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Design Example of a Six Storey Building

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Example — Seismic Analysis and Design of a Six Storey Building

Problem Statement:

A six storey building for a commercial complex has plan dimensions as shown in Figure 1. The building is located in seismic zone III on a site with medium soil. Design the building for seismic loads as per IS 1893 (Part 1): 2002.

General

1. The example building consists of the main block and a service block connected by expansion joint and is therefore structurally separated (Figure 1). Analysis and design for main block is to be performed.

2 The building will be used for exhibitions, as an art gallery or show room, etc., so that there are no walls inside the building. Only external walls 230 mm thick with 12 mm plaster on both sides are considered. For simplicity in analysis, no balconies are used in the building.

3. At ground floor, slabs are not provided and the floor will directly rest on ground. Therefore, only ground beams passing through columns are provided as tie beams. The floor beams are thus absent in the ground floor.

4. Secondary floor beams are so arranged that they act as simply supported beams and that maximum number of main beams get flanged beam effect.

5. The main beams rest centrally on columns to avoid local eccentricity.

6. For all structural elements, M25 grade concrete will be used. However, higher M30 grade concrete is used for central columns up to plinth, in ground floor and in the first floor.

7. Sizes of all columns in upper floors are kept the same; however, for columns up to plinth, sizes are increased.

8. The floor diaphragms are assumed to be rigid.

9. Centre-line dimensions are followed for analysis and design. In practice, it is advisable to consider finite size joint width.

10. Preliminary sizes of structural components are assumed by experience.

11. For analysis purpose, the beams are assumed to be rectangular so as to distribute slightly larger moment in columns. In practice a beam that fulfils requirement of flanged section in design, behaves in between a rectangular and a flanged section for moment distribution.

12. In Figure 1(b), tie is shown connecting the footings. This is optional in zones II and III; however, it is mandatory in zones IV and V.

13. Seismic loads will be considered acting in the horizontal direction (along either of the two principal directions) and not along the vertical direction, since it is not considered to be significant.

14. All dimensions are in mm, unless specified otherwise.

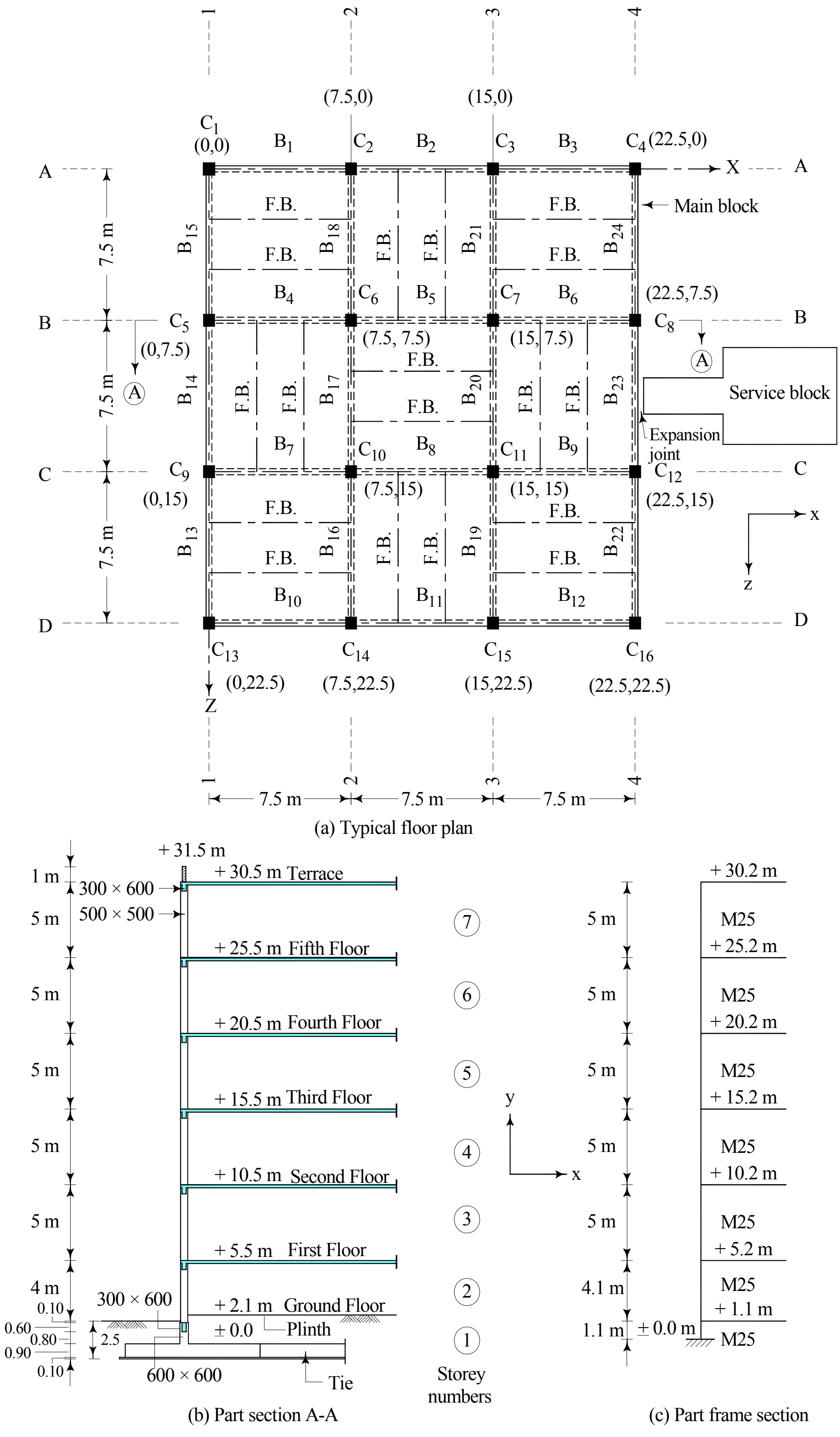


Figure 1 General lay-out of the Building.

1.1. Data of the Example

The design data shall be as follows:

Live load	: 4.0 kN/m^2 at typical floor
	: 1.5 kN/m^2 on terrace
Floor finish	: 1.0 kN/m^2
Water proofing	: 2.0 kN/m^2
Terrace finish	: 1.0 kN/m^2
Location	: Vadodara city
Wind load	: As per IS: 875-Not designed for wind load, since earthquake loads exceed the wind loads.
Earthquake load	: As per IS-1893 (Part 1) - 2002
Depth of foundation below ground	: 2.5 m
Type of soil	: Type II, Medium as per IS:1893
Allowable bearing pressure	: 200 kN/m^2
Average thickness of footing	: 0.9 m, assume isolated footings
Storey height	: Typical floor: 5 m, GF: 3.4 m
Floors	: G.F. + 5 upper floors.
Ground beams	: To be provided at 100 mm below G.L.
Plinth level	: 0.6 m
Walls	: 230 mm thick brick masonry walls only at periphery.

Material Properties

Concrete

All components unless specified in design: M25 grade all

$E_c = 5\,000\sqrt{f_{ck}}$ N/mm^2 = $5\,000\sqrt{f_{ck}}$ MN/m^2

= 25 000 N/mm^2 = 25 000 MN/m^2.

For central columns up to plinth, ground floor and first floor: M30 grade

$E_c = 5\,000\sqrt{f_{ck}}$ N/mm^2 = $5\,000\sqrt{f_{ck}}$ MN/m^2

= 27 386 N/mm^2 = 27 386 MN/m^2.

Steel

HYSD reinforcement of grade Fe 415 confirming to IS: 1786 is used throughout.

1.2. Geometry of the Building

The general layout of the building is shown in Figure 1. At ground level, the floor beams FB are not provided, since the floor directly rests on ground (earth filling and 1:4:8 c.c. at plinth level) and no slab is provided. The ground beams are provided at 100 mm below ground level. The numbering of the members is explained as below.

1.2.1. Storey number

Storey numbers are given to the portion of the building between two successive grids of beams. For the example building, the storey numbers are defined as follows:

Portion of the building	Storey no.
Foundation top – Ground floor	1
Ground beams – First floor	2
First Floor – Second floor	3
Second floor – Third floor	4
Third floor – Fourth floor	5
Fourth floor – Fifth floor	6
Fifth floor - Terrace	7

1.2.2. Column number

In the general plan of Figure 1, the columns from C_1 to C_{16} are numbered in a convenient way from left to right and from upper to the lower part of the plan. Column C_5 is known as column C_5 from top of the footing to the terrace level. However, to differentiate the column lengths in different stories, the column lengths are known as 105, 205, 305, 405, 505, 605 and 705 [Refer to Figure 2(b)]. The first digit indicates the storey number while the last two digits indicate column number. Thus, column length 605 means column length in sixth storey for column numbered C_5. The columns may also be specified by using grid lines.

1.2.3. Floor beams (Secondary beams)

All floor beams that are capable of free rotation at supports are designated as FB in Figure 1. The reactions of the floor beams are calculated manually, which act as point loads on the main beams. Thus, the floor beams are not considered as the part of the space frame modelling.

1.2.4. Main beams number

Beams, which are passing through columns, are termed as main beams and these together with the columns form the space frame. The general layout of Figure 1 numbers the main beams as beam B_1 to B_{12} in a convenient way from left to right and from upper to the lower part of the plan. Giving 90^o clockwise rotation to the plan similarly marks the beams in the perpendicular direction. To floor-wise differentiate beams similar in plan (say beam B_5 connecting columns C_6 and C_7) in various floors, beams are numbered as 1005, 2005, 3005, and so on. The first digit indicates the storey top of the beam grid and the last three digits indicate the beam number as shown in general layout of Figure 1. Thus, beam 4007 is the beam located at the top of 4^{th} storey whose number is B_7 as per the general layout.

1.3. Gravity Load calculations

1.3.1. Unit load calculations

Assumed sizes of beam and column sections are:

Columns: 500 x 500 at all typical floors

Area, $A = 0.25$ m^2, $I = 0.005208$ m^4

Columns: 600 x 600 below ground level

Area, $A = 0.36$ m^2, $I = 0.0108$ m^4

Main beams: 300 x 600 at all floors

Area, $A = 0.18$ m^2, $I = 0.0054$ m^4

Ground beams: 300 x 600
Area, $A = 0.18$ m^2, $I = 0.0054$ m^4

Secondary beams: 200 x 600

Member self- weights:

Columns (500 x 500)

0.50 x 0.50 x 25 = 6.3 kN/m

Columns (600 x 600)

0.60 x 0.60 x 25 = 9.0 kN/m

Ground beam (300 x 600)

0.30 x 0.60 x 25 = 4.5 kN/m

Secondary beams rib (200 x 500)

0.20 x 0.50 x 25 = 2.5 kN/m

Main beams (300 x 600)

0.30 x 0.60 x 25 = 4.5 kN/m

Slab (100 mm thick)

0.1 x 25 = 2.5 kN/m^2

Brick wall (230 mm thick)

0.23 x 19 (wall) +2 x 0.012 x 20 (plaster)

= 4.9 kN/m^2

Floor wall (height 4.4 m)
4.4 x 4.9 = 21.6 kN/m

Ground floor wall (height 3.5 m)
3.5 x 4.9 = 17.2 kN/m

Ground floor wall (height 0.7 m)
0.7 x 4.9 = 3.5 kN/m

Terrace parapet (height 1.0 m)
1.0 x 4.9 = 4.9 kN/m

1.3.2. Slab load calculations

Component	Terrace (DL + LL)	Typical (DL + LL)
Self (100 mm thick)	2.5 + 0.0	2.5 + 0.0
Water proofing	2.0 + 0.0	0.0 + 0.0
Floor finish	1.0 + 0.0	1.0 + 0.0
Live load	0.0 + 1.5	0.0 + 4.0
Total	5.5 + 1.5 kN/m^2	3.5 + 4.0 kN/m^2

1.3.3. Beam and frame load calculations:

(1) Terrace level:

Floor beams**:**

From slab

2.5 x (5.5 + 1.5) = 13.8 + 3.8 kN/m

Self weight = 2.5 + 0 kN/m

Total = 16.3 + 3.8 kN/m

Reaction on main beam

0.5 x 7.5 x (16.3 + 3.8) = 61.1 + 14.3 kN.

Note: Self-weights of main beams and columns will not be considered, as the analysis software will directly add them. However, in calculation of design earthquake loads (section 1.5), these will be considered in the seismic weight.

<u>Main beams B1–B2–B3 and B10–B11–B12</u>

Component	B1-B3	B2
From Slab		
0.5 x 2.5 (5.5 +1.5)	6.9 + 1.9	0 + 0
Parapet	4.9 + 0	4.9 + 0
Total	11.8 + 1.9 kN/m	4.9 + 0 kN/m

Two point loads on one-third span points for beams B_2 and B_{11} of (61.1 + 14.3) kN from the secondary beams.

<u>Main beams B4–B5–B6, B7–B8–B9, B16–B17– B18 and B19–B20–B21</u>

From slab
0.5 x 2.5 x (5.5 + 1.5) = 6.9 + 1.9 kN/m
Total = 6.9 + 1.9 kN/m

Two point loads on one-third span points for all the main beams (61.1 + 14.3) kN from the secondary beams.

<u>Main beams B13–B14–B15 and B22–B23–B24</u>

Component	B13 – B15 B22 – B24	B14 B23
From Slab 0.5 x 2.5 (5.5 +1.5)	----	6.9 + 1.9
Parapet	4.9 + 0	4.9 + 0
Total	4.9 + 0 kN/m	11.8 + 1.9 kN/m

Two point loads on one-third span points for beams B13, B15, B22 and B24 of (61.1+14.3) kN from the secondary beams.

(2) Floor Level:

Floor Beams:

From slab
2.5 x (3.5 + 4.0) = 8.75 + 10 kN/m
Self weight = 2.5 + 0 kN/m
Total = 11.25 + 10 kN/m
Reaction on main beam
0.5 x 7.5 x (11.25 + 10.0) = 42.2 + 37.5 kN.

Main beams B1–B2–B3 and B10–B11–B12

Component	B1 – B3	B2
From Slab		
0.5 x 2.5 (3.5 + 4.0)	4.4 + 5.0	0 + 0
Wall	21.6 + 0	21.6 + 0
Total	26.0 + 5.0 kN/m	21.6 + 0 kN/m

Two point loads on one-third span points for beams B2 and B11 (42.2 + 37.5) kN from the secondary beams.

Main beams B4–B5–B6, B7–B8–B9, B16–B17–B18 and B19–B20–B21

From slab 0.5 x 2.5 (3.5 + 4.0) = 4.4 + 5.0 kN/m

Total = 4.4 + 5.0 kN/m

Two point loads on one-third span points for all the main beams (42.2 + 37.5) kN from the secondary beams.

Main beams B13–B14–B15 and B22–B23–B24

Component	B13 – B15 B22 – B24	B14 B23
From Slab		
0.5 x 2.5 (3.5 + 4.0)	----	4.4 + 5.0
Wall	21.6 + 0	21.6 + 0
Total	21.6 + 0 kN/m	26.0 + 5.0 kN/m

Two point loads on one-third span points for beams B13, B15, B22 and B24 of

(42.2 +7.5) kN from the secondary beams.

(3) Ground level:

Outer beams: B1-B2-B3; B10-B11-B12; B13-B14-B15 and B22-B23-B24

Walls: 3.5 m high

17.2 + 0 kN/m

Inner beams: B4-B5-B6; B7-B8-B9; B16-B17-B18 and B19-B20-B21

Walls: 0.7 m high

3.5 + 0 kN/m

Loading frames

The loading frames using the above-calculated beam loads are shown in the figures 2 (a), (b), (c) and (d). There are total eight frames in the building. However, because of symmetry, frames A-A, B-B, 1-1 and 2-2 only are shown.

It may also be noted that since LL< (3/4) DL in all beams, the loading pattern as specified by Clause 22.4.1 (a) of IS 456:2000 is not necessary. Therefore design dead load plus design live load is considered on all spans as per recommendations of Clause 22.4.1 (b). In design of columns, it will be noted that DL + LL combination seldom governs in earthquake resistant design except where live load is very high. IS: 875 allows reduction in live load for design of columns and footings. This reduction has not been considered in this example.

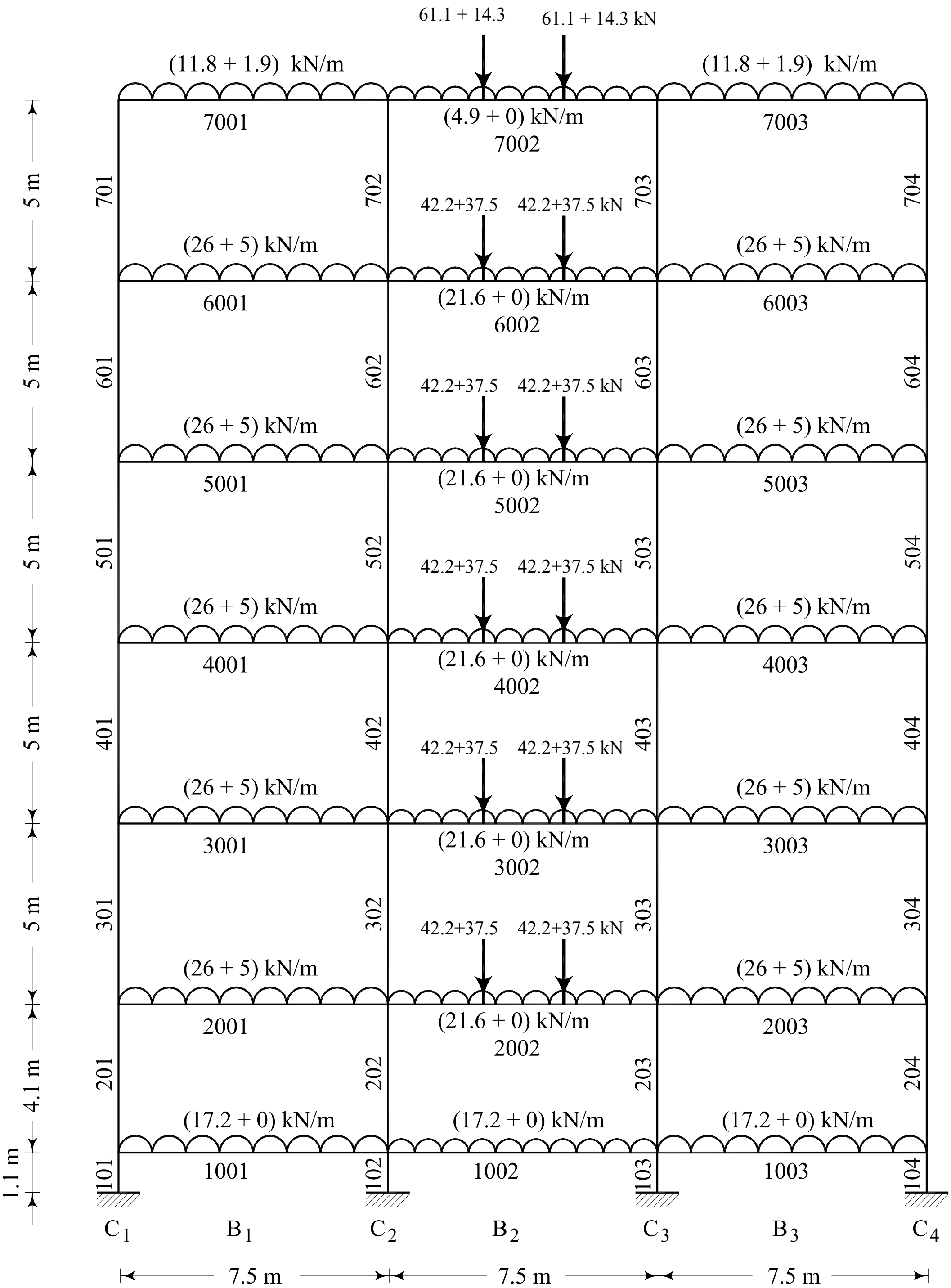


Figure 2 (a) Gravity Loads: Frame AA

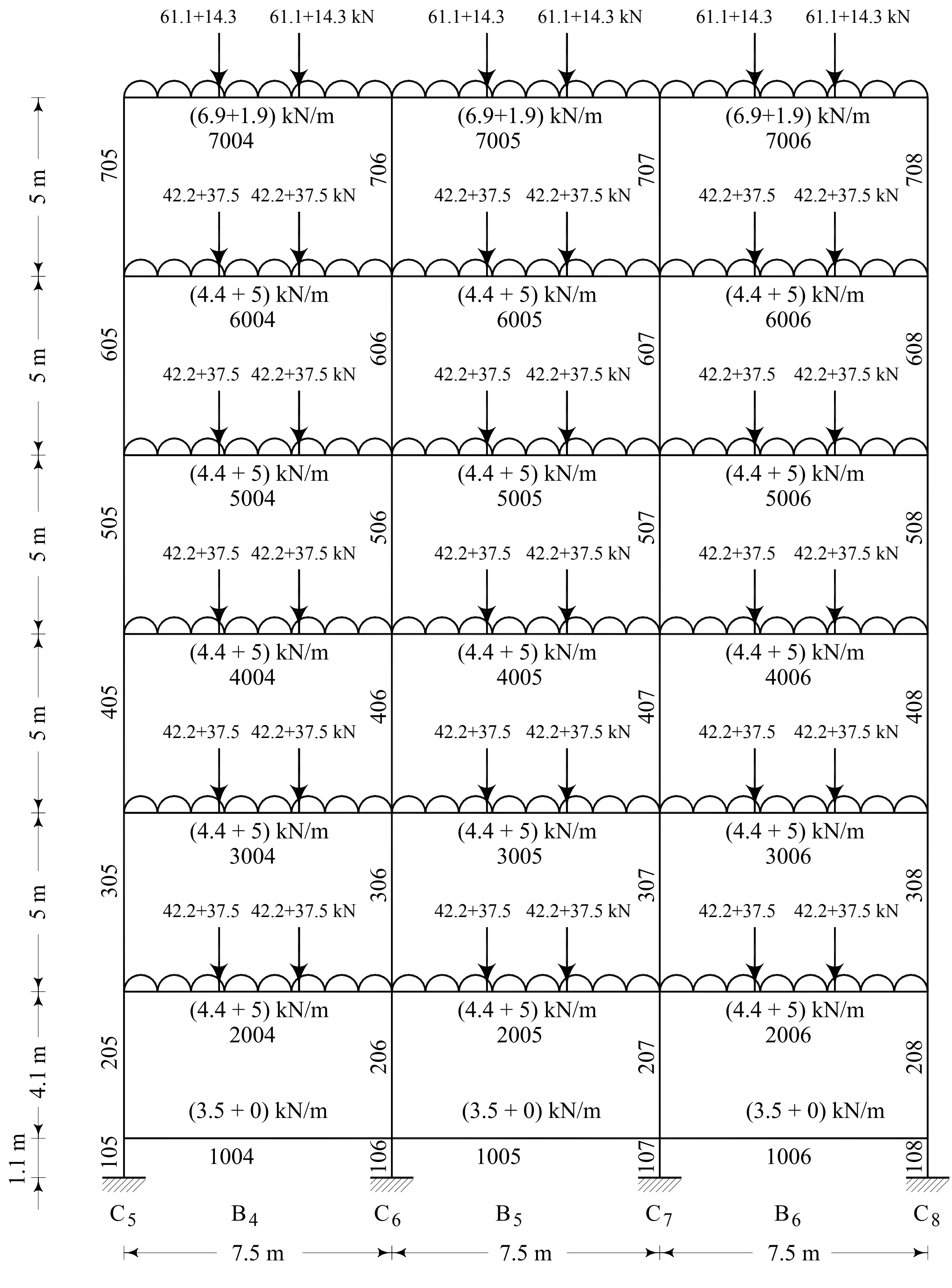


Figure 2(b) Gravity Loads: Frame BB

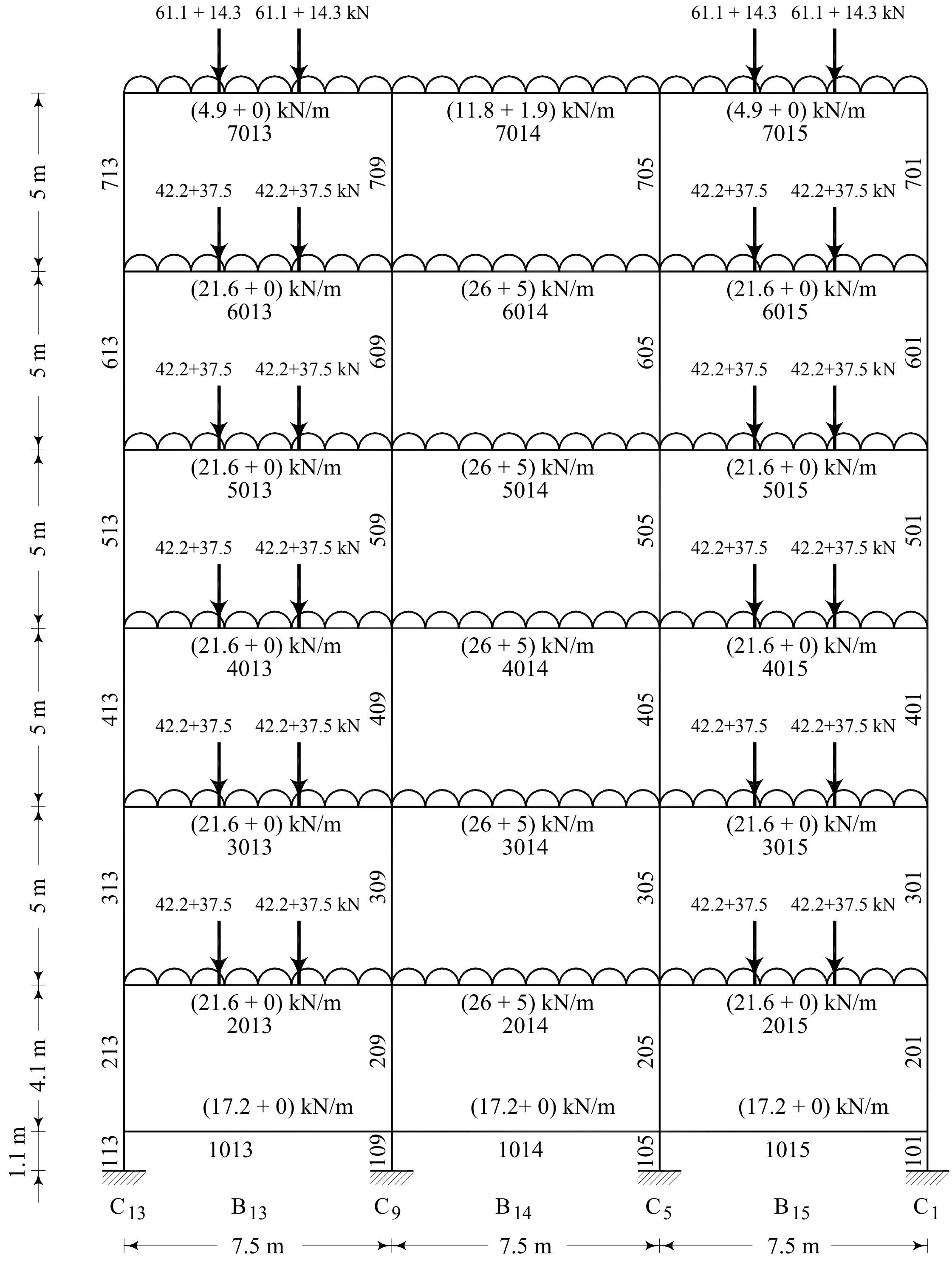


Figure 2(c) Gravity Loads: Frame 1-1

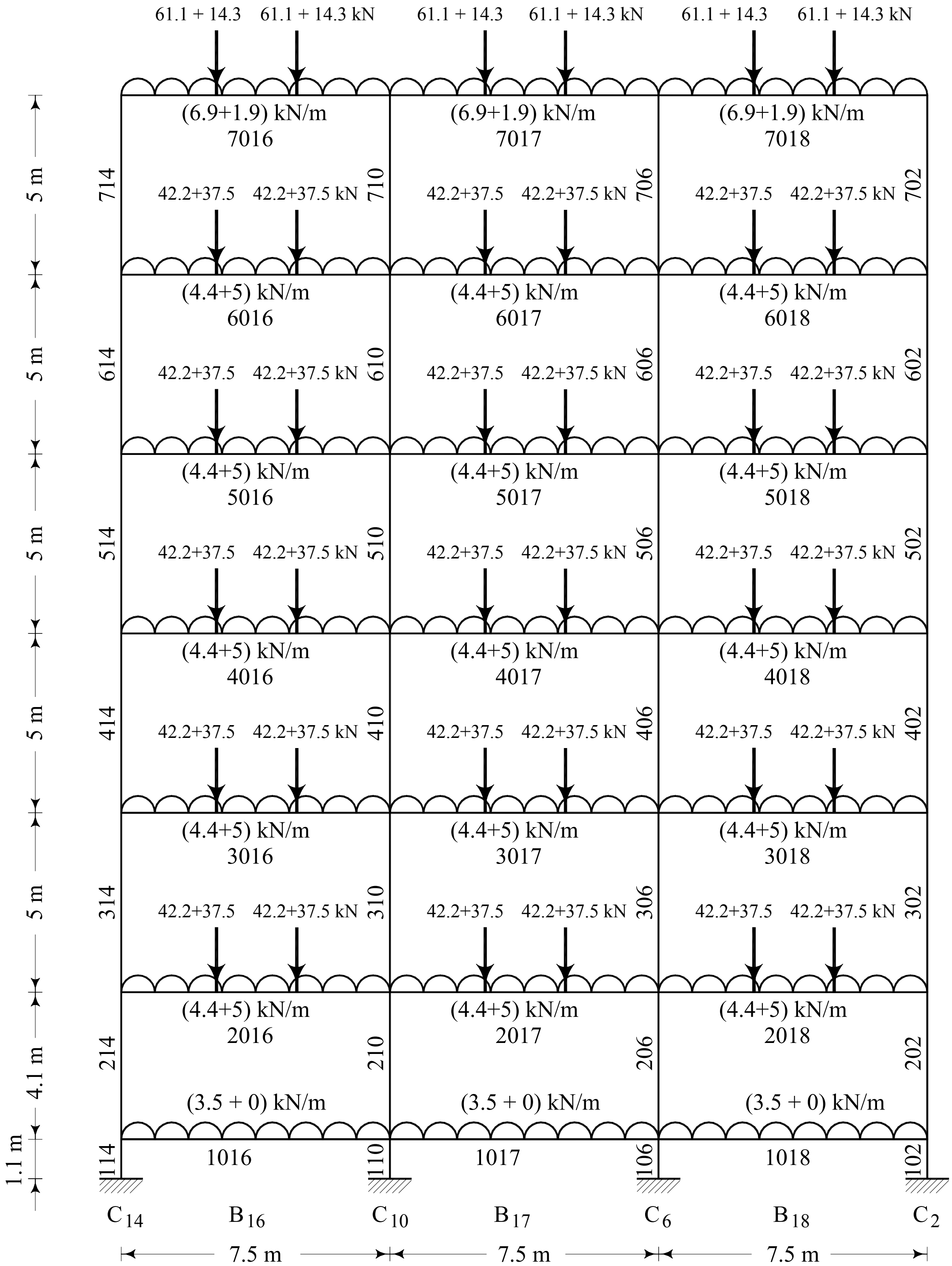


Figure 2(d) Gravity Loads: Frame 2-2

1.4. Seismic Weight Calculations

The seismic weights are calculated in a manner similar to gravity loads. The weight of columns and walls in any storey shall be equally distributed to the floors above and below the storey. Following reduced live loads are used for analysis: Zero on terrace, and 50% on other floors [IS: 1893 (Part 1): 2002, Clause 7.4)

(1) Storey 7 (Terrace):

		DL + LL
From slab	22.5 x 22.5 (5.5+0)	2 784 + 0
Parapet	4 x 22.5 (4.9 + 0)	441 + 0
Walls	0.5 x 4 x 22.5 x (21.6 + 0)	972 + 0
Secondary beams	18 x 7.5 x (2.5 + 0)	338 + 0
Main beams	8 x 22.5 x (4.5 + 0)	810 + 0
Columns	0.5 x 5 x 16 x (6.3 + 0)	252 + 0
Total		5 597 + 0 = 5 597 kN

(2) Storey 6, 5, 4, 3:

		DL + LL
From slab	22.5 x 22.5 x (3.5 + 0.5 x 4)	1 772 + 1 013
Walls	4 x 22.5 x (21.6 + 0)	1 944 + 0
Secondary beams	18 x 7.5 x (2.5 + 0)	338 + 0
Main beams	8 x 22.5 x (4.5 + 0)	810 + 0
Columns	16 x 5 x (6.3 + 0)	504+0
Total		5 368 +1 013 = 6 381 kN

(3) Storey 2:

		DL + LL
From slab	22.5 x 22.5 x (3.5 + 0.5 x 4)	1 772 + 1 013
Walls	0.5 x 4 x 22.5 x (21.6 + 0)	972 + 0
Walls	0.5 x 4 x 22.5 x (17.2 + 0)	774 + 0
Secondary beams	18 x 7.5 x (2.5 + 0)	338 + 0
Main beams	8 x 22.5 x (4.5 + 0)	810 + 0
Columns	16 x 0.5 x (5 + 4.1) x (6.3 + 0)	459 + 0
Total		5 125 +1 013 = 6 138 kN

(4) Storey 1 (plinth):

		DL + LL
Walls	0.5 x 4 x 22.5 (17.2 + 0)	774 + 0
Walls	0.5 x 4 x 22.5 x (3.5 + 0)	158 + 0
Main beams	8 x 22.5 x (4.5 + 0)	810 + 0
Column	16 x 0.5 x 4.1 x (6.3 + 0)	206 + 0
	16 x 0.5 x 1.1 x (9.0 + 0)	79 + 0
Total		2 027 + 0 = 2 027 kN

Seismic weight of the entire building

= 5 597 + 4 x 6 381 + 6 138 + 2 027

= 39 286 kN

The seismic weight of the floor is the lumped weight, which acts at the respective floor level at the centre of mass of the floor.

1.5. Design Seismic Load

The infill walls in upper floors may contain large openings, although the solid walls are considered in load calculations. Therefore, fundamental time period T is obtained by using the following formula:

$T_a = 0.075\ h^{0.75}$

[IS 1893 (Part 1):2002, Clause 7.6.1]

$= 0.075 \times (30.5)^{0.75}$

$= 0.97$ sec.

Zone factor, $Z = 0.16$ for Zone III

IS: 1893 (Part 1):2002, Table 2

Importance factor, $I = 1.5$ (public building)

Medium soil site and 5% damping

$$\frac{S_a}{g} = \frac{1.36}{0.97} = 1.402$$

IS: 1893 (Part 1): 2002, Figure 2.

Table1. Distribution of Total Horizontal Load to Different Floor Levels

Storey	W_i (kN)	h_i (m)	$W_i h_i^2$ x10^{-3}	Qi $= \frac{W_i h_i^2}{\Sigma W_i h_i^2}$ x V_B (kN)	V_i (kN)
7	5 597	30.2	5 105	480	480
6	6 381	25.2	4 052	380	860
5	6 381	20.2	2 604	244	1 104
4	6 381	15.2	1 474	138	1 242
3	6 381	10.2	664	62	1 304
2	6 138	5.2	166	16	1 320
1	2 027	1.1	3	0	1 320
Total			14 068	1 320	

$$\frac{S_a}{g} = \frac{1.36}{0.97} = 1.402$$

IS: 1893 (Part 1): 2002, Figure 2.

Ductile detailing is assumed for the structure. Hence, Response Reduction Factor, *R*, is taken equal to 5.0.

It may be noted however, that ductile detailing is mandatory in Zones III, IV and V.

Hence,

$$A_h = \frac{Z}{2} \times \frac{I}{R} \times \frac{S_a}{g}$$

$$= \frac{0.16}{2} \times \frac{1.5}{5} \times 1.402 = 0.0336$$

Base shear, $V_B = A_h W$

$= 0.0336 \times 39\ 286$

$= 1\ 320$ kN.

The total horizontal load of 1 320 kN is now distributed along the height of the building as per clause 7.7.1 of IS1893 (Part 1): 2002. This distribution is shown in Table 1.

1.5.1. Accidental eccentricity:

Design eccentricity is given by

$e_{di} = 1.5\ e_{si} + 0.05\ b_i$ or

$e_{si} – 0.05\ b_i$

IS 1893 (Part 1): 2002, Clause 7.9.2.

For the present case, since the building is symmetric, static eccentricity, $e_{si} = 0$.

$0.05\ b_i = 0.05 \times 22.5 = 1.125$ m.

Thus the load is eccentric by 1.125 m from mass centre. For the purpose of our calculations, eccentricity from centre of stiffness shall be calculated. Since the centre of mass and the centre of stiffness coincide in the present case, the eccentricity from the centre of stiffness is also 1.125 m.

Accidental eccentricity can be on either side (that is, plus or minus). Hence, one must consider lateral force Q_i acting at the centre of stiffness accompanied by a clockwise or an anticlockwise torsion moment (i.e., +1.125 Q_i kNm or -1.125 Q_i kNm).

Forces Q_i acting at the centres of stiffness and respective torsion moments at various levels for the example building are shown in Figure 3.

Note that the building structure is identical along the X- and Z- directions, and hence, the fundamental time period and the earthquake forces are the same in the two directions.

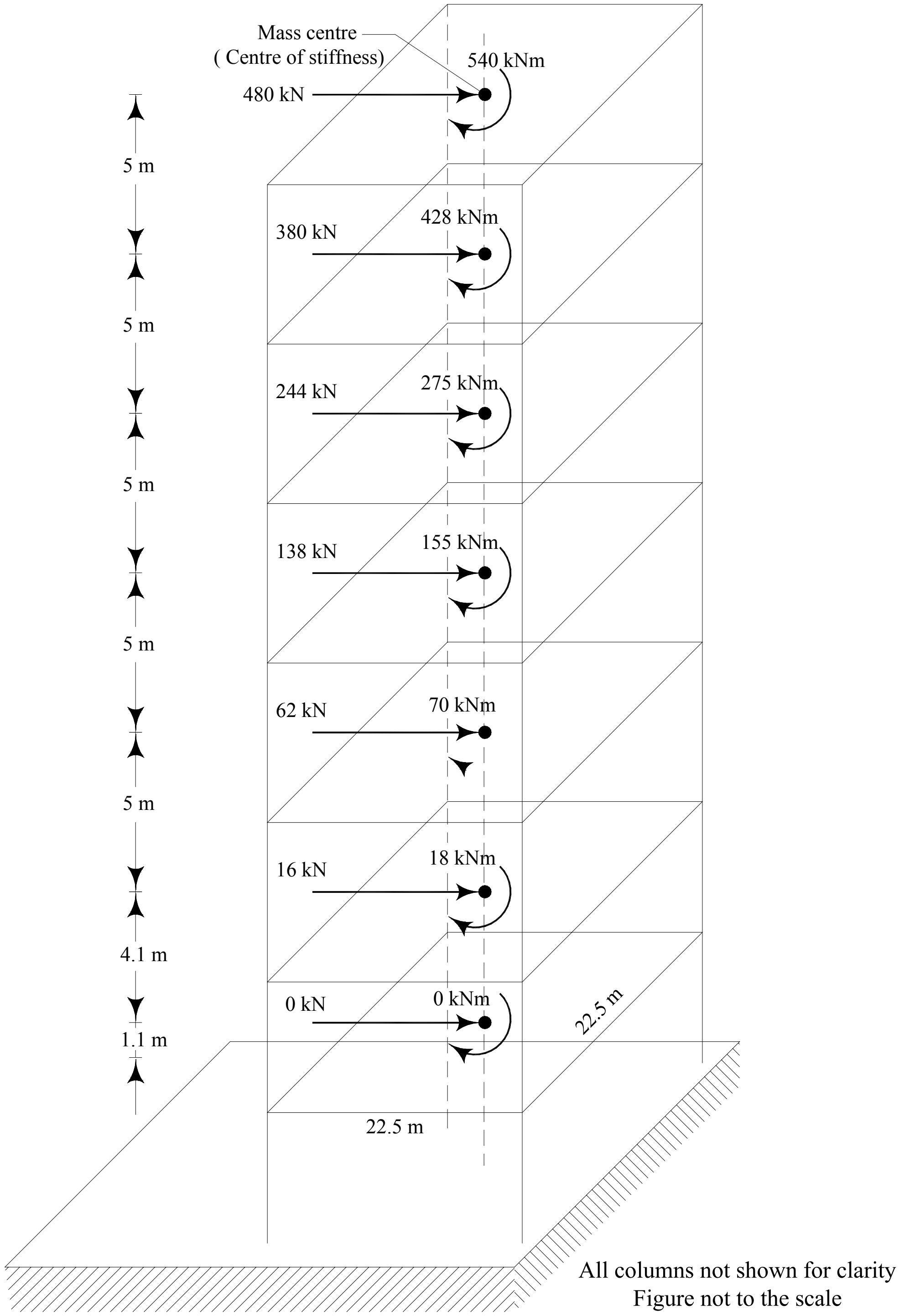


Figure 3 Accidental Eccentricity Inducing Torsion in the Building

1.6. Analysis by Space Frames

The space frame is modelled using standard software. The gravity loads are taken from Figure 2, while the earthquake loads are taken from Figure 3. The basic load cases are shown in Table 2, where X and Z are lateral orthogonal directions.

Table 2 Basic Load Cases Used for Analysis

No.	Load case	Directions
1	DL	Downwards
2	IL(Imposed/Live load)	Downwards
3	EXTP (+Torsion)	+X; Clockwise torsion due to EQ
4	EXTN (-Torsion)	+X; Anti-Clockwise torsion due to EQ
5	EZTP (+Torsion)	+Z; Clockwise torsion due to EQ
6	EZTN (-Torsion)	**+Z;** Anti-Clockwise torsion due to EQ

EXTP: EQ load in X direction with torsion positive

EXTN: EQ load in X direction with torsion negative

EZTP: EQ load in Z direction with torsion positive

EZTN: EQ load in Z direction with torsion negative.

1.7. Load Combinations

As per IS 1893 (Part 1): 2002 Clause no. 6.3.1.2, the following load cases have to be considered for analysis:

1.5 (DL + IL)

1.2 (DL + IL ± EL)

1.5 (DL ± EL)

0.9 DL ± 1.5 EL

Earthquake load must be considered for +X, -X, +Z and –Z directions. Moreover, accidental eccentricity can be such that it causes clockwise or anticlockwise moments. Thus, ±EL above implies 8 cases, and in all, 25 cases as per Table 3 must be considered. It is possible to reduce the load combinations to 13 instead of 25 by not using negative torsion considering the symmetry of the building. Since large amount of data is difficult to handle manually, all 25-load combinations are analysed using software.

For design of various building elements (beams or columns), the design data may be collected from computer output. Important design forces for selected beams will be tabulated and shown diagrammatically where needed. . In load combinations involving Imposed Loads (IL), IS 1893 (Part 1): 2002 recommends 50% of the imposed load to be considered for seismic weight calculations. However, the authors are of the opinion that the relaxation in the imposed load is unconservative. This example therefore, considers 100% imposed loads in load combinations.

For above load combinations, analysis is performed and results of deflections in each storey and forces in various elements are obtained.

Table 3 Load Combinations Used for Design

No.	Load combination
1	1.5 (DL + IL)
2	1.2 (DL + IL + EXTP)
3	1.2 (DL + IL + EXTN)
4	1.2 (DL + IL – EXTP)
5	1.2 (DL + IL – EXTN)
6	1.2 (DL + IL + EZTP)
7	1.2 (DL + IL + EZTN)
8	1.2 (DL + IL – EZTP)
9	1.2 (DL + IL – EZTN)
10	1.5 (DL + EXTP)
11	1.5 (DL + EXTN)
12	1.5 (DL – EXTP)
13	1.5 (DL – EXTN)
14	1.5 (DL + EZTP)
15	1.5 (DL + EZTN)
16	1.5 (DL – EZTP)
17	1.5 (DL – EZTN)

18	0.9 DL + 1.5 EXTP
19	0.9 DL + 1.5 EXTN
20	0.9 DL - 1.5 EXTP
21	0.9 DL - 1.5 EXTN
22	0.9 DL + 1.5 EZTP
23	0.9 DL + 1.5 EZTN
24	0.9 DL - 1.5 EZTP
25	0.9 DL - 1.5 EZTN

1.8. Storey Drift

As per Clause no. 7.11.1 of IS 1893 (Part 1): 2002, the storey drift in any storey due to specified design lateral force with partial load factor of 1.0, shall not exceed 0.004 times the storey height. From the frame analysis the displacements of the mass centres of various floors are obtained and are shown in Table 4 along with storey drift.

Since the building configuration is same in both the directions, the displacement values are same in either direction.

Table 4 Storey Drift Calculations

Storey	Displacement (mm)	Storey drift (mm)
7 (Fifth floor)	79.43	7.23
6 (Fourth floor)	72.20	12.19
5 (Third floor)	60.01	15.68
4 (Second floor)	44.33	17.58
3 (First floor)	26.75	17.26
2 (Ground floor)	9.49	9.08
1 (Below plinth)	0.41	0.41
0 (Footing top)	0	0

Maximum drift is for fourth storey = 17.58 mm.

Maximum drift permitted = 0.004 x 5000 = 20 mm. Hence, ok.

Sometimes it may so happen that the requirement of storey drift is not satisfied. However, as per Clause 7.11.1, IS: 1893 (Part 1): 2002; "For the purpose of displacement requirements only, it is permissible to use seismic force obtained from the computed fundamental period (T) of the building without the lower bound limit on design seismic force." In such cases one may check storey drifts by using the relatively lower magnitude seismic forces obtained from a dynamic analysis.

1.9. Stability Indices

It is necessary to check the stability indices as per Annex E of IS 456:2000 for all storeys to classify the columns in a given storey as non-sway or sway columns. Using data from Table 1 and Table 4, the stability indices are evaluated as shown in Table 5. The stability index Q_{si} of a storey is given by

$$Q_{si} = \frac{\sum P_u \Delta_u}{H_u h_s}$$

Where

Q_{si} = stability index of ith storey

$\sum P_u$ = sum of axial loads on all columns in the ith storey

Δ_u = elastically computed first order lateral deflection

H_u = total lateral force acting within the storey

h_s = height of the storey.

As per IS 456:2000, the column is classified as non-sway if $Q_{si} \leq 0.04$, otherwise, it is a sway column. It may be noted that both sway and non-sway columns are unbraced columns. For braced columns, $Q = 0$.

Table 5 Stability Indices of Different Storeys

Storey	Storey seismic weight Wi (kN)	Axial load $\Sigma P_u = \Sigma W_i$, (kN)	$\triangle_u$ (mm)	Lateral load $H_u = V_i$ (kN)	H_s (mm)	$Q_{si} = \frac{\Sigma P_u \Delta_u}{H_u h_s}$	Classification
7	5 597	5 597	7.23	480	5 000	0.0169	No-sway
6	6 381	11 978	12.19	860	5 000	0.0340	No-sway
5	6 381	18 359	15.68	1 104	5 000	0.0521	Sway
4	6 381	24 740	17.58	1 242	5 000	0.0700	Sway
3	6 381	31 121	17.26	1 304	5 000	0.0824	Sway
2	6 138	37 259	9.08	1 320	4 100	0.0625	Sway
1	2 027	39 286	0.41	1 320	1 100	0.0111	No-sway

1.10. Design of Selected Beams

The design of one of the exterior beam B2001-B2002-B2003 at level 2 along X-direction is illustrated here.

1.10.1. General requirements

The flexural members shall fulfil the following general requirements.

(IS 13920; Clause 6.1.2)

$$\frac{b}{D} \geq 0.3$$

Here $\quad \frac{b}{D} = \frac{300}{600} = 0.5 > 0.3$

Hence, ok.

(IS 13920; Clause 6.1.3)

$$b \geq 200 \quad \text{mm}$$

Here $b = 300\,\text{mm} \geq 200\,\text{mm}$

Hence, ok.

(IS 13920; Clause 6.1.4)

$$D \leq \frac{L_c}{4}$$

Here, $\quad L_c = 7500 - 500 = 7000$ mm

$$D = 600\ \text{mm} < \frac{7000}{4}\ \text{mm}$$

Hence, ok.

1.10.2. Bending Moments and Shear Forces

The end moments and end shears for six basic load cases obtained from computer analysis are given in Tables 6 and 7. Since earthquake load along Z-direction (EZTP and EZTN) induces very small moments and shears in these beams oriented along the X-direction, the same can be neglected from load combinations. Load combinations 6 to 9, 14 to 17, and 22 to 25 are thus not considered for these beams. Also, the effect of positive torsion (due to accidental eccentricity) for these beams will be more than that of negative torsion. Hence, the combinations 3, 5, 11, 13, 19 and 21 will not be considered in design. Thus, the combinations to be used for the design of these beams are 1, 2, 4, 10, 12, 18 and 20.

The software employed for analysis will however, check all the combinations for the design moments and shears. The end moments and end shears for these seven load combinations are given in Tables 8 and 9. Highlighted numbers in these tables indicate maximum values.

From the results of computer analysis, moment envelopes for B2001 and B2002 are drawn in Figures 4 (a) and 4 (b) for various load combinations, viz., the combinations 1, 2, 4,10,12,18 and 20. Design moments and shears at various locations for beams B2001-B2002–B2003 are given in Table 10.

To get an overall idea of design moments in beams at various floors, the design moments and shears for all beams in frame *A-A* are given in Tables 11 and 12. It may be noted that values of level 2 in Tables 11 and 12 are given in table 10.

Table 6 End Moments (kNm) for Six Basic Load Cases

S.No.	Load case	B2001		B2002		B2003	
		Left	Right	Left	Right	Left	Right
1	(DL)	117.95	-157.95	188.96	-188.96	157.95	-117.95
2	(IL/LL)	18.18	-29.85	58.81	-58.81	29.85	-18.18
3	(EXTP)	-239.75	-215.88	-197.41	-197.40	-215.90	-239.78
4	(EXTN)	-200.03	-180.19	-164.83	-164.83	-180.20	-200.05
5	(EZTP)	-18.28	-17.25	-16.32	-16.20	-18.38	-21.37
6	(EZTN)	19.39	16.61	14.58	14.70	15.47	16.31

Sign convention: Anti-clockwise moment (+); Clockwise moment (-)

Table 7 End Shears (kN) For Six Basic Load Cases

S.No.	Load case	B2001		B2002		B2003	
		Left	Right	Left	Right	Left	Right
1	(DL)	109.04	119.71	140.07	140.07	119.71	109.04
2	(IL/LL)	17.19	20.31	37.5	37.5	20.31	17.19
3	(EXTP)	-60.75	60.75	-52.64	52.64	-60.76	60.76
4	(EXTN)	-50.70	50.70	-43.95	43.95	-50.70	50.70
5	(EZTP)	-4.74	4.74	-4.34	4.34	-5.30	5.30
6	(EZTN)	4.80	-4.80	3.90	-3.90	4.24	-4.24

Sign convention: (+) = Upward force; (--) = Downward force

Table 8 Factored End Moments (kNm) for Load Combinations

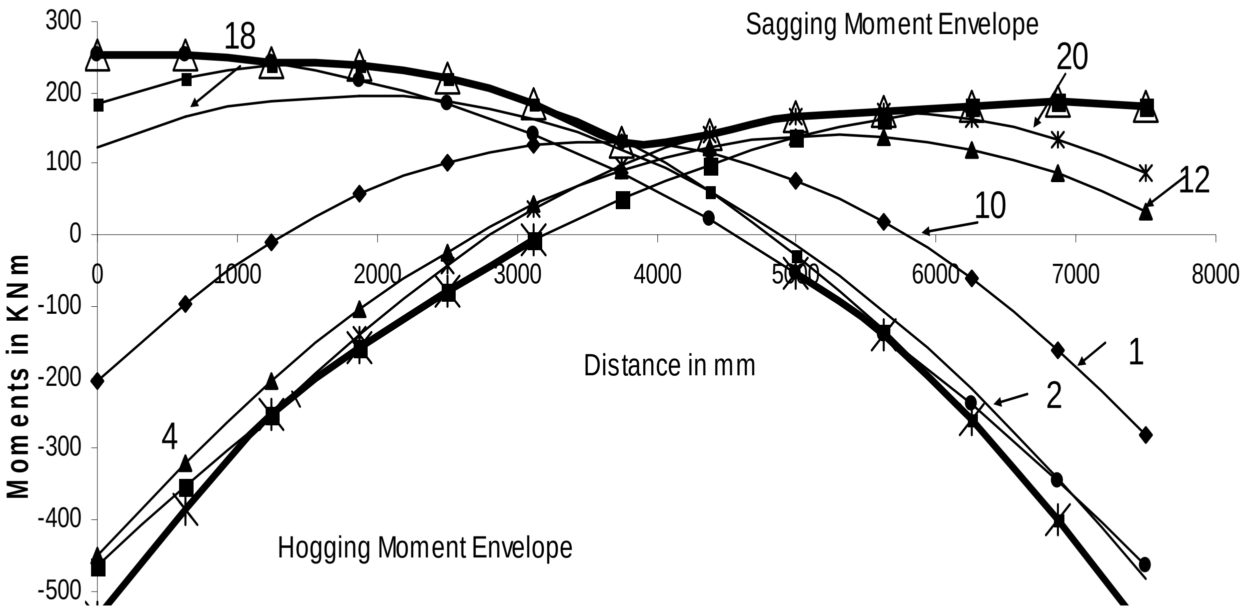
Combn No:	Load combination	B2001		B2002		B2003	
		Left	Right	Left	Right	Left	Right
1	[1.5(DL+IL)]	204.21	-281.71	371.66	-371.66	281.71	-204.21
2	[1.2(DL+IL+EXTP)]	-124.34	-484.43	60.44	-534.21	-33.71	-451.10
4	[1.2(DL+IL-EXTP)]	451.07	33.69	**534.21**	-60.44	484.45	124.37
10	[1.5(DL+EXTP)]	-182.69	**-560.76**	-12.66	**-579.55**	-86.91	**-536.60**
12	[1.5(DL-EXTP)]	**536.56**	86.90	579.55	12.66	**560.78**	182.73
18	[0.9DL+1.5EXTP]	**-253.47**	-465.99	**-126.04**	-466.18	**-181.69**	-465.82
20	[0.9DL-1.5EXTP]	465.79	**181.67**	466.18	**126.04**	466.00	**253.51**

Sign convention: (+) = Anti-clockwise moment; (--) = Clockwise moment

Table 9 Factored End Shears (kN) for Load Combinations

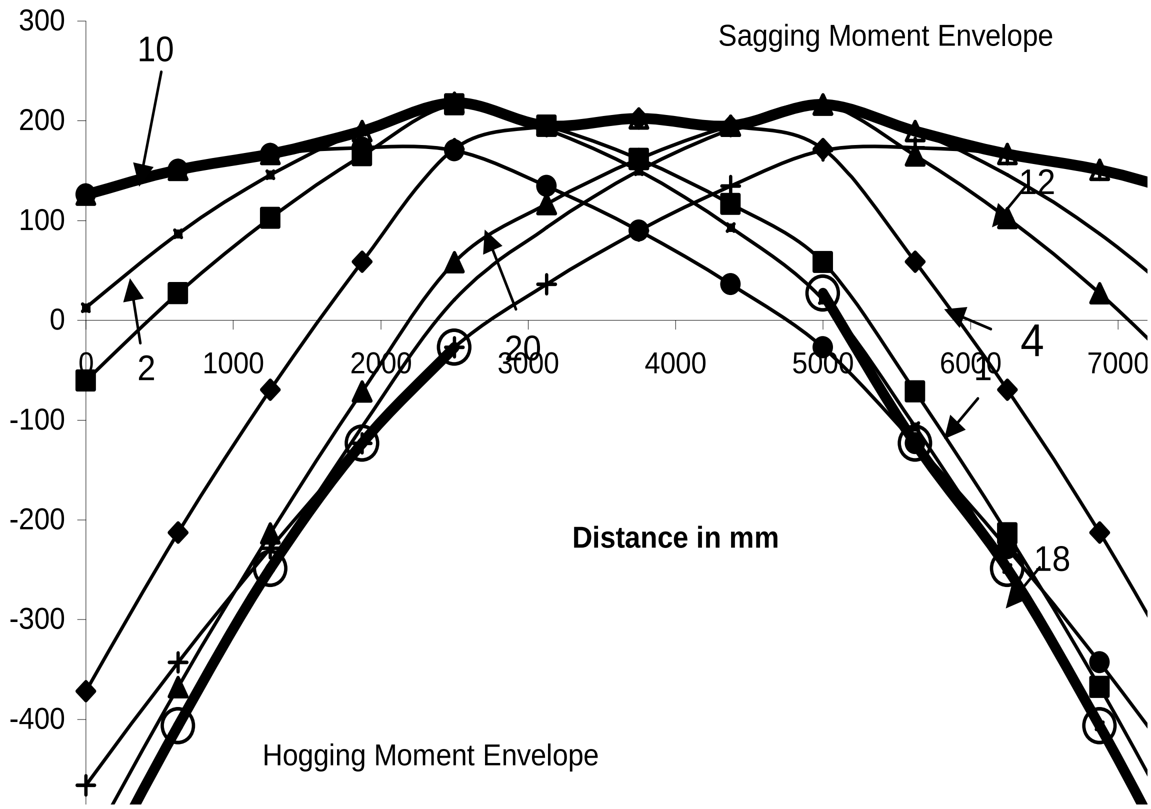
Combn No:	Load combination	B2001		B2002		B2003	
		Left	Right	Left	Right	Left	Right
1	[1.5(DL+IL)]	189.35	210.02	266.36	266.36	210.02	189.35
2	[1.2(DL+IL+EXTP)]	78.58	240.92	149.92	276.26	95.11	224.39
4	[1.2(DL+IL-EXTP)]	224.38	95.12	276.26	149.92	240.93	78.57
10	[1.5(DL+EXTP)]	72.44	**270.69**	131.15	**289.07**	88.43	**254.70**
12	[1.5(DL-EXTP)]	**254.69**	88.44	**289.07**	131.15	**270.70**	72.43
18	[0.9DL+1.5EXTP]	7.01	198.86	47.11	205.03	16.60	189.27
20	[0.9DL-1.5EXTP]	189.26	16.61	205.03	47.11	198.87	7.00

Sign convention: (+) = Upward force; (--) = Downward force



Note: 1, 2, 4,10,12,18 and 20 denote the moment envelopes for respective load combinations.

Figure 4(a) Moments Envelopes for Beam 2001



Note: 1, 2, 4,10,12,18 and 20 denote the moment envelopes for respective load combinations

Figure 4(b) Moment Envelopes for Beam 2002

Table 10 Design Moments and Shears at Various Locations

Beam →	B2001		B2002		B2003	
Distance from left end (mm)	Moment (kNm)	Shear (kN)	Moment (kNm)	Shear (kN)	Moment (kNm)	Shear (kN)
0	**-537** **253**	**255**	**-580** **126**	**289**	**-561** **182**	**271**
625	-386 252	226	-407 151	265	-401 188	242
1250	-254 241	198	-249 167	240	-258 181	214
1875	-159 238	169	-123 190	218	-141 172	185
2500	-78 221	140	-27 218	198	-55 165	156
3125	-8 186	112	0 195	103	0 140	128
3750	0 130	-99	0 202	79	0 130	99
4375	0 140	-128	0 195	-103	-8 186	-112
5000	-55 165	-156	-27 218	-128	-78 221	-140
5625	-141 172	-185	-123 190	-218	-159 238	-169
6250	-258 181	-214	-249 167	-240	-254 241	-198
6875	-401 187	-242	-407 151	-265	-386 253	-226
7500	**-561** **182**	**-271**	**-580** **126**	**-290**	**-537** **254**	**-255**

Table 11 Design Factored Moments (kNm) for Beams in Frame *AA*

Level	External Span (Beam $B_{1)}$							Internal Span (B_2)			
	0	1250	2500	3750	5000	6250	7500	0	1250	2500	3750
7 (-)	190	71	11	0	3	86	221	290	91	0	0
(+)	47	69	87	67	54	33	2	0	39	145	149
6 (-)	411	167	29	0	12	162	414	479	182	0	0
(+)	101	137	164	133	134	106	65	25	99	190	203
5 (-)	512	237	67	0	41	226	512	559	235	20	0
(+)	207	209	202	132	159	164	155	107	154	213	204
4 (-)	574	279	90	0	60	267	575	611	270	37	0
(+)	274	255	227	131	176	202	213	159	189	230	200
3 (-)	596	294	99	0	68	285	602	629	281	43	0
(+)	303	274	238	132	182	215	234	175	199	235	202
2 (-)	**537**	**254**	**78**	**0**	**55**	**259**	**561**	**580**	**249**	**27**	**0**
(+)	**253**	**241**	**221**	**130**	**165**	**181**	**182**	**126**	**167**	**218**	**202**
1 (-)	250	90	3	0	4	98	264	259	97	5	0
(+)	24	63	94	81	87	55	13	10	55	86	76

Table 12 Design Factored Shears (kN) for Beams in Frame *AA*

Level	External Span (Beam B_1)							Internal Span (B_2)			
	0	1250	2500	3750	5000	6250	7500	0	1250	2500	3750
7-7	110	79	49	-31	-61	-92	-123	168	150	133	-23
6-6	223	166	109	52	-116	-173	-230	266	216	177	52
5-5	249	191	134	77	-143	-200	-257	284	235	194	74
4-4	264	207	150	93	-160	-218	-275	298	247	205	88
3-3	270	213	155	98	-168	-225	-282	302	253	208	92
2-2	**255**	**198**	**140**	**-99**	**-156**	**-214**	**-271**	**289**	**240**	**198**	**79**
1-1	149	108	67	-31	-72	-112	-153	150	110	69	-28

1.10.3. Longitudinal Reinforcement

Consider mild exposure and maximum 10 mm diameter two-legged hoops. Then clear cover to main reinforcement is 20 +10 = 30 mm. Assume 25 mm diameter bars at top face and 20 mm diameter bars at bottom face. Then, d = 532 mm for two layers and 557 mm for one layer at top; d = 540 mm for two layers and 560 mm for one layer at bottom. Also consider d'/d = 0.1 for all doubly reinforced sections.

Design calculations at specific sections for flexure reinforcement for the member B2001 are shown in Table 13 and that for B2002 are tabulated in Table 14. In tables 13 and 14, the design moments at the face of the support, i.e., 250 mm from the centre of the support are calculated by linear interpolation between moment at centre and the moment at 625 mm from the centre from the table 10. The values of p_c and p_t have been obtained from SP: 16. By symmetry, design of beam B2003 is same as that of B2001. Design bending moments and required areas of reinforcement are shown in Tables 15 and 16. The underlined steel areas are due to the minimum steel requirements as per the code.

Table 17 gives the longitudinal reinforcement provided in the beams B2001, B 2002 and B2003.

Table 13 Flexure Design for B2001

Location from left support	M_u (kNm)	b (mm)	d (mm)	$\frac{M_u}{bd^2}$ (N/mm^2)	Type	p_t	p_c	A_{st} (mm^2)	A_{sc} (mm^2)
250	-477 +253	300 300	532 540	5.62 2.89	D S	1.86 0.96	0.71 -	2 969 1 555	1 133 -
1 250	-254 +241	300 300	532 540	2.99 2.75	S S	1.00 0.90	- -	1 596 1 458	- -
2 500	-78 +221	300 300	557 540	0.84 2.53	S S	0.25 0.81	- -	418 1 312	- -
3 750	0 +130	300 300	557 560	0 1.38	S S	0 0.42	- -	0 706	- -
5 000	-55 +165	300 300	557 540	0.59 1.89	S S	0.18 0.58	- -	301 940	- -
6 250	-258 +181	300 300	532 540	3.04 2.07	S S	1.02 0.65	- -	1 628 1 053	- -
7 250	-497 +182	300 300	532 540	5.85 2.08	D S	1.933 0.65	0.782 -	3 085 1 053	1 248 -

D = Doubly reinforced section; S = Singly reinforced section

Table 14 Flexure Design for B2002

Location from left support	M_u, (kNm)	b (mm)	d (mm)	$\frac{M_u}{bd^2}$, (kNm)	Type	p_t	p_c	A_{st} (mm^2)	A_{sc} (mm^2)
250	-511 +136	300 300	532 540	6.02 1.55	D S	1.99 0.466	0.84 -	3 176 755	744 ,-
1 250	-249 +167	300 300	532 540	2.93 1.91	S S	0.97 0.59	- -	1 548 956	- -
2 500	-27 +218	300 300	557 540	0.29 2.49	S S	0.09 0.80	- -	150 1 296	- -
3 750	0 +202	300 300	557 560	0 2.15	S S	0 0.67	- -	0 1 126	- -
5 000	-27 +218	300 300	557 540	0.29 2.49	S S	0.09 0.80	- -	150 1 296	- -
6 250	-249 +167	300 300	532 540	2.93 1.91	S S	0.97 0.59	- -	1 548 956	- -
7 250	-511 +136	300 300	532 540	6.02 1.55	D S	1.99 0.466	0.84 -	3 176 755	744 ,-

D = Doubly reinforced section; S = Singly reinforced section

Table 15 Summary of Flexure Design for B2001 and B2003

B2001	A						B
Distance from left (mm)	250	1250	2500	3750	5000	6250	7250
M (-) at top (kNm)	477	254	78	0	55	258	497
Effective depth *d* (mm)	532	532	557	557	557	532	532
A_{st}, top bars (mm^2)	2969	1596	<u>486</u>	<u>486</u>	<u>486</u>	1628	3085
A_{sc}, bottom bars (mm^2)	1133	-	-	-	-	-	1248
M (+) at bottom (kNm)	253	241	221	130	165	181	182
Effective depth *d* (mm)	540	540	540	560	540	540	540
A_{st}, (bottom bars) (mm^2)	1555	1458	1312	706	940	1053	1053

Table 16 Summary of Flexure Design for B2002

B2002	B					C	
Distance from left (mm)	250	1250	2500	3750	5000	6250	7250
M (-), at top (kNm)	511	249	27	0	27	249	511
Effective depth *d*, (mm)	532	532	557	557	557	532	532
A_{st}, top bars (mm^2)	3176	1548	<u>486</u>	<u>486</u>	<u>486</u>	1548	3176
A_{sc}, bottom bars (mm^2)	744	-	-	-	-	-	744
M (+) at bottom (kNm)	136	167	218	202	218	167	136
Effective depth *d*, (mm)	540	540	540	560	540	540	540
A_{st}, (bottom bars) (mm^2)	755	956	1296	1126	1296	956	755

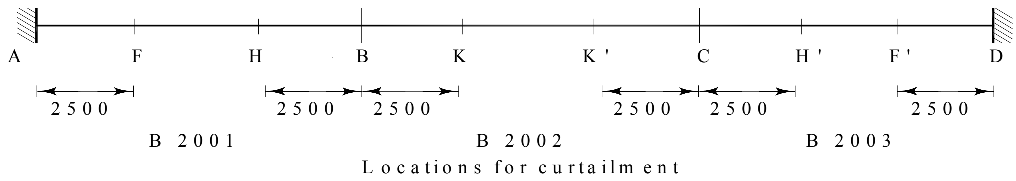


Figure 5 Critical Sections for the Beams

Table 17: Summary of longitudinal reinforcement provided in beams

B2001 and B2003		
At *A* and *D* (External supports)	Top bars Bottom bars	7 – 25 #, A_{st} = 3437 mm^2, with 250 mm (=10 d_b) internal radius at bend, where d_b is the diameter of the bar 6 – 20 #, A_{st} = 1884 mm^2, with 200 mm (=10 d_b) internal radius at bend
At Centre	Top bars Bottom bars	2- 25 #, A_{st} = 982 mm^2 5 – 20 #, A_{st} = 1570 mm^2
At *B* and *C* (Internal supports)	Top bars Bottom bars	7- 25 # , A_{st} = 3437 mm^2 6 – 20 #, A_{st} = 1884 mm^2
B2002		
At Centre	Top bars Bottom bars	2- 25 #, A_{st} = 982 mm^2 5 – 20 #, A_{st} = 1570 mm^2

At *A* and *D*, as per requirement of Table 14, 5-20 # bars are sufficient as bottom bars, though the area of the compression reinforcement then will not be equal to 50% of the tension steel as required by Clause 6.2.3 of IS 13920:1993. Therefore, at *A* and *D*, 6-20 # are provided at bottom. The designed section is detailed in Figure.6. The top bars at supports are extended in the spans for a distance of (l /3) = 2500 mm.

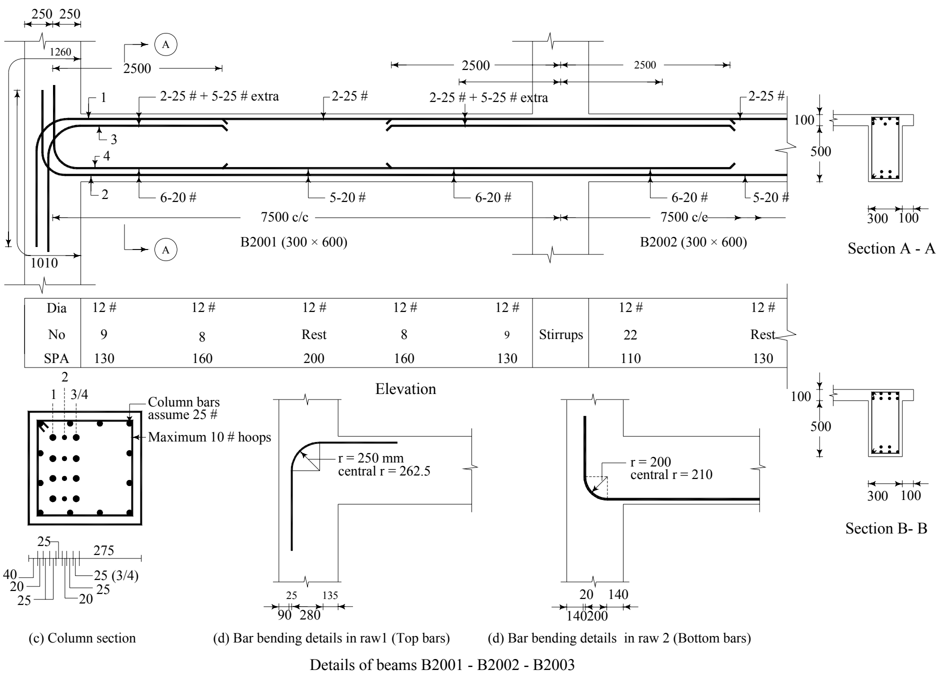


Figure 6 Details of Beams B2001, B2002 and B2003

1.10.3.1. Check for reinforcement

(IS 13920; Clause 6.2.1)

1.10.3.2. (a) Minimum two bars should be continuous at top and bottom.

Here, 2–25 mm # (982 mm^2) are continuous throughout at top; and 5–20 mm # (1 570 mm^2) are continuous throughout at bottom. Hence, ok.

(b) $p_{t,\min} = \dfrac{0.24\sqrt{f_{ck}}}{f_y} = \dfrac{0.24\sqrt{25}}{415}$

=0.00289, i.e., 0.289%.

$$A_{st,\min} = \frac{0.289}{100} \times 300 \times 560 = 486 \text{ mm}^2$$

Provided reinforcement is more. Hence, ok.

(IS 13920; Clause 6.2.2)
Maximum steel ratio on any face at any section should not exceed 2.5, i.e.,

$p_{\max} = 2.5\%.$

$$A_{st,\max} = \frac{2.5}{100} \times 300 \times 532 = 3990\, mm^2$$

Provided reinforcement is less. Hence ok.

(IS 13920; Clause 6.2.3)

The positive steel at a joint face must be at least equal to half the negative steel at that face.

Joint A

Half the negative steel = $\dfrac{3437}{2}$ = 1718 mm^2

Positive steel = 1884 mm^2 > 1718 mm^2
Hence, ok.

Joint B

Half the negative steel = $\dfrac{3437}{2}$ = 1718 mm^2

Positive steel = 1 884 mm^2 > 1 718 mm^2

Hence, ok.

(IS 13920; Clause 6.2.4)

Along the length of the beam,

A_{st} at top or bottom ≥ 0.25 A_{st} at top at joint A or B

A_{st} at top or bottom ≥ 0.25 × 3 437

≥ 859 mm^2

Hence, ok.

(IS 13920; Clause 6.2.5)

At external joint, anchorage of top and bottom bars = L_d in tension + 10 d_b.

L_d of Fe 415 steel in M25 concrete = 40.3 d_b

Here, minimum anchorage = 40.3 d_b + 10 d_b = 50.3 d_b. The bars must extend 50.3 d_b
(i.e. 50.3 x 25 = 1258 mm, say 1260 mm for 25 mm diameter bars and 50.3 x 20 = 1006 mm, say 1010 mm for 20 mm diameter bars) into the column.

At internal joint, both face bars of the beam shall be taken continuously through the column.

1.10.4. Web reinforcements

Vertical hoops (IS: 13920:1993, Clause 3.4 and Clause 6.3.1) shall be used as shear reinforcement.

Hoop diameter ≥ 6 mm

≥ 8 mm if clear span exceeds 5 m.

(IS 13920:1993; Clause 6.3.2)

Here, clear span = 7.5 – 0.5 = 7.0 m.

Use 8 mm (or more) diameter two-legged hoops.

The moment capacities as calculated in Table 18 at the supports for beam B2001 and B2003 are:

$M_u^{As} = 321\ \text{kNm}$ $\quad$ $M_u^{Bs} = 321\ \text{kNm}$

$M_u^{Ah} = 568\ \text{kNm}$ $\quad$ $M_u^{Bh} = 568\ \text{kNm}$

The moment capacities as calculated in Table 18 at the supports for beam B2002 are:

$M_u^{As} = 321\ \text{kNm}$ $\quad$ $M_u^{Bs} = 321\ \text{kNm}$

$M_u^{Ah} = 585\ \text{kNm}$ $\quad$ $M_u^{Bh} = 585\ \text{kNm}$

1.2 (DL+LL) for U.D.L. load on beam B2001 and B2003.

= 1.2 (30.5 + 5) = 42.6 kN/m.

1.2 (DL+LL) for U.D.L. load on beam B2002

= 1.2 (26.1 + 0) = 31.3 kN/m.

1.2 (DL+LL) for two point loads at third points on beam B2002

= 1.2 (42.2+37.5) = 95.6 kN.

The loads are inclusive of self-weights.

For beam B2001 and B2003:

$$V_a^{D+L} = V_b^{D+L} = 0.5 \times 7.5 \times 42.6 = 159.7\ kN.$$

<u>For beam 2002:</u>

$$V_a^{D+L} = V_b^{D+L} = 0.5 \times 7.5 \times 31.3 + 95.6 = 213\ \text{kN}.$$

Beam B2001 and B2003:

Sway to right

$$V_{u,a} = V_a^{D+L} - 1.4\left[\frac{M_{u,\lim}^{As} + M_{u,\lim}^{Bh}}{L_{AB}}\right]$$

$$= V_a^{D+L} - 1.4\left[\frac{321 + 568}{7.5}\right]$$

$$= 159.7 - 166 = -6.3 \text{ kN}$$

$$V_{u,b} = 159.7 + 166 = 325.7 \text{ kN}.$$

Sway to left

$$V_{u,a} = V_a^{D+L} - 1.4\left[\frac{M_{u,\lim}^{Ah} + M_{u,\lim}^{Bs}}{L_{AB}}\right]$$

$$= 159.7 - 1.4\left[\frac{568 + 321}{7.5}\right]$$

$$= 159.7 + 166 = 325.7 \text{ kN}$$

$$V_{u,b} = 159.7 - 166 = -6.3 \text{ kN}$$

Maximum design shear at A and B = 325.7 kN, say 326 kN

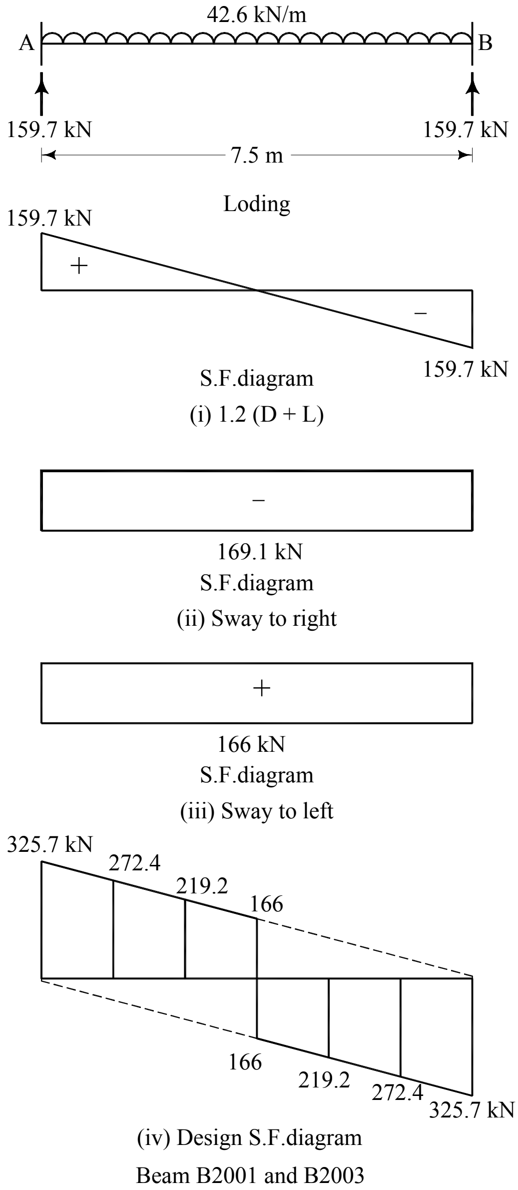


Figure 7 Beam Shears due to Plastic Hinge Formation for Beams B2001 and B2003

Beam 2002

Sway to right

$$V_{u,a} = V_a^{D+L} - 1.4\left[\frac{M_{u,\lim}^{As} + M_{u,\lim}^{Bh}}{L_{AB}}\right]$$

$$= V_a^{D+L} - 1.4\left[\frac{321 + 568}{7.5}\right]$$

$$= 213 - 166 = 47 \text{ kN}$$

$$V_{u,b} = 213 + 166 = 379 \text{ kN}.$$

Sway to left

$$V_{u,a} = 213 + 166 = 379 \text{ kN}$$

$$V_{u,b} = 213 - 166 = 47 \text{ kN}$$

Maximum design shear at A = 379 kN.

Maximum design shear at B = 379 kN.

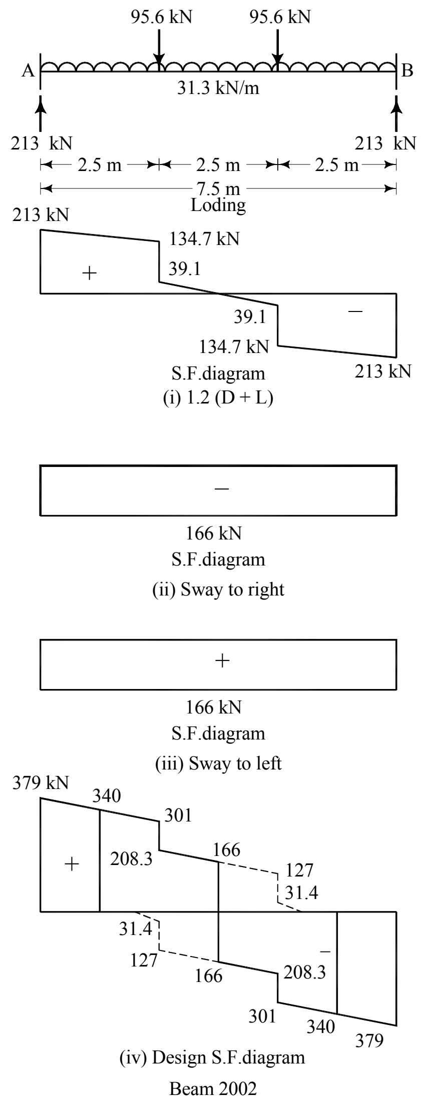


Figure 8 Beam Shears due to Plastic Hinge Formation for Beam B 2002

Maximum shear forces for various cases from analysis are shown in Table 19(a). The shear force to be resisted by vertical hoops shall be greater of:

i) Calculated factored shear force as per analysis.

ii) Shear force due to formation of plastic hinges at both ends of the beam plus the factored gravity load on the span.

The design shears for the beams B2001 and B2002 are summarized in Table 19.

As per Clause 6.3.5 of IS 13920:1993,the first stirrup shall be within 50 mm from the joint face.

Spacing, *s*, of hoops within 2 *d* (2 x 532 = 1064 mm) from the support shall not exceed:

(a) *d*/4 = 133 mm

(b) 8 times diameter of the smallest longitudinal bar = 8 x 20 = 160 mm

Hence, spacing of 133 mm c/c governs.

Elsewhere in the span, spacing, $s \leq \frac{d}{2} = \frac{532}{2} = 266$ mm.

Maximum nominal shear stress in the beam

$$\tau_c = \frac{379 \times 10^3}{300 \times 532} = 2.37 \ \text{N/mm}^2 < 3.1\,\text{N/mm}^2$$

($\tau_{c,max}$, for M25 mix)

The proposed provision of two-legged hoops and corresponding shear capacities of the sections are presented in Table 20.

Table 18 Calculations of Moment Capacities at Supports

	All sections are rectangular. For all sections: b = 300 mm, d = 532 mm, d'=60 mm, d'/d = 0.113 f_{sc} = 353 N/mm^2, $x_{u,max}$ = 0.48d = 255.3 mm.			
	M_u^{As} (kNm)	M_u^{Ah} (kNm)	M_u^{Bs} (kN-m)	M_u^{Bh} (kN-m)
Top bars	7-25 # = 3 437 mm^2	7-25 # = 3 437 mm^2	7-25 # = 3 437 mm^2	7-25 # = 3 437 mm^2
Bottom bars	6-20 # = 1 884 mm^2	6-20 # = 1 884 mm^2	6-20 # = 1 884 mm^2	6-20 # = 1 884 mm^2
A_{st} (mm^2)	1 884	3 437	1 884	3 437
A_{sc} (mm^2)	3 437	1 884	3 437	1 884
C_1= 0.36 $f_{ck}\, b\, x_u$ = $A\, x_u$	2 700 x_u	2 700 x_u	2 700 x_u	2 700 x_u
$C_2 = A_{sc}\, f_{sc}$ (kN)	1 213.2	665	1 213.2	665
T = 0.87 $f_y A_{st}$ (kN)	680.2	1 240.9	680.2	1 240.9
x_u= (T-C_2) /A	Negative i.e. x_u<d' Under-reinforced	213.3 x_u< $x_{u,max}$ Under-reinforced	Negative i.e. x_u<d' Under-reinforced	213.3 x_u< $x_{u,max}$ Under-reinforced
M_{uc1} = (0.36$f_{ck}\, b\, x_u$) $\times$(d-0.42x_u)	-	254	-	254
M_{uc2} = $A_{sc}\, f_{sc}$ (d-d')	-	314	-	314
M_u = 0.87$f_y A_{st}$ $\times$ (d-d')	321.06		321.06	
M_u = M_{u1}+ M_{u2}, (kNm)	321	568	321	568

Table 19 (a) Design Shears for Beam B2001 and B2003

B2001 B2003	A D						B C
Distance (mm)	0	1 250	2 500	3 750	5 000	6 250	7 500
Shear from analysis (kN)	255	198	140	-99	-156	-214	-271
Shear due to yielding (kN)	326	272	219	166	-219	-272	-326
Design shears	326	272	219	166	-219	-272	-326

Table 19 (b) Design Shears for Beam B2002

B2002	C						D
Distance (mm)	0	1 250	2 500	3 750	5 000	6 250	7 500
Shear (kN)	281	240	198	-79	-198	-240	-289
Shear due to yielding (kN)	379	340	301	166	-301	-340	-379
Design shears	379	340	301	166	-301	-340	-379

Table 20 Provisions of Two-Legged Hoops and Calculation of Shear Capacities

(a) Provision of two-legged hoops

	B2001 and B2003 (by symmetry)					B2002		
Distance (m)	0-1.25	1.25-2.5	2.5-5.0	5.0-6.25	6.25-7.5	0-2.5	2.5-5.0	5.0-7.5
Diameter (mm)	12	12	12	12	12	12	12	12
Spacing (mm)	130	160	200	160	130	110	130	110

(b)Calculation of Shear Capacities

	B2001 and B2003 (by symmetry)					B2002		
Distance (m)	0-1.25	1.25-2.5	2.5-5.0	5.0-6.25	6.25-7.5	0-2.5	2.5-5.0	5.0-7.5
V_u (kN)	326	272	219	272	326	379	301	379
B x d (mm)	300 x 532	300 x 540	300 x540	300 x540	300 x532	300x 532	300x540	300 x 532
V_{us}/d (N/mm)	628.6	510.4	408.3	510.4	628.6	742.4	628.6	742.4
V_{us} (kN)	334.4	275.6	220.4	275.6	334.4	395	334.4	395

Note: The shear resistance of concrete is neglected.

The designed beam is detailed in Figure 6.

1.11.Design of Selected Columns

Here, design of column C2 of external frame AA is illustrated. Before proceeding to the actual design calculations, it will be appropriate to briefly discuss the salient points of column design and detailing.

Design:

The column section shall be designed just above and just below the beam column joint, and larger of the two reinforcements shall be adopted. This is similar to what is done for design of continuous beam reinforcements at the support. The end moments and end shears are available from computer analysis. The design moment should include:

(a) The additional moment if any, due to long column effect as per clause 39.7 of IS 456:2000.

(b) The moments due to minimum eccentricity as per clause 25.4 of IS 456:2000.

All columns are subjected to biaxial moments and biaxial shears.

The longitudinal reinforcements are designed for axial force and biaxial moment as per IS: 456.

Since the analysis is carried out considering centre-line dimensions, it is necessary to calculate the moments at the top or at the bottom faces of the beam intersecting the column for economy. Noting that the B.M. diagram of any column is linear, assume that the points of contraflexure lie at 0.6 h from the top or bottom as the case may be; where h is the height of the column. Then obtain the column moment at the face of the beam by similar triangles. This will not be applicable to columns of storey 1 since they do not have points of contraflexure.

Referring to figure 9, if M is the centre-line moment in the column obtained by analysis, its moment at the beam face will be:

0.9 M for columns of 3 to 7th storeys, and

0.878 M for columns of storey 2.

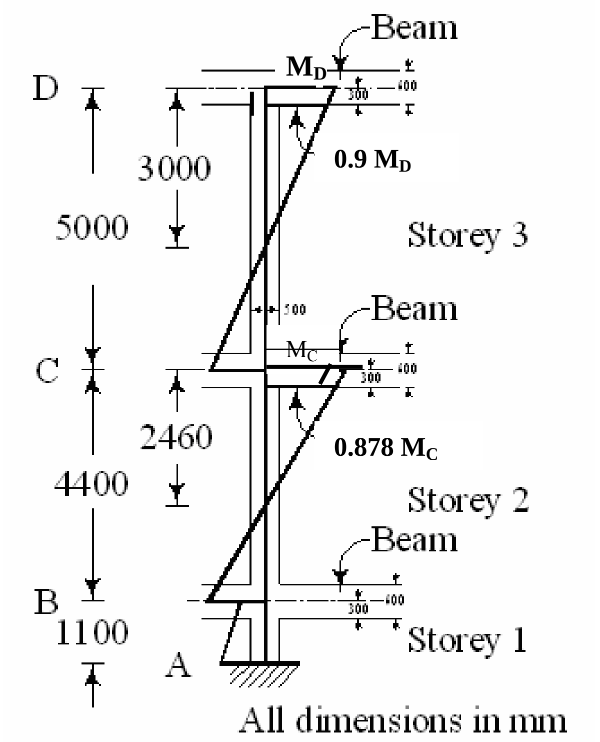


Figure 9 Determining moments in the column at the face of the beam.

Critical load combination may be obtained by inspection of analysis results. In the present example, the building is symmetrical and all columns are of square section. To obtain a trial section, the following procedure may be used:

Let a rectangular column of size *b* x *D* be subjected to P_u, M_{ux} (moment about major axis) and M_{uz} (moment about minor axis). The trial section with uniaxial moment is obtained for axial load and a combination of moments about the minor and major axis.

For the trial section

$$P_u^{'} = P_u \text{ and } M_{uz}^{'} = M_{uz} + \frac{b}{D} M_{ux} .$$

Determine trial reinforcement for all or a few predominant (may be 5 to 8) combinations and arrive at a trial section.

It may be emphasized that it is necessary to check the trial section for all combinations of loads since it is rather difficult to judge the governing combination by visual inspection.

Detailing:

Detailing of reinforcement as obtained above is discussed in context with Figure 10. Figure 10(a) shows the reinforcement area as obtained above at various column-floor joints for lower and upper column length. The areas shown in this figure are fictitious and used for explanation purpose only. The area required at the beam-column joint shall have the larger of the two values, viz., for upper length and lower length. Accordingly the areas required at the joint are shown in Figure. 10 (b).

Since laps can be provided only in the central half of the column, the column length for the purpose of detailing will be from the centre of the lower column to the centre of the upper column. This length will be known by the designation of the lower column as indicated in Figure 9(b).

It may be noted that analysis results may be such that the column may require larger amounts of reinforcement in an upper storey as compared to the lower storey. This may appear odd but should be acceptable.

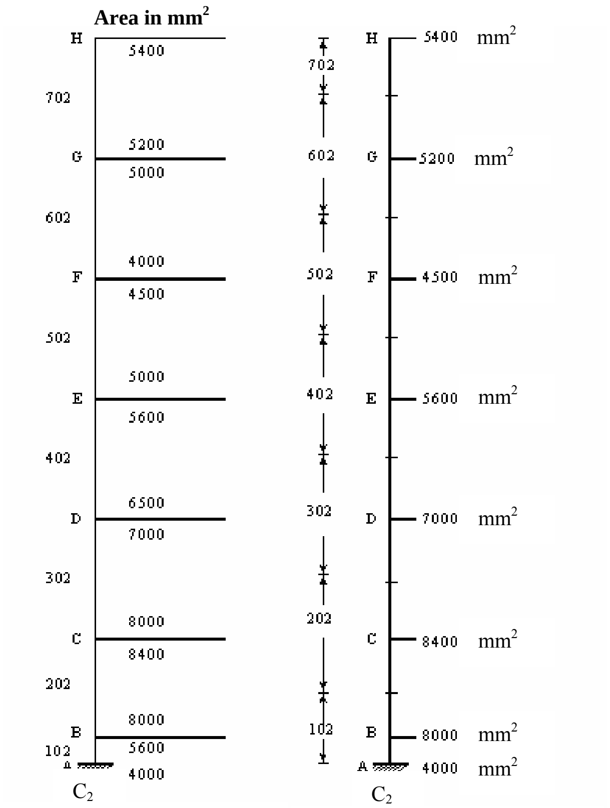
1.11.1. Effective length calculations:

Effective length calculations are performed in accordance with Clause 25.2 and Annex E of IS 456:2000.

Stiffness factor

Stiffness factors (I / *l*) are calculated in Table 21. Since lengths of the members about both the bending axes are the same, the suffix specifying the directions is dropped.

Effective lengths of the selected columns are calculated in Table 22 and Table 23.



(a) Required areas (fictitious) **(b) Proposed areas at joints**

Figure 10 Description of procedure to assume reinforcement in a typical column

Table 21 Stiffness factors for Selected Members

Member	Size (mm)	I (mm^4)	*l* (mm)	Stiffness Factor $(I/l)x10^{-3}$
All Beams	300 x 600	5.4 x 10^9	7 500	720
Columns				
C101, C102	600 x 600	1.08 x 10^{10}	1 100	9 818
C201, C202	500 x 500	5.2 x 10^9	4 100	1 268
C301, C302	500 x 500	5.2 x 10^9	5 000	1 040
C401, C402	500x 500	5.2 x 10^9	5 000	1 040

Table 22 Effective Lengths of Columns 101, 201 and 301

Column no.	Unsupp. Length	K_c	Upper joint $\Sigma(K_c + K_b)$	Lower joint $\Sigma(K_c + K_b)$	β_1	β_2	l_{ef}/L	l_{ef}	l_{ef}/b or l_{ef}/D	Type
About Z (EQ In X direction)										
101 (Non-sway)	800	9 818	9 818 +1 268 + 720 = 11 806	Infinite	0.832	0	0.67	536	1.07	Pedestal
201 (Sway)	3 500	1 268	1 040 +1 268 +720 = 3 028	9 818+1 268+720 = 11 806	0.418	0.107	1.22 ≥1.2	4 270	8.54	Short
301 (Sway)	4 400	1 040	1 040 +1 040 +720 = 2 800	1 040 +1 268 +720 = 3 028	0.371	0.341	1.28 ≥1.2	5 632	11.26	Short
About X (EQ In Z direction)										
101 (No-sway)	800	9 818	9 818 +1 268 +720 = 11 806	Infinite	0.832	0	0.67	536	1.07	Pedestal
201 (Sway)	3 500	1 268	1 040 +1 268 +720 = 3 028	9 818 +1 268 +720 = 11 806	0.418	0.107	1.22 ≥1.2	4 270	8.54	Short
301 (Sway)	4 400	1 040	1 040 +1 040 +720 = 2 800	1 040 +1 268 +720 = 3 028	0.371	0.341	1.28 ≥1.2	5 632	11.26	Short

Table 23 Effective Lengths of Columns 102, 202 and 302

Column no.	Unsupp. Length	K_c	Upper joint $\Sigma(K_c + K_b)$	Lower joint $\Sigma(K_c + K_b)$	β_1	β_2	l_{ef}/L	l_{ef}	l_{ef}/b or l_{ef}/D	Type
About Z (EQ In X direction)										
102 (No-sway)	800	9 818	9 818 +1 268 +720 x 2 = 12 526	Infinite	0.784	0	0.65	520	1.04	Pedestal
202 (Sway)	3 500	1 268	1 040 +1 268 +720 x 2 = 3 748	9 818 +1 268 +720 x 2 = 12 526	0.338	0.101	1.16 Hence use 1.2	4 200	8.4	Short
302 (Sway)	4 400	1 040	1 040 x 2 +720 x 2 = 3 520	1 040 +1 268 +720 x 2 = 3 748	0.295	0.277	1.21 Hence use 1.2	5 324	10.65	Short
About X (EQ In Z direction)										
102 (No-sway)	800	9 818	9 818 +1 268 +720 = 11 806	Infinite	0.832	0	0.67	536	1.07	Pedestal
202 (Sway)	3 500	1 268	1 040 +1 268+720 = 3 028	9 818 +1 268 +720 = 11,806	0.418	0.107	1.22 Hence use 1.2	4 270	8.54	Short
302 (Sway)	4 400	1 040	1 040 +1 040 +720 = 2 800	1 040 +1 268 +720 = 3 028	0.371	0.341	1.28 Hence use 1.2	5 632	11.26	Short

1.11.2. Determination of trial section:

The axial loads and moments from computer analysis for the lower length of column 202 are shown in Table 24 and those for the upper length of the column are shown in Table 26.In these tables, calculations for arriving at trial sections are also given. The calculations are performed as described in Section 1.11.1 and Figure 10.

Since all the column are short, there will not be any additional moment due to slenderness. The minimum eccentricity is given by

$$e_{\min} = \frac{L}{500} + \frac{D}{30}$$

(IS 456:2000, Clause 25.4)

For lower height of column, L = 4,100 – 600 = 3,500 mm.

$$e_{x,\min} = e_{y,\min} = \frac{3500}{500} + \frac{500}{30} = 23.66mm > 20mm$$

$e_{x,\min} = e_{z,\min}$ = 23.7 mm.

Similarly, for all the columns in first and second storey, $e_{x,\min} = e_{y,\min}$ = 25 mm.

For upper height of column, L = 5,000 – 600 = 4,400 mm.

$$e_{x,\min} = e_{z,\min} = \frac{4,400}{500} + \frac{500}{30} = 25.46mm > 20mm$$

For all columns in 3rd to 7th storey.

$e_{x,\min} = e_{z,\min}$ = 25.46 mm.

For column C_2 in all floors, i.e., columns C102, C202, C302, C402, C502, C602 and C702, f_{ck} = 25 N/mm^2, f_y = 415 N/mm^2, and $\frac{d'}{d} = \frac{50}{500} = 0.1$.

Calculations of Table 25 and 27 are based on uniaxial moment considering steel on two opposite faces and hence, Chart 32 of SP: 16 is used for determining the trial areas. Reinforcement obtained for the trial section is equally distributed on all four sides. Then, Chart 44 of SP: 16 is used for checking the column sections, the results being summarized in Tables 25 and 27.

The trial steel area required for section below joint C of C202 (from Table 25) is p/f_{ck} = 0.105 for load combination 1 whereas that for section above joint C, (from Table 27) is p/f_{ck} = 0.11 for load combination 12.

For lower length, $\frac{p}{f_{ck}} = 0.105$,

i.e., p = 0.105 x 25 = 2.625, and

$$A_{sc} = \frac{pbD}{100} = \frac{2.625 \times 500 \times 500}{100} = 6562\ mm^2.$$

For upper length, $\frac{p}{f_{ck}} = 0.11$,

i.e., p = 0.11 x 25 = 2.75, and

$$A_{sc} = \frac{pbD}{100} = \frac{2.75 \times 500 \times 500}{100} = 6875\ mm^2.$$

Trial steel areas required for column lengths C102, C202, C302, etc., can be determined in a similar manner. The trial steel areas required at various locations are shown in Figure 10(a). As described in Section 1.12. the trial reinforcements are subsequently selected and provided as shown in figure 11 (b) and figure 11 (c). Calculations shown in Tables 25 and 27 for checking the trial sections are based on provided steel areas.

For example, for column C202 (mid-height of second storey to the mid-height of third storey), provide 8-25 # + 8-22 # = 6968 mm^2, equally distributed on all faces.

A_{sc} = 6968 mm^2, p = 2.787, $\frac{p}{f_{ck}} = 0.111$.

P_{uz} = [0.45 x 25(500 x 500 – 6968)

+ 0.75 x 415 x 6968] x 10^{-3} = 4902 kN.

Calculations given in Tables 24 to 27 are self-explanatory.

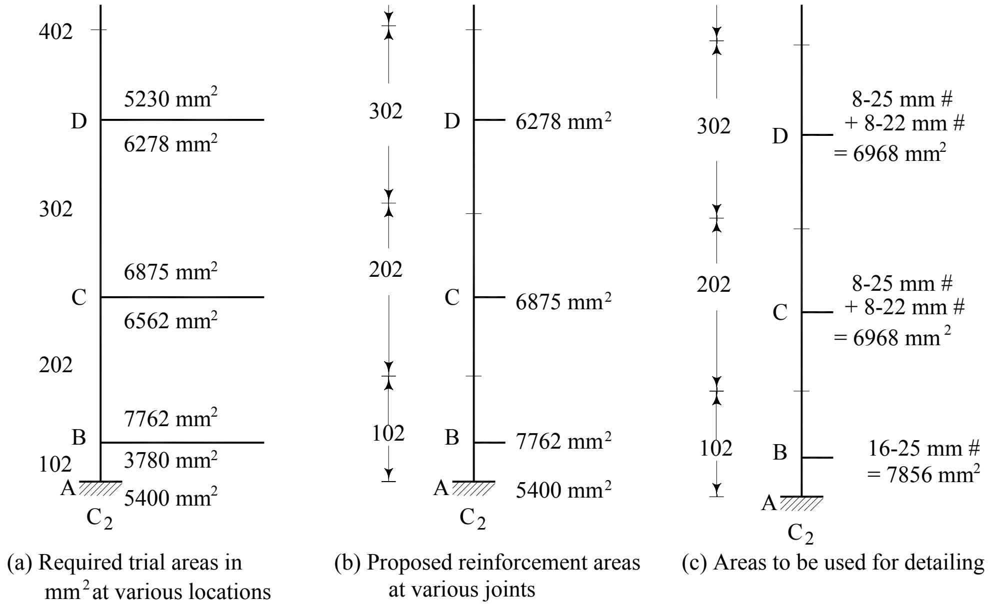


Figure 11 Required Area of Steel at Various Sections in Column

TABLE 24 TRIAL SECTION BELOW JOINT C

Comb. No.	P_u, kN	Centreline moment		Moment at face		Cal. Ecc.,mm		Des. Ecc.,mm		M_{ux}, kNm	M_{uz}, kNm	P'_u	M'_{uz}	$\frac{P_u'}{f_{ck}bD}$	$\frac{M_u'}{f_{ck}bD^2}$	$\frac{p}{f_{ck}}$
		M_{ux}, kNm	M_{uz}, kNm	M_{ux}, kNm	M_{uz}, kNm	e_x	e_z	e_{dx}	e_{dz}							
1	4002	107	36	93.946	31.608	23.47	7.90	25.00	25.00	100	100	4002	200	0.64	0.06	0.105
2	3253	89	179	78.14	157.16	24.02	48.31	25.00	48.31	81	157	3253	238	0.52	0.08	0.083
3	3225	83	145	72.87	127.31	22.60	39.48	25.00	39.48	81	127	3225	208	0.52	0.07	0.078
4	3151	82	238	72.00	208.96	22.85	66.32	25.00	66.32	79	209	3151	288	0.50	0.09	0.083
5	3179	88	203	77.26	178.23	24.30	56.07	25.00	56.07	79	178	3179	258	0.51	0.08	0.08
6	2833	17	12	14.93	10.54	5.27	3.72	25.00	25.00	71	71	2833	142	0.45	0.05	0.042
7	2805	23	45	20.19	39.51	7.20	14.09	25.00	25.00	70	70	2805	140	0.45	0.04	0.038
8	3571	189	46	165.94	40.39	46.47	11.31	46.47	25.00	166	89	3571	255	0.57	0.08	0.096
9	3598	195	13	171.21	11.41	47.58	3.17	47.58	25.00	171	90	3598	261	0.58	0.08	0.1
10	3155	65	242	57.07	212.48	18.09	67.35	25.00	67.35	79	212	3155	291	0.50	0.09	0.083
11	3120	58	199	50.92	174.72	16.32	56.00	25.00	56.00	78	175	3120	253	0.50	0.08	0.079
12	3027	57	279	50.05	244.96	16.53	80.93	25.00	80.93	76	245	3027	321	0.48	0.10	0.097
13	3063	65	236	57.07	207.21	18.63	67.65	25.00	67.65	77	207	3063	284	0.49	0.09	0.082
14	2630	68	3	59.70	2.63	22.70	1.00	25.00	25.00	66	66	2630	132	0.42	0.04	0.024
15	2596	75	38	65.85	33.36	25.37	12.85	25.37	25.00	66	65	2596	131	0.42	0.04	0.024
16	3552	190	40	166.82	35.12	46.97	9.89	46.97	25.00	167	89	3552	256	0.57	0.08	0.1
17	3587	198	1	173.84	0.88	48.47	0.24	48.47	25.00	174	90	3587	264	0.57	0.08	0.1
18	1919	41	249	36.00	218.62	18.76	113.92	25.00	113.92	48	219	1919	267	0.31	0.09	0.04
19	1883	33	206	28.97	180.87	15.39	96.05	25.00	96.05	47	181	1883	228	0.30	0.07	0.023
20	1791	33	272	28.97	238.82	16.18	133.34	25.00	133.34	45	239	1791	284	0.29	0.09	0.038
21	1826	40	229	35.12	201.06	19.23	110.11	25.00	110.11	46	201	1826	247	0.29	0.08	0.03
22	1394	92	10	80.78	8.78	57.95	6.30	57.95	25.00	81	35	1394	116	0.22	0.04	negative
23	1359	100	31	87.80	27.22	64.61	20.03	64.61	25.00	88	34	1359	122	0.22	0.04	negative
24	2316	166	32	145.75	28.10	62.93	12.13	62.93	25.00	146	58	2316	204	0.37	0.07	0.038
25	2351	173	9	151.89	7.90	64.61	3.36	64.61	25.00	152	59	2351	211	0.38	0.07	0.04

TABLE 25
CHECKING THE DESIGN OF TABLE 24

Comb. No.	P_u	$\frac{P_u}{P_{uz}}$	α_n	$\frac{P_u}{f_{ck}bD}$	M_{ux}, kNm	M_{uz}, kNm	$\frac{M_{u1}}{f_{ck}bd^2}$	M_{u1}	$\left[\frac{M_{ux}}{M_{u1}}\right]^{\alpha_n}$	$\left[\frac{M_{uz}}{M_{u1}}\right]^{\alpha_n}$	Check
1	4002	0.82	2.03	0.64	100	100	0.09	281	0.123	0.123	0.246
2	3253	0.66	1.77	0.52	81	157	0.13	406	0.058	0.186	0.243
3	3225	0.66	1.76	0.52	81	127	0.13	406	0.058	0.129	0.187
4	3151	0.64	1.74	0.50	79	209	0.13	406	0.058	0.315	0.373
5	3179	0.65	1.75	0.51	79	178	0.13	406	0.058	0.237	0.295
6	2833	0.58	1.63	0.45	71	71	0.135	422	0.055	0.055	0.109
7	2805	0.57	1.62	0.45	70	70	0.135	422	0.055	0.055	0.109
8	3571	0.73	1.88	0.57	166	89	0.105	328	0.277	0.086	0.364
9	3598	0.73	1.89	0.58	171	90	0.105	328	0.292	0.087	0.379
10	3155	0.64	1.74	0.50	79	212	0.13	406	0.058	0.324	0.382
11	3120	0.64	1.73	0.50	78	175	0.13	406	0.058	0.233	0.291
12	3027	0.62	1.70	0.48	76	245	0.135	422	0.054	0.398	0.452
13	3063	0.62	1.71	0.49	77	207	0.135	422	0.054	0.297	0.351
14	2630	0.54	1.56	0.42	66	66	0.145	453	0.049	0.049	0.098
15	2596	0.53	1.55	0.42	66	65	0.145	453	0.050	0.049	0.100
16	3552	0.72	1.87	0.57	167	89	0.105	328	0.281	0.086	0.368
17	3587	0.73	1.89	0.57	174	90	0.105	328	0.302	0.087	0.388
18	1919	0.39	1.32	0.31	48	219	0.17	531	0.042	0.310	0.352
19	1883	0.38	1.31	0.30	47	181	0.18	563	0.039	0.227	0.266
20	1791	0.37	1.28	0.29	45	239	0.18	563	0.040	0.335	0.375
21	1826	0.37	1.29	0.29	46	201	0.18	563	0.039	0.266	0.305
22	1394	0.28	1.14	0.22	81	35	0.175	547	0.113	0.043	0.156
23	1359	0.28	1.13	0.22	88	34	0.175	547	0.127	0.043	0.170
24	2316	0.47	1.45	0.37	146	58	0.16	500	0.166	0.043	0.210
25	2351	0.48	1.47	0.38	152	59	0.16	500	0.174	0.043	0.218

TABLE 26 TRIAL SECTION ABOVE JOINT C																
Comb. No.	P_u, kN	Centreline moment		Moment at face		Cal. Ecc.,mm		Des. Ecc.,mm		M_{ux}, kNm	M_{uz}, kNm	P'_u	M'_{uz}	$\frac{P'_u}{f_{ck}bD}$	$\frac{M'_u}{f_{ck}bD^2}$	$\frac{p}{f_{ck}}$
		M_{ux}, kNm	M_{uz}, kNm	M_{ux}, kNm	M_{uz}, kNm	e_x	e_z	e_{dx}	e_{dz}							
1	3339	131	47	117.9	42.3	35.31	12.67	35.31	25.00	118	83	3339	201	0.53	0.06	0.075
2	2710	111	293	99.9	263.7	36.86	97.31	36.86	97.31	100	264	2710	364	0.43	0.12	0.095
3	2687	99	238	89.1	214.2	33.16	79.72	33.16	79.72	89	214	2687	303	0.43	0.10	0.075
4	2632	98	368	88.2	331.2	33.51	125.84	33.51	125.84	88	331	2632	419	0.42	0.13	0.1
5	2654	110	313	99	281.7	37.30	106.14	37.30	106.14	99	282	2654	381	0.42	0.12	0.09
6	2377	87	11	78.3	9.9	32.94	4.16	32.94	25.00	78	59	2377	138	0.38	0.04	0.018
7	2355	98	63	88.2	56.7	37.45	24.08	37.45	25.00	88	59	2355	147	0.38	0.05	0.022
8	2965	296	65	266.4	58.5	89.85	19.73	89.85	25.00	266	74	2965	341	0.47	0.11	0.095
9	2987	307	13	276.3	11.7	92.50	3.92	92.50	25.00	276	75	2987	351	0.48	0.11	0.096
10	2643	78	389	70.2	350.1	26.56	132.46	26.56	132.46	70	350	2643	420	0.42	0.13	0.1
11	2616	64	321	57.6	288.9	22.02	110.44	25.00	110.44	65	289	2616	354	0.42	0.11	0.082
12	2547	63	437	56.7	393.3	22.26	154.42	25.00	154.42	64	393	2547	457	0.41	0.15	0.11
13	2548	77	368	69.3	331.2	27.20	129.98	27.20	129.98	69	331	2548	401	0.41	0.13	0.096
14	2228	169	10	152.1	9	68.27	4.04	68.27	25.00	152	56	2228	208	0.36	0.07	0.038
15	2201	183	55	164.7	49.5	74.83	22.49	74.83	25.00	165	55	2201	220	0.35	0.07	0.037
16	2963	310	58	279	52.2	94.16	17.62	94.16	25.00	279	74	2963	353	0.47	0.11	0.095
17	2990	324	7	291.6	6.3	97.53	2.11	97.53	25.00	292	75	2990	366	0.48	0.12	0.102
18	1605	50	399	45	359.1	28.04	223.74	28.04	223.74	45	359	1605	404	0.26	0.13	0.062
19	1577	36	330	32.4	297	20.55	188.33	25.00	188.33	39	297	1577	336	0.25	0.11	0.046
20	1509	35	427	31.5	384.3	20.87	254.67	25.00	254.67	38	384	1509	422	0.24	0.14	0.07
21	1537	49	358	44.1	322.2	28.69	209.63	28.69	209.63	44	322	1537	366	0.25	0.12	0.056
22	1189	197	20	177.3	18	149.12	15.14	149.12	25.00	177	30	1189	207	0.19	0.07	0.016
23	1162	211	45	189.9	40.5	163.43	34.85	163.43	34.85	190	41	1162	230	0.19	0.07	0.016
24	1925	281	48	252.9	43.2	131.38	22.44	131.38	25.00	253	48	1925	301	0.31	0.10	negative
25	1952	295	17	265.5	15.3	136.01	7.84	136.01	25.00	266	49	1952	314	0.31	0.10	negative

TABLE 27

Design Check on Trial Section of Table 26 above Joint C

Comb. No.	P_u	$\frac{P_u}{P_{uz}}$	α_n	$\frac{P_u}{f_{ck}bD}$	M_{ux}, kNm	M_{uz}, kNm	$\frac{M_{u1}}{f_{ck}bd^2}$	M_{u1}	$\left[\frac{M_{ux}}{M_{u1}}\right]^{\alpha_n}$	$\left[\frac{M_{uz}}{M_{u1}}\right]^{\alpha_n}$	Check
1	3339	0.68	1.80	0.53	118	83	0.12	375	0.124	0.067	0.191
2	2710	0.55	1.59	0.43	100	264	0.145	453	0.091	0.423	0.514
3	2687	0.55	1.58	0.43	89	214	0.145	453	0.076	0.306	0.382
4	2632	0.54	1.56	0.42	88	331	0.145	453	0.078	0.613	0.691
5	2654	0.54	1.57	0.42	99	282	0.145	453	0.092	0.474	0.566
6	2377	0.48	1.48	0.38	78	59	0.155	484	0.068	0.045	0.113
7	2355	0.48	1.47	0.38	88	59	0.155	484	0.082	0.045	0.127
8	2965	0.60	1.68	0.47	266	74	0.13	406	0.493	0.058	0.551
9	2987	0.61	1.68	0.48	276	75	0.13	406	0.523	0.058	0.581
10	2643	0.54	1.57	0.42	70	350	0.145	453	0.054	0.668	0.722
11	2616	0.53	1.56	0.42	65	289	0.14	438	0.052	0.524	0.576
12	2547	0.52	1.53	0.41	64	393	0.14	438	0.052	0.849	0.901
13	2548	0.52	1.53	0.41	69	331	0.14	438	0.059	0.653	0.712
14	2228	0.45	1.42	0.36	152	56	0.17	531	0.168	0.040	0.209
15	2201	0.45	1.42	0.35	165	55	0.17	531	0.191	0.040	0.231
16	2963	0.60	1.67	0.47	279	74	0.13	406	0.533	0.058	0.591
17	2990	0.61	1.68	0.48	292	75	0.13	406	0.572	0.058	0.630
18	1605	0.33	1.21	0.26	45	359	0.17	531	0.050	0.622	0.672
19	1577	0.32	1.20	0.25	39	297	0.17	531	0.044	0.497	0.541
20	1509	0.31	1.18	0.24	38	384	0.17	531	0.044	0.682	0.727
21	1537	0.31	1.19	0.25	44	322	0.17	531	0.052	0.552	0.603
22	1189	0.24	1.07	0.19	177	30	0.18	563	0.290	0.043	0.333
23	1162	0.24	1.06	0.19	190	41	0.18	563	0.316	0.061	0.377
24	1925	0.39	1.32	0.31	253	48	0.17	531	0.375	0.042	0.417
25	1952	0.40	1.33	0.31	266	49	0.17	531	0.397	0.042	0.439

1.11.3. Design of Transverse reinforcement

Three types of transverse reinforcement (hoops or ties) will be used. These are:

i) General hoops: These are designed for shear as per recommendations of IS 456:2000 and IS 13920:1993.

ii) Special confining hoops, as per IS 13920:1993 with spacing smaller than that of the general hoops

iii) Hoops at lap: Column bars shall be lapped only in central half portion of the column. Hoops with reduced spacing as per IS 13920:1993 shall be used at regions of lap splicing.

1.11.3.1. Design of general hoops

(A) Diameter and no. of legs

Rectangular hoops may be used in rectangular column. Here, rectangular hoops of 8 mm diameter are used.

Here h = 500 – 2 x 40 + 8 (using 8# ties)

= 428 mm > 300 mm (Clause 7.3.1, IS 13920:1993)

The spacing of bars is (395/4) = 98.75 mm, which is more than 75 mm. Thus crossties on all bars are required

(IS 456:2000, Clause 26.5.3.2.b-1)

Provide 3 no open crossties along X and 3 no open crossties along Z direction. Then total legs of stirrups (hoops) in any direction = 2 +3 = 5.

(B) Spacing of hoops

As per IS 456:2000, Clause 26.5.3.2.(c), the pitch of ties shall not exceed:

(i) b of the column = 500 mm

(ii) 16 ϕ_{min} (smallest diameter) = 16 x 20

= 320 mm

(iii) 300 mm …. **(1)**

The spacing of hoops is also checked in terms of maximum permissible spacing of shear reinforcement given in IS 456:2000, Clause 26.5.1.5

b x d = 500 x 450 mm. Using 8# hoops,

A_{sv} = 5 x 50 = 250 mm^2.

The spacing should not exceed

(i) $\dfrac{0.87\ f_y\ A_{SV}}{0.4b}$ (requirement for minimum shear reinforcement)

$$= \frac{0.87 \times 415 \times 250}{0.4 \times 500} = 451.3 \text{ mm}$$

(ii) 0.75 d = 0.75 X 450 = 337.5 mm

(iii) 300 mm; i.e., 300 mm … **(2)**

As per IS 13920:1993, Clause 7.3.3,

Spacing of hoops $\leq b/2$ of column

= 500 / 2 = 250 mm … **(3)**

From **(1), (2)** and **(3)**, maximum spacing of stirrups is 250 mm c/c.

1.11.3.2. Design Shear

As per IS 13920:1993, Clause 7.3.4, design shear for columns shall be greater of the followings:

(a) Design shear as obtained from analysis

For C_{202}, lower height, V_u = 161.2 kN, for load combination 12.

For C_{202}, upper height, V_u = 170.0 kN, for load combination 12.

(b) $V_u = 1.4 \left[\dfrac{M^{bL}_{u,\lim} + M^{bR}_{u,\lim}}{h_{st}} \right]$.

For C202, lower height, using sections of B2001 and B2002

$M^{bL}_{u,\lim}$ = 568 kNm (Table 18)

$M^{bR}_{u,\lim}$ = 568 kNm, (Table 18)

h_{st} = 4.1 m.

Hence,

$$V_u = 1.4 \left[\frac{M^{bL}_{u,\lim} + M^{bR}_{u,\lim}}{h_{st}} \right] = 1.4 \left[\frac{568 + 568}{4.1} \right]$$

= 387.9 kN say 390 kN.

For C202, upper height, assuming same design as sections of B2001 and B2002

$M^{bL}_{u,\lim}$ (Table 18) = 585 kNm

$M^{bR}_{u,\lim}$ (Table 18) = 585 kNm, and

h_{st} = 5.0 m.

Then

$$V_u = 1.4\left[\frac{M^{bL}_{u,lim} + M^{bR}_{u,lim}}{h_{st}}\right]$$

$$= 1.4\left[\frac{585+585}{5.0}\right] = 327.6 \text{ kN.}$$

Design shear is maximum of (a) and (b).

Then, design shear V_u = 390 kN. Consider the column as a doubly reinforced beam, b = 500 mm and d = 450 mm.

$A_s = 0.5\, A_{sc}$ = 0.5 x 6 968 = 3 484 mm^2.

For load combination 12, P_u = 3,027 kN for lower length and P_u = 2,547 kN for upper length.

Then,

$$\delta = 1 + \frac{3P_u}{A_g f_{ck}} \qquad \text{(IS456: 2000, Clause 40.2.2)}$$

$$= 1 + \frac{3 \times 3027 \times 1000}{500 \times 500 \times 25} = 2.45, \text{ for lower length, and}$$

$$= 1 + \frac{3 \times 2547 \times 1000}{500 \times 500 \times 25} = 2.22, \text{ for upper length.}$$

$$\leq 1.5$$

$Take\ \delta = 1.5.$

$$\frac{100A_s}{bd} = \frac{100 \times 3484}{500 \times 450} = 1.58$$

$$\tau_c = 0.753 \text{ N/mm}^2 \text{ and } \delta\tau_c = 1.5 \times 0.753 = 1.13 \text{ N/mm}^2$$

$$V_{uc} = \delta\tau_c \text{bd} = 1.13 \times 500 \times 450 \times 10^{-3} = 254.5 \text{ kN}$$

$$V_{us} = 390 - 254.5 = 135.5 \text{ kN}$$

$$A_{sv} = 250 \text{mm}^2, \text{using 8 mm \# 5 legged stirrups.}$$

Then

$$s_v = \frac{0.87 f_y A_{sv} d}{V_{us}} = \frac{0.87 \times 415 \times 250 \times 450}{135.5 \times 1000} = 299.8 \text{ mm}$$

Use 200 mm spacing for general ties.

1.11.3.3. *Design of Special Confining Hoops:*

As per Clause 7.4.1 of IS 13920:1993, special confining reinforcement shall be provided over a length ℓ_0, where flexural yielding may occur.

ℓ_0 shall not be less than

(i) D of member, i.e., 500 mm

(ii) $\frac{L_c}{6}$,

i.e., $\frac{(4100 - 600)}{6}$ = 583 mm for column C202

and, $\frac{(5000 - 600)}{6}$ =733 mm for column C302.

Provide confining reinforcement over a length of 600 mm in C202 and 800 mm in C302 from top and bottom ends of the column towards mid height.

As per Clause 7.4.2 of IS 13920:1993, special confining reinforcement shall extend for minimum 300 mm into the footing. It is extended for 300 mm as shown in Figure 12.

As per Clause 7.4.6 of IS 13920:1993, the spacing, s, of special confining reinforcement is governed by:

$$s \leq 0.25\, D = 0.25 \times 500 = 125 \text{ mm} \geq 75 \text{ mm}$$

$$\leq 100 \text{mm}$$

i.e. Spacing = 75 mm to 100 mm c/c...… (1)

As per Clause 7.4.8 of IS 13920:1993, the area of special confining reinforcement, A_{sh}, is given by:

$$A_{sh} = 0.18\, s \leq h\, \frac{f_{ck}}{f_y}\left[\frac{A_g}{A_k} - 1.0\right]$$

Here average h referring to fig 12 is

$$h = \frac{100 + 130 + 98 + 100}{4} = 107 \text{ mm}$$

A_{sh} = 50.26 mm^2

A_k = 428 mm x 428 mm

$$50.26 = 0.18 \times s \times 107 \times \frac{25}{415}\left[\frac{500 \times 500}{428 \times 428} - 1\right]$$

50.26 = 0.4232 s

s = 118.7 mm

≤ 100 mm … … (2)

Provide 8 mm # 5 legged confining hoops in both the directions @ 100 mm c/c.

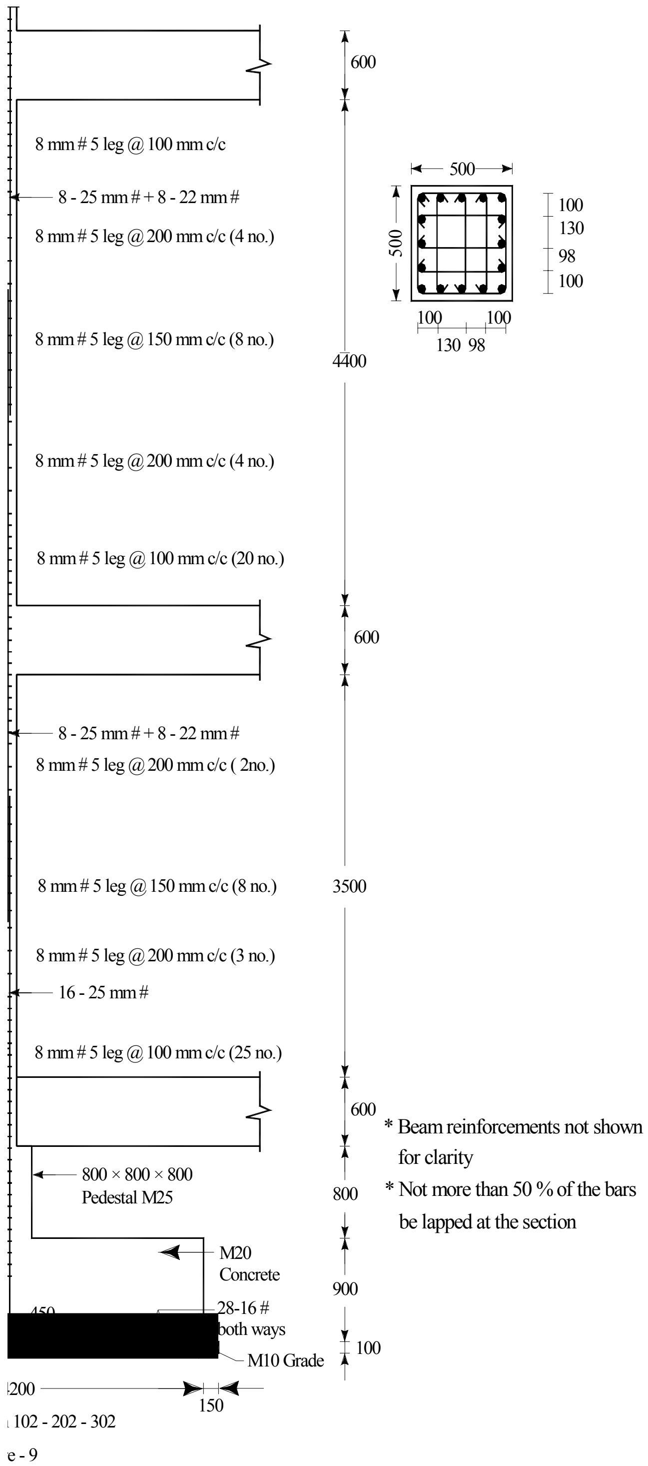


Figure 12 Reinforcement Details

1.11.3.4. Design of hoops at lap

As per Clause 7.2.1 of IS 13920:1993, hoops shall be provided over the entire splice length at a spacing not exceeding 150 mm centres

Moreover, not more than 50 percent of the bars shall be spliced at any one section.

Splice length = L_d in tension = 40.3 d_b.

Consider splicing the bars at the centre (central half) of column 302.

Splice length = 40.3 x 25 = 1008 mm, say 1100 mm. For splice length of 40.3 d_b, the spacing of hoops is reduced to 150 mm. Refer to Figure 12.

1.11.3.5. Column Details

The designed column lengths are detailed in Figure 12. Columns below plinth require smaller areas of reinforcement; however, the bars that are designed in ground floor (storey 1) are extended below plinth and into the footings. While detailing the shear reinforcements, the lengths of the columns for which these hoops are provided, are slightly altered to provide the exact number of hoops. Footings also may be cast in M25 grade concrete.

1.12. Design of footing: (M20 Concrete):

It can be observed from table 24 and table 26 that load combinations 1 and 12 are governing for the design of column. These are now tried for the design of footings also. The footings are subjected to biaxial moments due to dead and live loads and uniaxial moment due to earthquake loads. While the combinations are considered, the footing is subjected to biaxial moments. Since this building is very symmetrical, moment about minor axis is just negligible. However, the design calculations are performed for biaxial moment case. An isolated pad footing is designed for column C2.

Since there is no limit state method for soil design, the characteristic loads will be considered for soil design. These loads are taken from the computer output of the example building. Assume thickness of the footing pad D = 900 mm.

(a) *Size of footing*:

Case 1:

Combination 1, i.e., (DL + LL)

P = (2291 + 608) = 2899 kN

H_x = 12 kN, H_z = 16 kN

M_x = 12 kNm, M_z = 6 kNm.

At the base of the footing

P = 2899 kN

P' = 2899 + 435 (self-weight) = 3334 kN,

assuming self-weight of footing to be 15% of the column axial loads (DL + LL).

$M_{x1} = M_x + H_y \times D$

$= 12 + 16 \times 0.9 = 26.4$ kNm

$M_{z1} = M_z + H_y \times D$

$= 6 + 12 \times 0.9 = 18.8$ kNm.

For the square column, the square footing shall be adopted. Consider 4.2 m × 4.2 m size.

$A = 4.2 \times 4.2 = 17.64$ m^2

$Z = \frac{1}{6} \times 4.2 \times 4.2^2 = 12.348$ m^3.

$$\frac{P}{A} = \frac{3344}{17.64} = 189 \text{ kN/m}^2$$

$$\frac{M_{x1}}{Z_x} = \frac{26.4}{12.348} = 2.14 \text{ kN/m}^2$$

$$\frac{M_{z1}}{Z_z} = \frac{18.8}{12.348} = 1.52 \text{ kN/m}^2$$

Maximum soil pressure

= 189 + 2.14 + 1.52

= 192.66 kN/m^2 < 200 kN/m^2

Minimum soil pressure

= 189 – 2.14 – 1.52

= 185.34 kN/m^2 > 0 kN/m^2.

Case 2:

Combination 12, i.e., (DL - EXTP)

Permissible soil pressure is increased by 25%.

i.e., allowable bearing pressure = 200 $\times 1.25$

= 250 kN/m^2.

P = (2291 - 44) = 2247 kN

H_x = 92 kN, H_z = 13 kN

M_x = 3 kNm, M_z = 216 kNm.

At the base of the footing

$P = 2247$ kN

$P' = 2247 + 435$ (self-weight) $= 2682$ kN.

$M_{x1} = M_x + H_y \times D$

$= 3 + 13 \times 0.9 = 14.7$ kNm

$M_{z1} = M_z + H_y \times D$

$= 216 + 92 \times 0.9 = 298.8$ kNm.

$$\frac{P'}{A} = \frac{2682}{17.64} = 152.04 \text{ kN/m}^2$$

$$\frac{M_{x1}}{Z_x} = \frac{14.7}{12.348} = 1.19 \text{ kN/m}^2$$

$$\frac{M_{z1}}{Z_z} = \frac{298.8}{12.348} = 24.20 \text{ kN/m}^2$$

Maximum soil pressure

$= 152.04 + 1.19 + 24.2$

$= 177.43$ kN/m^2 $<$ 250 kN/m^2.

Minimum soil pressure

$= 152.04 - 1.19 - 24.2$

$= 126.65$ kN/m^2 $>$ 0 kN/m^2.

Case 1 governs.

In fact all combinations may be checked for maximum and minimum pressures and design the footing for the worst combination.

Design the footing for combination 1, i.e., DL + LL.

$$\frac{P}{A} = \frac{2899}{17.64} = 164.34 \text{ kN/mm}^2$$

Factored upward pressures for design of the footing with biaxial moment are as follows.

For M_x

$p_{up} = 164.34 + 2.14 = 166.48$ kN/m^2

$p_{u,up} = 1.5 \times 166.48 = 249.72$ kN/m^2

For M_z

$p_{up} = 164.34 + 1.52 = 165.86$ kN/m^2

$p_{u,up} = 1.5 \times 165.86 = 248.8$ kN/m^2

Since there is no much difference in the values, the footing shall be designed for M_z for an upward pressure of 250 kN/m^2 on one edge and 167 kN/m^2 on the opposite edge of the footing.

The same design will be followed for the other direction also.

Net upward forces acting on the footing are shown in fig. 13.

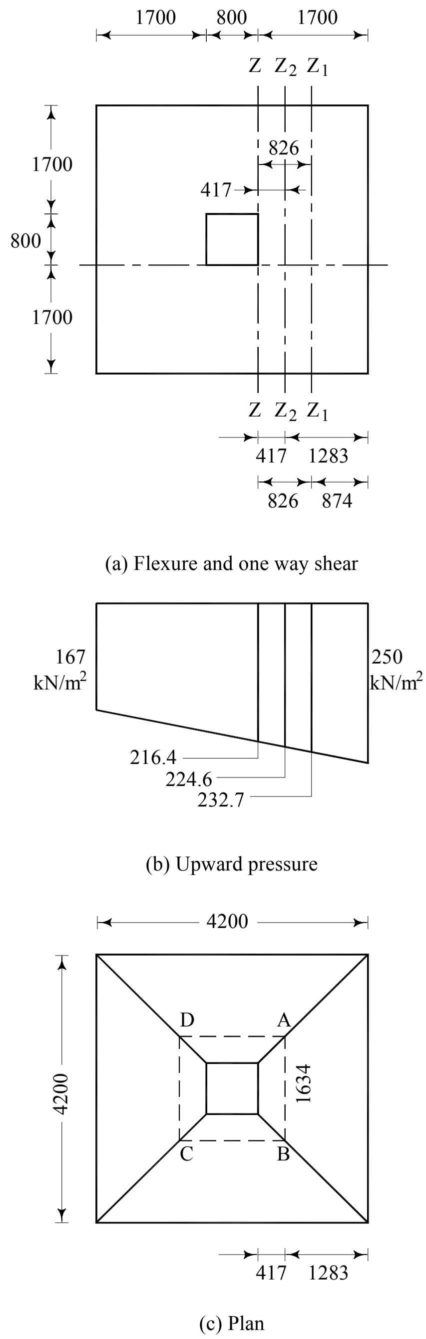


Figure 13

(b) *Size of pedestal*:

A pedestal of size 800 mm × 800 mm is used.

For a pedestal

$A = 800 \times 800 = 640000 \text{ mm}^2$

$$\boldsymbol{Z} = \frac{1}{6} \times 800 \times 800^2 = 85333333 \text{ mm}^3$$

For case 1

$$q_{01} = \frac{2899 \times 1000}{800 \times 800} + \frac{(26.4 + 18.8) \times 10^6}{85333333}$$

$= 4.53 + 0.53 = 5.06 \text{ N/mm}^2$ … (1)

For case 2

$$q_{02} = \frac{2247 \times 1000}{800 \times 800} + \frac{(14.7 + 298.8) \times 10^6}{85333333}$$

$= 3.51 + 3.67 = 7.18 \text{ N/mm}^2$

Since 33.33 % increase in stresses is permitted due to the presence of EQ loads, equivalent stress due to DL + LL is

$7.18 \div 1.33 = 5.4 \text{ N/mm}^2$. … (2)

From (1) and (2) consider $q_0 = 5.4 \text{ N/mm}^2$.

For the pedestal

$$\tan\alpha \geq 0.9\sqrt{\frac{100 \times 5.4}{20} + 1}$$

This gives

$\tan\alpha \geq 4.762$, i.e., $\alpha \geq 78.14^0$

Projection of the pedestal = 150 mm

Depth of pedestal = 150 × 4.762 = 714.3 mm.

Provide 800 mm deep pedestal.

(c) *Moment steel*:

Net cantilever on x-x or z-z

= 0.5(4.2-0.8) = 1.7 m.

Refer to fig. 13.

$$M_{uz} = \left[\frac{1}{2} \times 216.4 \times 1.7 \times \frac{1}{3} \times 1.7 + \frac{1}{2} \times 250 \times 1.7 \times \frac{2}{3} \times 1.7\right] \times 4.2$$

= 1449 kNm

For the pad footing, width b = 4200 mm

For M20 grade concrete, $Q_{bal} = 2.76$.

Balanced depth required

$$= \sqrt{\frac{1449 \times 10^6}{2.76 \times 4200}} = 354 \text{ mm}$$

Try a depth of 900 mm overall. Larger depth may be required for shear design. Assume 16 mm diameter bars.

$d_x = 900 - 50 - 8 = 842$ mm

$d_z = 842 - 16 = 826$ mm.

Average depth = 0.5(842+826) = 834 mm.

Design for z direction.

$$\frac{M_{uz}}{bd^2} = \frac{1449 \times 10^6}{4200 \times 826 \times 826} = 0.506$$

$p_t = 0.145$, from table 2, SP :16

$$A_{st} = \frac{0.145}{100} \times 4200 \times 900 = 5481 \text{ mm}^2$$

$$A_{st,\min} = \frac{0.12}{100} \times 4200 \times 900 = 4536 \text{ mm}^2$$

(Clause 34.5, IS: 456)

Provide 28 no. 16 mm diameter bars.

$A_{st} = 5628 \text{ mm}^2$.

$$\text{Spacing} = \frac{4200 - 100 - 16}{27} = 151.26 \text{ mm}$$

$< 3 \times 826$ mm …… …. (o.k.)

(d) *Development length*:

HYSD bars are provided without anchorage.

Development length = 47 × 16 = 752 mm

Anchorage length available

= 1700 – 50 (cover) = 1650 mm … (o.k.)

(e) *One-way shear:*

About z_1-z_1

At d = 826 mm from the face of the pedestal

$$V_u = 0.874 \times \frac{232.7 + 250}{2} \times 4.2 = 886 \text{ kN}$$

b = 4200 mm, d = 826 mm

$$\tau_v = \frac{V_u}{bd} = \frac{886 \times 1000}{4200 \times 826} = 0.255 \text{ N/mm}^2$$

$$\frac{100A_{st}}{bd} = \frac{100 \times 5628}{4200 \times 826} = 0.162$$

$\tau_c = 0.289$ N/mm2

$\tau_v < \tau_c$ … … … (o.k.)

(f) *Two-way shear*:

This is checked at $d/2$, where d is an average depth, i.e., at 417 mm from the face of the pedestal. Refer to fig. 13 (c).

Width of punching square

$= 800 + 2\times417 = 1634$ mm.

Two-way shear along linr *AB*

$$= \left(\frac{224.6+250}{2}\right)\left(\frac{1.634+4.2}{2}\right)\times 1.283 \;=\; 883\,\text{kN}.$$

$$\tau_v = \frac{V_u}{bd} = \frac{883\times1000}{1634\times834} = 0.648\ \text{N/mm}^2$$

Design shear strength = $k_s \tau_c$, where

$k_s = 0.5 + \tau_c$ and $\tau_c = (b_c/\ell_c) = 500/500 = 1$

$k_s = 0.5 + 1 = 1.5 \le 1$, i.e., $k_s = 1$

Also,

$$\tau_c = 0.25\sqrt{f_{ck}} = 0.25\sqrt{20} = 1.118\ \text{N/mm}^2$$

Then $k_s \tau_c = 1.118 = 1.118$ N/mm^2.

Here $\tau_v < \tau_c$ … … … (o.k.)`

(g) *Transfer of load from pedestal to footing*:

Design bearing pressure at the base of pedestal

$$= 0.45 f_{ck} = 0.45\times25 = 11.25\ \ \text{N/mm}^2$$

Design bearing pressure at the top of the footing

$$= \sqrt{\frac{A_1}{A_2}}\times 0.45 f_{ck} = 2\times0.45\times20 = 18\ \text{N/mm}^2$$

Thus design bearing pressure = 11.25 N/mm^2.

Actual bearing pressure for case 1

$= 1.5 \times q_{01} = 1.5 \times 5.06 = 7.59$ N/mm^2.

Actual bearing pressure for case 2

.

$= 1.2 \times q_{02} = 1.2 \times 7.18 = 8.62$ N/mm^2.

Thus dowels are not required.

Minimum dowel area = (0.5/100) × 800 × 800

= 3200 mm^2.

Area of column bars = 7856 mm^2

It is usual to take all the bars in the footing to act as dowel bars in such cases.

Minimum Length of dowels in column = L_d of column bars

= 28 × 25 = 700 mm.

Length of dowels in pedestal = 800 mm.

Length of dowels in footing

= D + 450 = 900 + 450 = 1350 mm.

This includes bend and ell of the bars at the end.

The Dowels are lapped with column bars in central half length of columns in ground floors. Here the bars are lapped at mid height of the column width 1100 mm lapped length.

Total length of dowel (Refer to fig. 12)

= 1350 + 800 + 600 + 1750 + 550

= 5050 mm.

Note that 1100 mm lap is given about the mid-height of the column.

(h) Weight of the footing:

= 4.2 × 4.2 × 0.9 × 25 = 396.9 kN

< 435 kN, assumed.

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