 186 mm

A_g = 400 x 500 = 2,00,000 mm^2

A_k = (400 -2 x 40 + 2 x 10) x (500- 2 x 40 + 2 x 10)

= 340 x 440

= 1,46,496 mm^2

A_{sh} = 78.54 mm^2

Substituting we get

$$78.54 = \frac{0.18 \times S \times 236 \times 20}{f_y}\left(\frac{2,00,000}{1,46,496} - 1\right)$$

S = 116 mm

Provide 10Φ confining links @ 100 c/c in the joint.

Example 8 — Exterior Beam-Column Roof Joint Design for Zone V

8. Problem Statement:

Detailed design as per draft revision of IS 13920:1993 of an exterior roof joint in an intermediate RC frame is explained for the ground plus four storey RC office building of Example-1 (Refer Figures 1.1-1.4)

Solution:

Preliminary Data

The joint of column marked in Figure 8.1 for Frame 2 is considered for design. The plan of the building and the sectional elevation of a typical RC frame are shown in Figures 8.1 and 8.2.

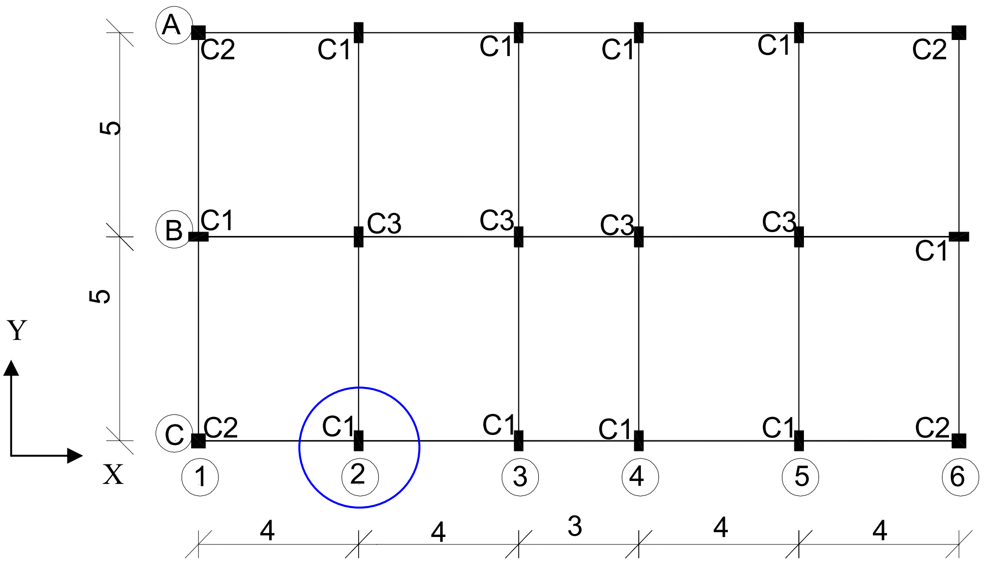


Figure 8. 1 Plan of building (All dimensions in meters)

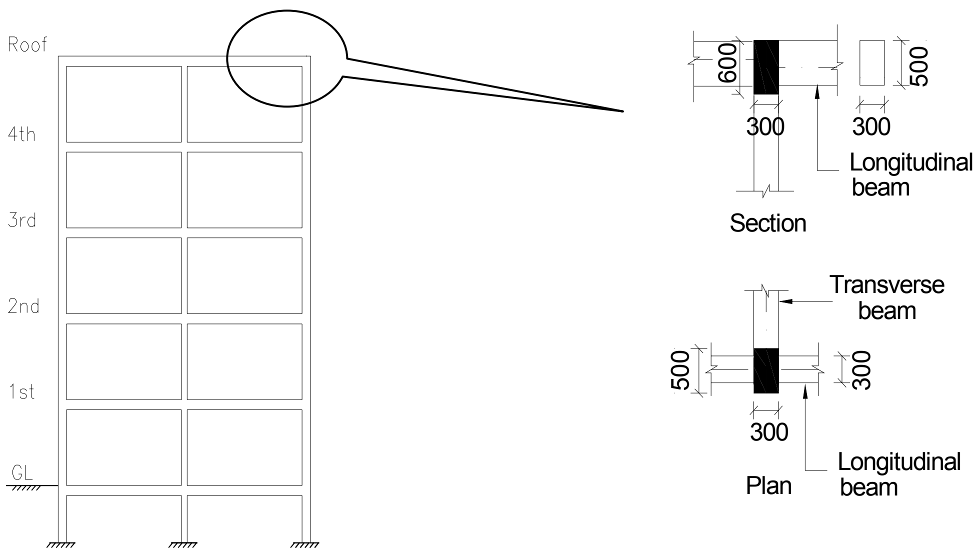


Figure 8.2 Column location in elevation

8.2 Design Data

The details of the column and beam reinforcement meeting at the joint are shown in Figure 8.3.

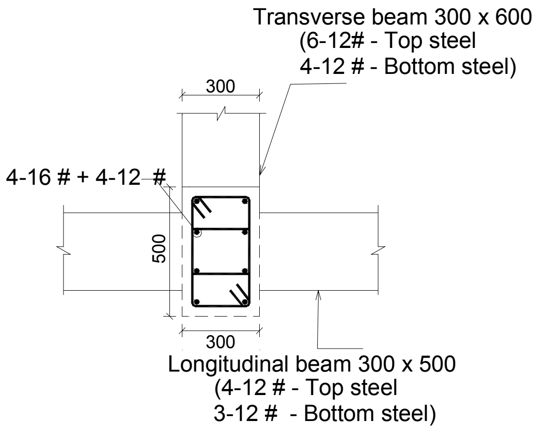


Figure 8.3 Reinforcement details for column and beams.

The transverse beam of size 300x600 is reinforced with 6-12Φ (678 mm^2 i.e. 0.41 %) at top and 4-12Φ (452 mm^2 i.e. 0.27%) at bottom. The hogging and sagging moment capacity is evaluated as 121 kN-m and 83 kN-m, respectively.

The longitudinal beam of size 300 x 500 is reinforced with 4-12Φ (452 mm^2 i.e. 0.34%) at top and 3-12 (339 mm^2 i.e. 0.26%) at bottom. The hogging and sagging moment capacity is evaluated as 67 kN-m and 52 kN-m, respectively.

8.3 Check for Earthquake in Y Direction

8.3.1 Joint Shear

The joint shear equilibrium is shown in Figure 8.4

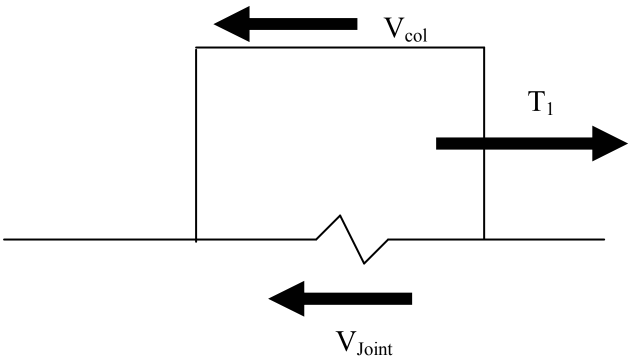


Figure 8.4 Joint shear

Column Shear

The column shear is evaluated as explained below. (Refer Figures 8.5 and 8.6 for sway to left and right conditions respectively).

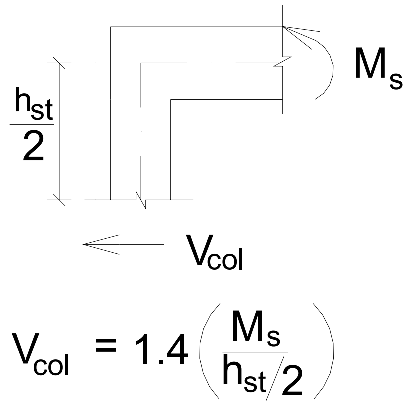


Figure 8.5 Column with sway to left

$$V_{col} = 1.4\left(\frac{M_s}{h_{st}/2}\right)$$

$$= 1.4 \times \left(\frac{83}{3/2}\right)$$

= 77 kN

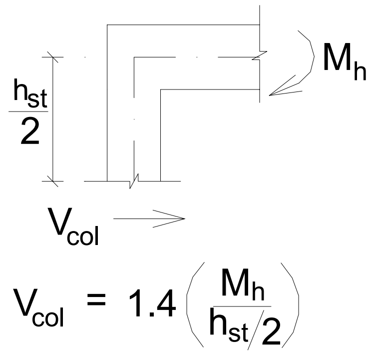


Figure 8.6 Column with sway to right

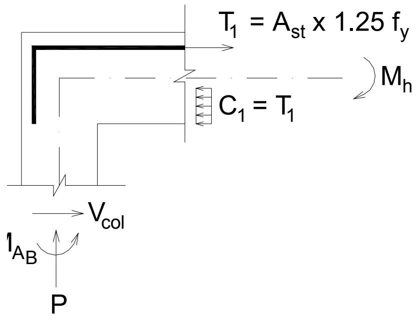
$$V_{col} = 1.4\left(\frac{M_h}{h_{st}}\right)$$

$$= 1.4 \times \left(\frac{121}{3/2}\right)$$

= 113 kN

Force Developed in Beam Reinforcement

Figures 8.7 and 8.8 shows the development of forces in the joint due to beam reinforcement, for sway to right and to left, respectively.



Sway to right

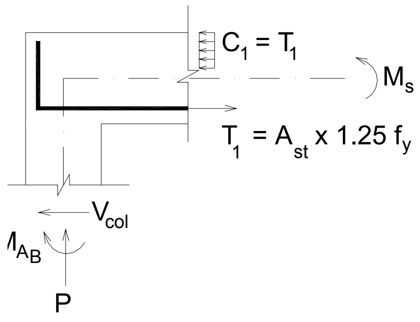
Figure 8.7 Free body diagram of the joint

Force developed in the top bars

$T_1 = A_{st}$ x 1.25 x f_y

= 678 x1.25 x 415 /1,000

= 352 kN



Sway to left

Figure 8.8 Free body diagram of the joint

Max developed force in the bottom bars

$T_1 = A_{st}$ x 1.25 x f_y

= 452 x 1.25 x 415 /1,000

= 235 kN.

Referring to Figure 8.4,

$V_{Joint} = T_1 - V_{col}$

= 352-77

= 275 kN

8.3.2 Check for joint shear strength

The effective width provisions for joints are shown in Figure 8.9

The calculation of the effective width and the design shear strength of the joint is based on the draft revision of IS 13920:1993

The effective width of the joint is the lesser of:

i) $b_j = b_b + 0.5 \text{ x } h$

ii) $b_j = b_c$

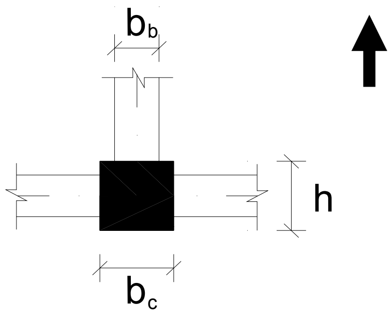


Figure 8.9 Effective widths for joint

$b_j = b_b + h/2$

= 300 + 500 /2

= 550

$b_j = b_c$

= 300 mm

Take effective width of joint as 300 mm

h = full depth of column

= 500 mm

Effective area of joint resisting shear = $A_c = b_j h$

Shear strength of joint not confined as per Clause 8.1.3 of draft revision of IS 13920:1993

Shear strength = 1.0 $\sqrt{f_{ck}}\ A_c$

= 1.0 x $\sqrt{20}$ x 300 x 500 /1,000

= 670 kN > 352 kN

Hence, ok.

(Clause 8.3 of IS 13920 proposed draft)

8.3.3 Check for Flexural Strength Ratio

The hogging and sagging moment capacity of the transverse beam is evaluated as 121 kN-m and 83 kN-m, respectively.

The column is reinforced with 4-16Φ + 4-12Φ bars with total A_{sc} = 1,256 mm^2 i.e. 1,256 x 100 / (300 x 500) = 0.83%.

p/f_{ck} = 0.83 / 20 = 0.042

It is conservative here to calculate the moment capacity of the column with zero axial loads. In actual practice it is desirable to take minimum $\dfrac{M_u}{f_{ck}bD^2}$ corresponding to actual $\dfrac{P_u}{f_{ck}bD}$ obtained from different load combinations. Referring to chart 44 of SP: 16, corresponding to

$\dfrac{P_u}{f_{ck}bD}$ = 0.00 at A_B, p/f_{ck} = 0.042 and

d'/D = (40 +25 /2) / 500 = 0.105, we get

$\dfrac{M_u}{f_{ck}bD^2}$ = 0.06.

M = 0.06 x 20 x 300 x 500 x 500 /1x 10^6

= 90 kN-m

$\sum M_c$ = 90 kN-m $\sum M_b$ = 121 kN-m

(Maximum moment of resistance of the beam is considered)

As per Figure 8.10, the joint is checked for strong column - weak beam condition.

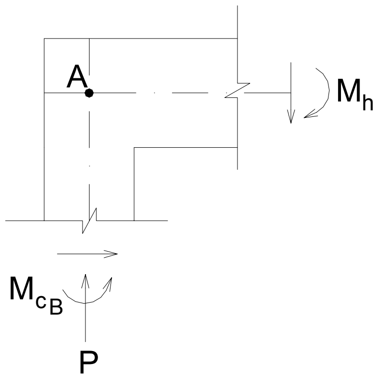


Figure 8.10 Check for strong column - weak beam condition

The ratio of $\dfrac{\sum M_c}{\sum M_b}$ = 90 / 121 = 0.74 < 1.1

Hence, requirement of strong column-weak beam condition as per draft revision of IS 13920:1993 is not satisfied.

Hence, not ok..

8.4 Check for Earthquake in X Direction

4.8.1 Shear Due to Plastic Hinge in Beam

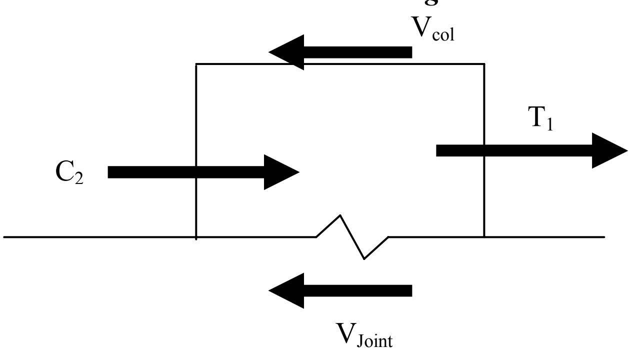


Figure 8.11 Joint shear

Referring to figure 8.11 and 8.12,

$$V_{col} = 1.4\left(\frac{M_s + M_h}{h_{st}/2}\right)$$

$$= 1.4 \times \left(\frac{67+52}{3/2}\right)$$

$$= 111 \text{ kN}$$

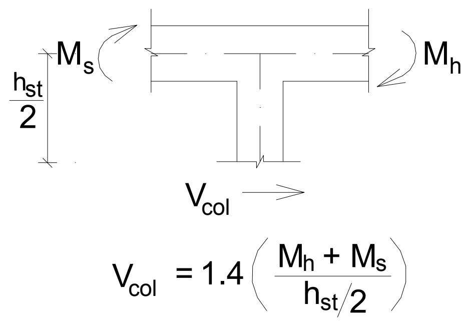


Figure 8.12 Column with sway to right

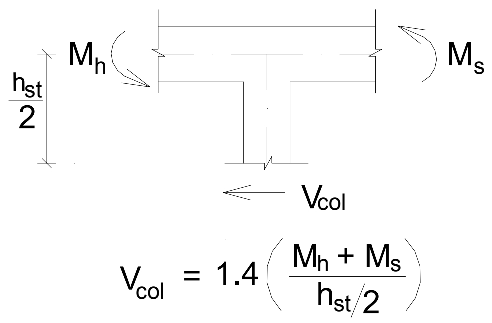


Figure 8.13 Column with sway to left

Joint Shear

Figures 8.14 and 8.15 show the development of forces in the joint due to beam reinforcement, for sway to right and to left respectively.

Force developed in top bars,

$T_1 = A_{st}$ x 1.25 x f_y

= 452 x 1.25 x 415 /1,000

= 235 kN = C_1

Force developed in bottom bars,

$T_2 = A_{st}$ x 1.25 x f_y

= 339 x 1.25 x 415 /1,000

= 176 kN = C_2

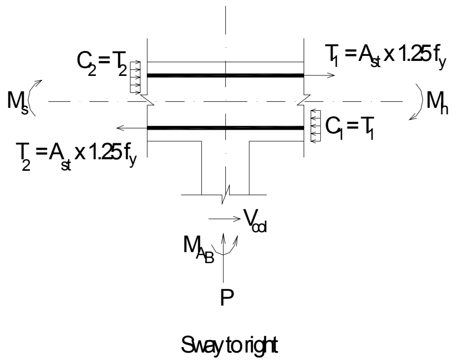


Figure 8.14 Free body diagram of the joint

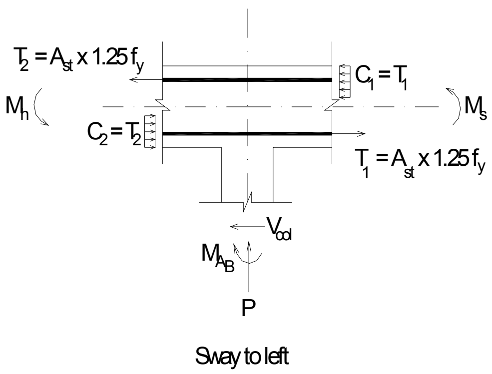


Figure 8.15 Free body diagram of the joint

Referring to the Figure 8.11.

$V_{Joint} = T_1 + C_2 - V_{col}$

= 235 + 176 - 111

= 300 kN

8.4.2 Check for joint shear strength

The effective width calculations for joint are explained in Figure 8.16.

The effective width of the joint is evaluated as:

$b_j = b_b + h/2$

= 300 + 300 /2

= 450 mm

$b_j = b_c$

= 500 mm

Take b_j = 450 mm

h = full depth of column or full depth of beam

= 300 mm.

Shear strength of joint not confined ($b_c > ¾\, b_b$ on two opposite faces) as per Clause 8.1.3 of draft revision of IS 13920:1993

Shear strength = 1.0 $\sqrt{f_{ck}}\, A_c$

= 1.0 x $\sqrt{20}$ x 450 x 300 /1,000

= 603 kN > 300 kN

Hence, ok.

(Clause 8.3 of IS: 13920 proposed draft)

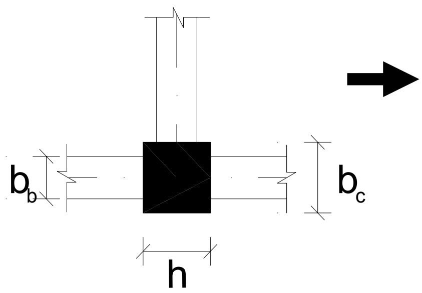


Figure 8.16 Effective width of joint

8.4.3 Check for Flexural Strength Ratio

The hogging and sagging moment capacity of the longitudinal beam is evaluated as 67 kN-m and 52 kN-m, respectively.

The column is reinforced with 4-16Φ + 4-12Φ bars with total A_{sc} = 1,256 mm^2 i.e. 1,256 x100 / (300 x 500) = 0.83%.

p/f_{ck} = 0.83 / 20 = 0.042

It is conservative here to calculate moment capacity of column with zero axial loads. In actual practice it is desirable to take minimum $\frac{M_u}{f_{ck}bD^2}$ corresponding to actual $\frac{P_u}{f_{ck}bD}$ obtained from different load combinations. Referring to charts 45/46 of SP: 16, corresponding to

$\frac{P_u}{f_{ck}bD}$ = 0.00 at A_B, p/f_{ck} = 0.042 and

d'/D = (40 +25 /2) / 300 = 0.175, we get

$\frac{M_u}{f_{ck}bD^2}$ = 0.055.

M = 0.055 x 20 x 300 x 300 x 500 /1x 10^6

= 50 kN-m

$\sum M_c$ = 50 kN-m

$\sum M_b = 67 + 52 = 119$ kN-m

(Maximum moment resistance is considered)

As per Figure 8.17, the joint is checked for strong column - weak beam condition.

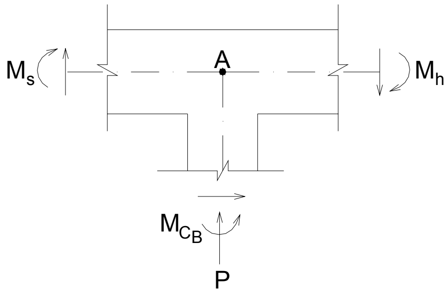


Figure 8.17 Check for strong column - weak beam condition

The ratio of $\dfrac{\sum M_c}{\sum M_b}$ = 50 / 119 = 0.42 < 1.1

Hence, requirement of strong column-weak beam condition as per draft revision of IS 13920:1993 is not satisfied.

Hence, not ok.

(Clause 7.2.1 of IS: 13920 proposed draft)

8.5 Re-design

As can be seen from the checks in sections 8.4.3 and 8.5.3, the joint is not safe. In such cases it is recommended to either increase the column section or the reinforcement or both so that $\sum M_c$ is increased.

It is proposed to increase the reinforcement in the column. Member forces are taken as earlier without reanalysis of the structure. In practice the structure may be reanalyzed.

The $\sum M_c$ required in the transverse direction is 121 x 1.1 = 133 kN-m and 1.1 x 119 = 131 kN-m in longitudinal direction.

Hence, the required moment capacity for the column is M_c = 133 kN-m in the transverse direction and 131kN-m in the longitudinal direction.

Using SP: 16, the steel required to get the above moment capacity of the column is calculated as 2.4% steel. Hence, revise the main longitudinal steel to 8-20ϕ + 6-16ϕ bars (3,718 mm^2, 2.47% steel). The revised reinforcement details are shown in Figure 8.18.

The redesigned column section satisfies the flexural strength check.

While redesigning the column, a few load combinations may give axial stresses less than 0.1 f_{ck}. The section needs to be checked for flexure for these load combinations.

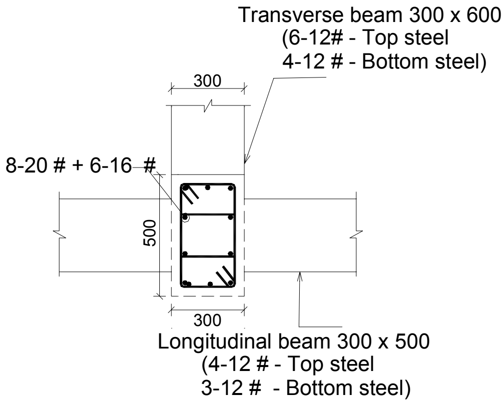


Figure 8.18 Revised reinforcement details for column and beams.

8.6 Confining Links

In this case, the column dimensions have been revised to 300 x 500. The width of the beam is 300 mm which is more than 3/4 width of the column (i.e., 3/4 x 300 = 225 mm) in the transverse direction but less than ¾ of column width (i.e., 3/4 x 500 = 375 mm) in the longitudinal direction. The column offset on either side of beam is equal to (500-300) /2 = 100 mm. Also, since the column is confined by beams framing into its vertical faces from three sides only, special confining reinforcement as per Clauses 7.4.6 and 7.4.8 of IS 13920: 1993 is required.

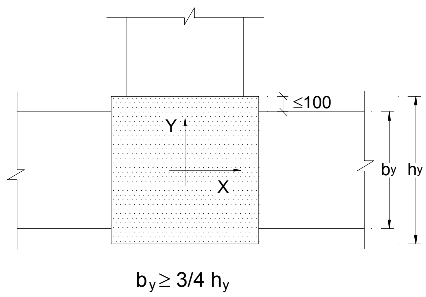


Figure 8.19 Confinement of joint concrete by beams

The spacing of hoops used as special confining reinforcement shall not exceed:

(i) ¼ of minimum column dimension i.e.,

300 / 4 = 75 mm

(ii) But spacing not be less than 75mm nor more than 100 mm.

(Clause 7.4.6 of IS 13920:1993)

The area of cross section A_{sh} of the bar forming the rectangular hoop to be used as special confining reinforcement shall not be less than

(Clause 7.4.8 of IS 13920:1993)

$$A_{sh} = \frac{0.18 \times S \times h \times f_{ck}}{f_y}\left(\frac{A_g}{A_k} - 1\right)$$

h = longer dimension of the rectangular confining measured to its outer face

= (300 - 40- 40 +10 +10) = 240 mm

A_g = 300 x 500 = 1, 50,000 mm^2

A_k = (300-2 x 40 +2 x 10) x (500- 2 x 40 + 2 x 10)

= 240 x 440

= 92,400 mm^2

$$78.54 = \frac{0.18 \times S \times 240 \times 20}{f_y}\left(\frac{1{,}50{,}000}{1{,}05{,}600} - 1\right)$$

Solving we get, S = 90 mm.

Adopt 10 mm diameter bar for special confining reinforcement at a spacing of 90 c/c.

Example 9 - Shear Wall Design for a Building in Zone III

9. Problem Statement:

Design a shear wall for a two-storey building shown in (Figure 9.1).The materials are M20 concrete and Fe415 steel. The example shows design for load combination 1.2(DL + LL +EL) only. In practice all other combinations should also be considered. The unfactored forces in the panel between the ground level and first floor are obtained by analysis as.

S.No	Load Case	Bending Moment	Axial Force	Shear Force
1.	(DL+LL)	-577.5	1922.9	19.7
2.	Earthquake	4830.9	255.7	699.1

The problem and the solution have been adopted from Medhekar M S and Jain S K, "Seismic Behavior and Detailing of R C Shear Walls, part II: Design and Detailing, "The Indian Concrete Journal", Vol. 67, No.8, September 1993, 451-457".

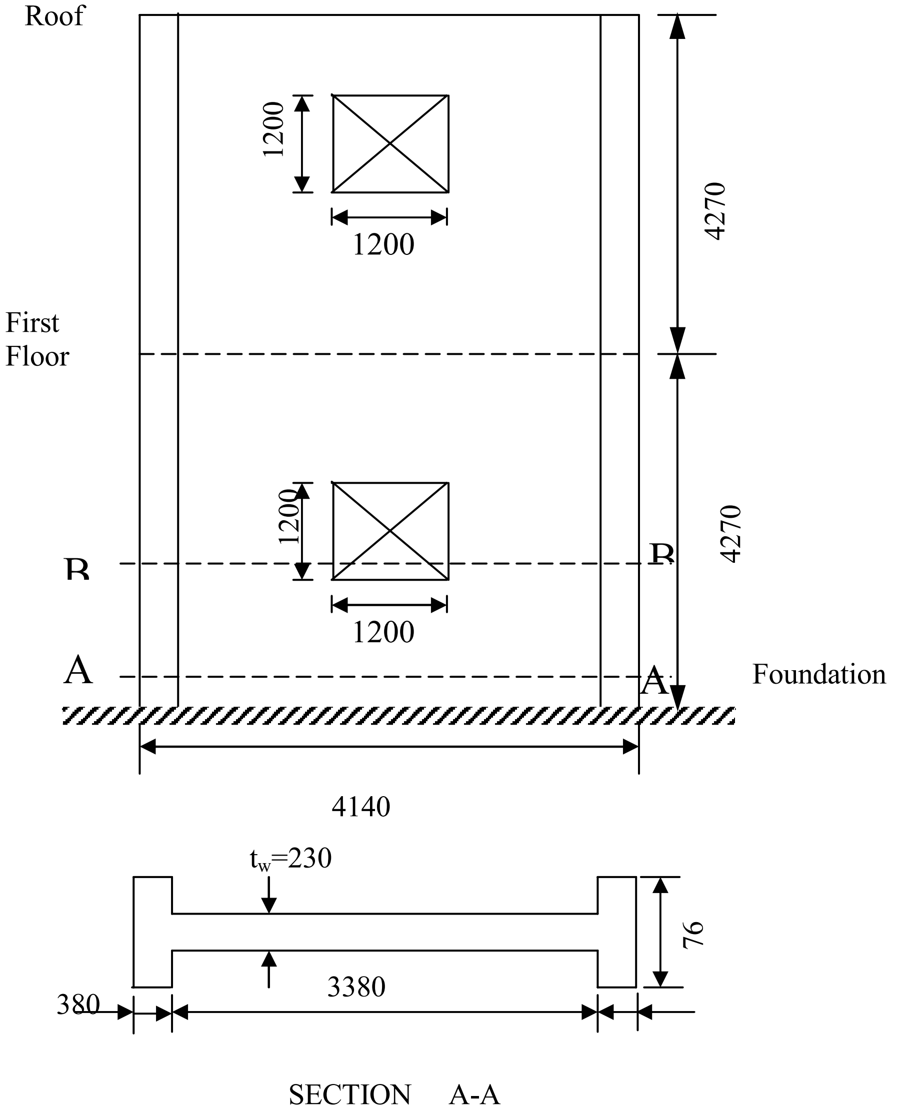


Figure 9.1– Shear wall details for example

9.1 Solution:

The maximum factored bending moment on the section, Mu = 1.2 × (577.5 + 4830.9) = 6490 kNm. The maximum factored shear force, Vu =1.2 × (19.7 + 699.1) = 863 kN.

9.1.1 *Shear Design:*

At section A-A, the design shear force is given by, Vu = 863 kN. Let the effective depth in resisting shear be 3760 mm (=3380+380). Therefore, τ_v = 0.998 N/mm2. Let minimum vertical reinforcement (0.25%) be provided in the web. Therefore, as per Table 13 of IS: 456-1978, τ_v = 0.36 N/mm2. Shear carried by concrete, Vuc = 311 kN. Hence, shear to be resisted by horizontal reinforcement is Vus = 552 kN. This requires the ratio Ah/Sv to be 0.407. However, provision of minimum horizontal reinforcement (0.25%) requires this ratio to be 0.575.

As tw > 200 mm, the reinforcement shall be in 2 layers. Thus, horizontal reinforcement of 8mm diameter bars at 175 mm c/c in 2 layers will suffice. An opening is present at section B-B. Taking depth of wall on each side of opening that is resisting shear as 1280 mm, τ_v =1.466 N/mm2. Thus, shear to be resisted by reinforcement on each side of opening is, Vus= 326 kN. Therefore, provide 8 mm diameter 2-legged stirrups at 140 mm c/c on each side of opening.

9.1.2 *Flexural strength of web:*

The vertical reinforcement in the web is 0.25%. The length of wall, lw, is 4140 mm and its web thickness, tw, is 230 mm. Axial compression will increase the moment capacity of the wall. Therefore, the factored axial force should be taken as Pu = 0.8 × 1922.9 +1.2 × 255.7 = 1845 kN. Assuming this axial load to be uniformly distributed, load on web = 0.574 × 1845 = 1059 kN. Thus, from equations (2), (3), and (5), we get λ = 0.056, φ = 0.045, xu/lw = 0.233, xu*/lw = 0.660, and the value of β is 0.516. As xu/lw is less than x*u/lw, the moment of resistance of the web is obtained from equation (4) as, Muv = 3296 kNm. The remaining moment, i.e., (Mu-Muv) = 3194 kN shall be resisted by reinforcement in the boundary elements.

9.1.3 *Boundary elements:*

The axial compression at the extreme fiber due to combined axial load and bending on the section is 6.805 N/mm2. As this is greater than 0.2fck, provision of boundary elements along the wall edges is mandatory. The center to center distance between the boundary elements, Cw, is 3.76 m. The axial force on the boundary element due to earthquake loading is (Mu-Muv)/Cw = 3194/3.76 = 849 kN. Thus, the maximum factored compression on the boundary element is [849 + 0.213 × 1.2 × (1922.9 + 255.7)] = 1406 kN. The factored tension on the boundary element is [0.213 × (0.8 × 1922.9 - 1.2 × 255.7) -849] = -587 kN. Assuming short column action, the axial load capacity of the boundary element with minimum reinforcement of 0.8% is 2953 kN. Therefore, 12 bars of 16 mm diameter will be adequate to take the compression as well as tension. The arrangement of reinforcement in the boundary element as per Figure 9.2 requires 10 mm diameter rectangular hoops to be provided at 95 mm c/c as special confining reinforcement.

9.1.4 *Reinforcement around opening:*

The opening is of size 1200 mm by 1200 mm. The area of vertical and horizontal reinforcement in the web (0.25%) that is interrupted by it is 690 mm2. Therefore, one bar of 16 mm diameter should be provided per layer of reinforcement on each side of the opening. The vertical bar should extend for the full storey height. The horizontal bar should be provided with development length in tension beyond the sides of the opening.

Figure 9.2 illustrates the reinforcement details.

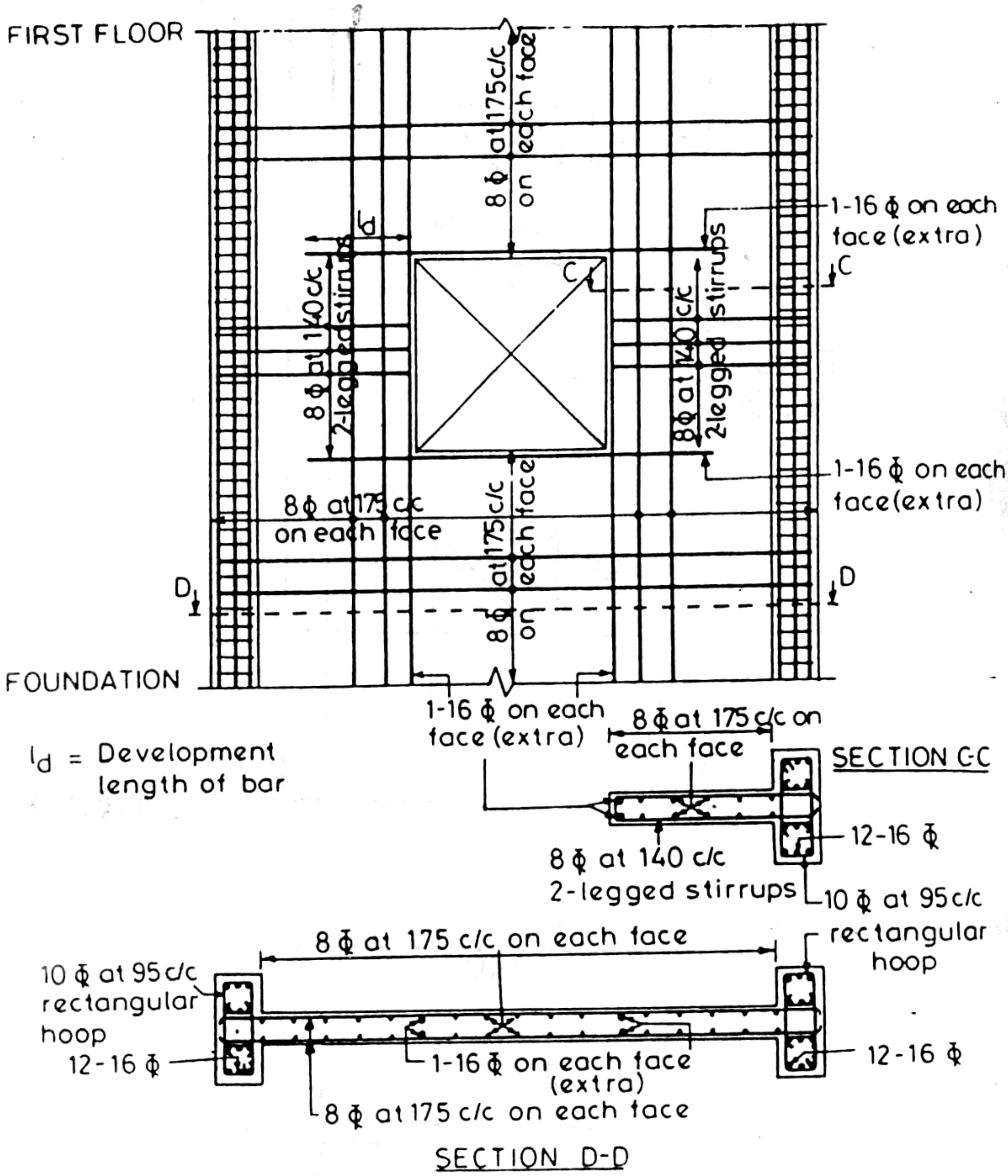


Figure 9.2 Reinforcement details for example