Detailed Design of a Six StoreyBuilding Near Bhuj.(Gujrat)

NATIONAL INSTITUTE OF TECHNOLOGY
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DESIGN OF EARTH-QUAKE RESISTANT

SIX-STORIED BUILDING LOCATED AT BHUJ

A THESIS SUBMITTED IN PARTIAL FULFILLMENT
OF THE REQUIREMENTS FOR THE DEGREE OF

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CERTIFICATE

This is to certify that the thesis entitled, Design of Earth-Quake Resistant Six-storied Building located at Bhuj submitted by Sri Bijay Kumar Behera in partial fulfillment of the requirements for the award of Bachelor of technology in Civil Engineering with specialization in Structural Engineering at the National Institute of Technology, Rourkela (Deemed Unversity) is an authentic work carried out by him under my supervision and guidance.

To the best of my knowledge, the matter embodied in the thesis has not been submitted to any other Unversity/Institute for the award of any Degree or Diploma.

Date:

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ACKNOWLEDGEMENT

I would like to express my sincere gratitude to Dr. K.C .Biswal for his invaluable guidance. His continuous encouragement and support has always been an inspiration and a source of energy for me. He has not only guided in just technical matters but has always taught several important points to gain maturity to work and his nature of thinking has always influence me in many ways. I thank him for all of his efforts and help. I gratefully acknowledged Dr. K.C. Patra of Civil Engg. Department,NIT, Rourkela. for many good suggestions during the progress of project work.

CONTENTS

Abstract

Abstract

Seismic design procedures have historically been developed by engineers from observations of the behavior of real buildings and structures when subjected to destructive earthquakes. Prescriptive requirements, based on features observed to result in good performance, were developed after each damaging earthquake. This knowledge was supplemented by a systematic process of improving our basic understanding of earthquakes and our ability to characterize and predict the effects of earthquakes and the response of structures. The ordinary objective of seismic stabilization of historic buildings in earthquake regions has always been the protection of life safety and prevention of collapse of the structure. Recent years have seen the introduction of many new concepts and technological advances in the field of seismic design, including state of the art methods of ground motion characterization and zonation, and direct consideration of non-**linear structural changes.**

INTRODUCTION

Seismology is the study of earth vibrations mainly caused by earthquakes. The study of these vibrations by various techniques, understanding the nature and various physical processes that generate them from the major part of the seismology.

A seismic design of high rise buildings has assumed considerable importance in recent times. In traditional methods adopted based on fundamental mode of the structure and distribution of earthquake forces as static forces at various stories may be adequate for structures of small height subjected to earthquake of very low intensity but as the number of stories increases the seismic design demands more rigor.

CHAPTER-1

PROBLEM STATEMENT

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 V 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.

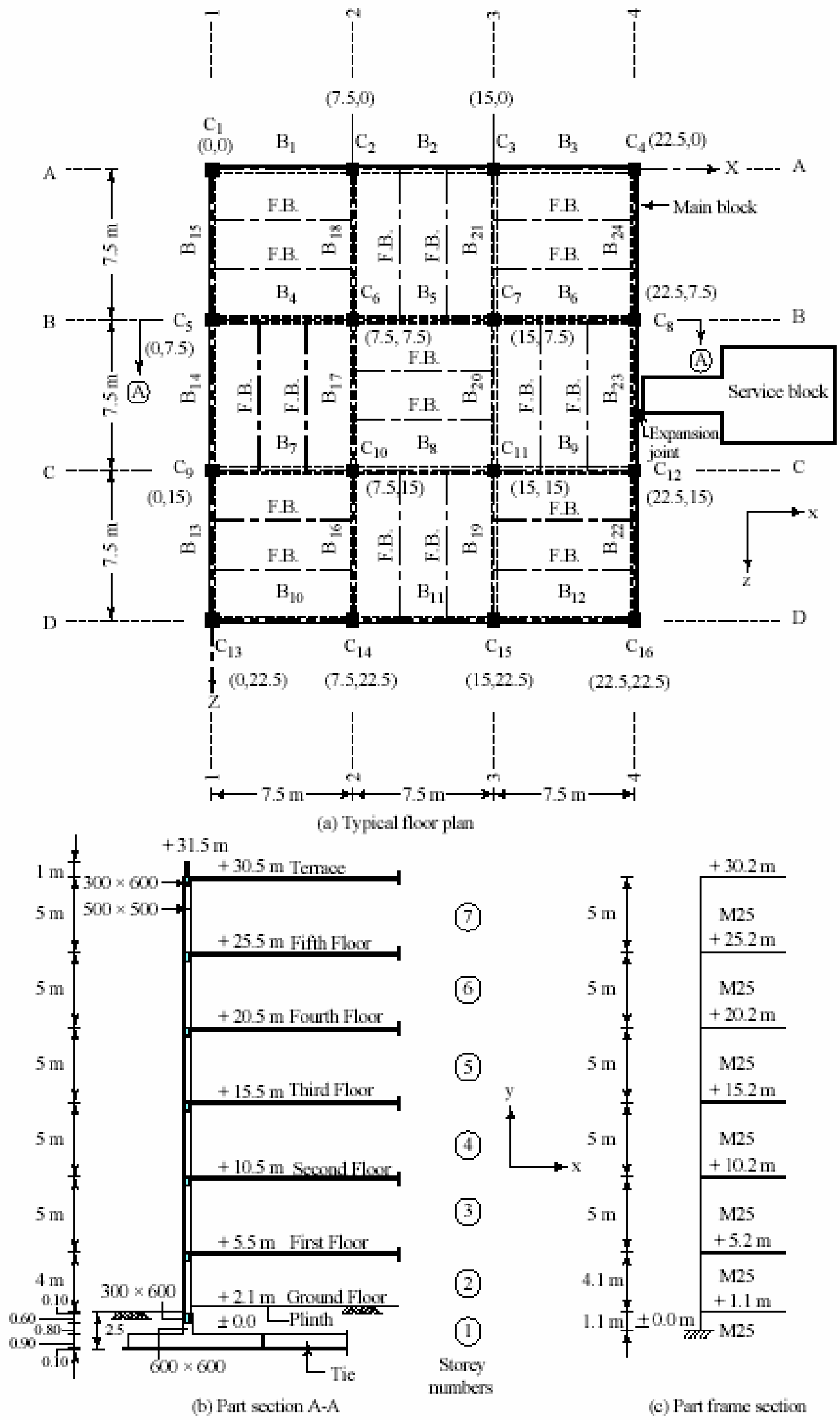


Figure 1 General lay-out of the Building.

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	:	BHUJ 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/m2
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

Ec = 5 000 $\sqrt{ck}$ N/mm2 = 5 000 $\sqrt{ck}$ MN/m2 = 25 000 N/mm2 = 25 000 MN/m2 .

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

Ec = 5 000 $\sqrt{ck}$ N/mm2 = 5 000 $\sqrt{ck}$ MN/m2 = 27 386 N/mm2 = 27 386 MN/m2 .

Steel

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

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.

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

Column number

In the general plan of Figure 1, the columns from C1 to C16 are numbered in a convenient way from left to right and from upper to the lower part of the plan. Column C5 is known as column C5 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 C5. The columns may also be specified by using grid lines.

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.

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 B1 to B12 in a convenient way from left to right and from upper to the lower part of the plan. Giving 90o clockwise rotation to the plan similarly marks the beams in the perpendicular direction. To floor-wise differentiate beams similar in plan (say beam B5 connecting columns C6 and C7) 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 4th storey whose number is B7 as per the general layout.

CHAPTER-2

GRAVITY LOAD CALCULATION

Gravity Load calculations

Unit load calculations

Assumed sizes of beam and column sections are:

Columns: 500 x 500 at all typical floors

Area, A = 0.25 m2, I = 0.005208 m4

Columns: 600 x 600 below ground level

Area, A = 0.36 m2, I = 0.0108 m4

Main beams: 300 x 600 at all floors

Area, A = 0.18 m2, I = 0.0054 m4

Ground beams: 300 x 600

Area, A = 0.18 m2, I = 0.0054 m4

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/m2

Brick wall (230 mm thick)

0.23 x 19 (wall) +2 x 0.012 x 20 (plaster) = 4.9 kN/m2

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

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/m2	3.5 + 4.0 kN/m2

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 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 + 1.9kN/m	4.9 + 0 kN/m

Two point loads on one-third span points for beams

B2 and B11 of (61.1 + 14.3) 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 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.

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

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 + 0kN/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.

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

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

CHAPTER-3

LOADING FRAMES

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.

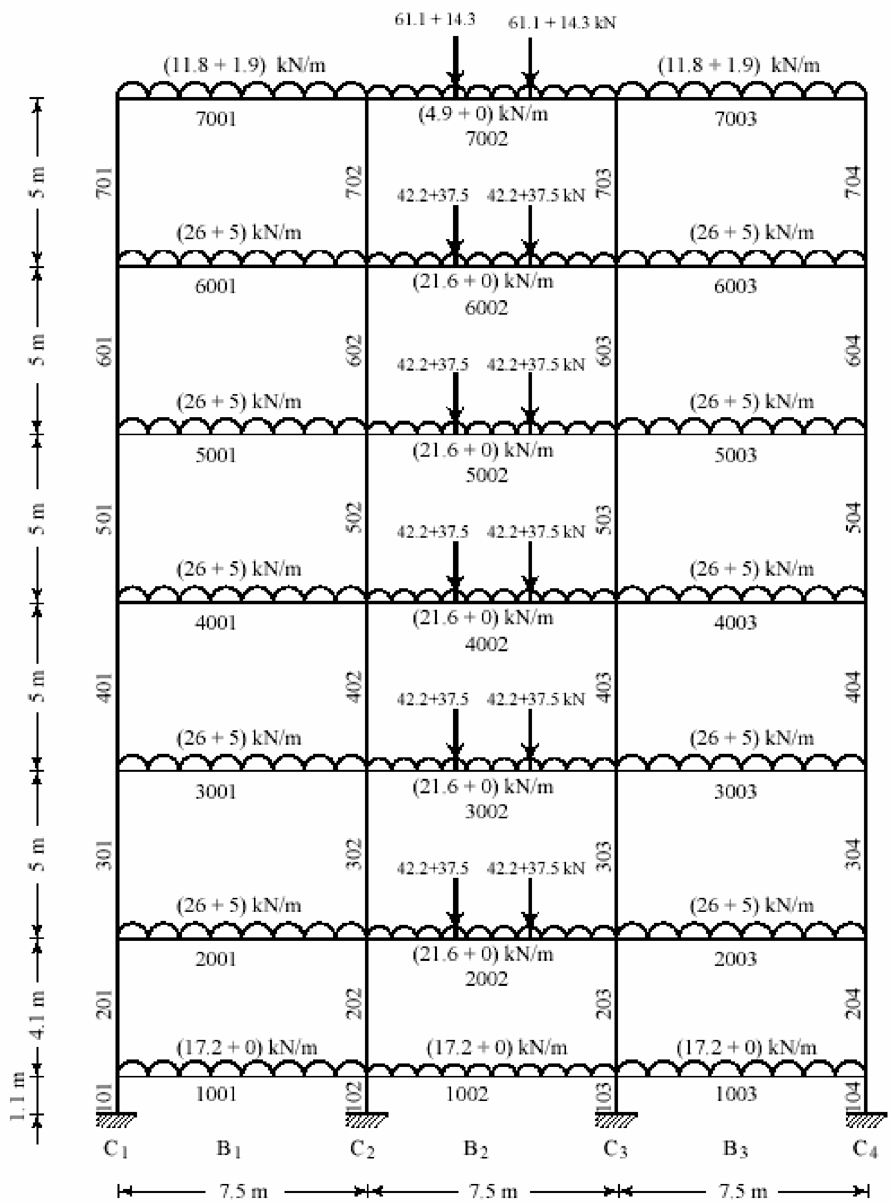


Figure 2 (a) Gravity Loads: Frame AA

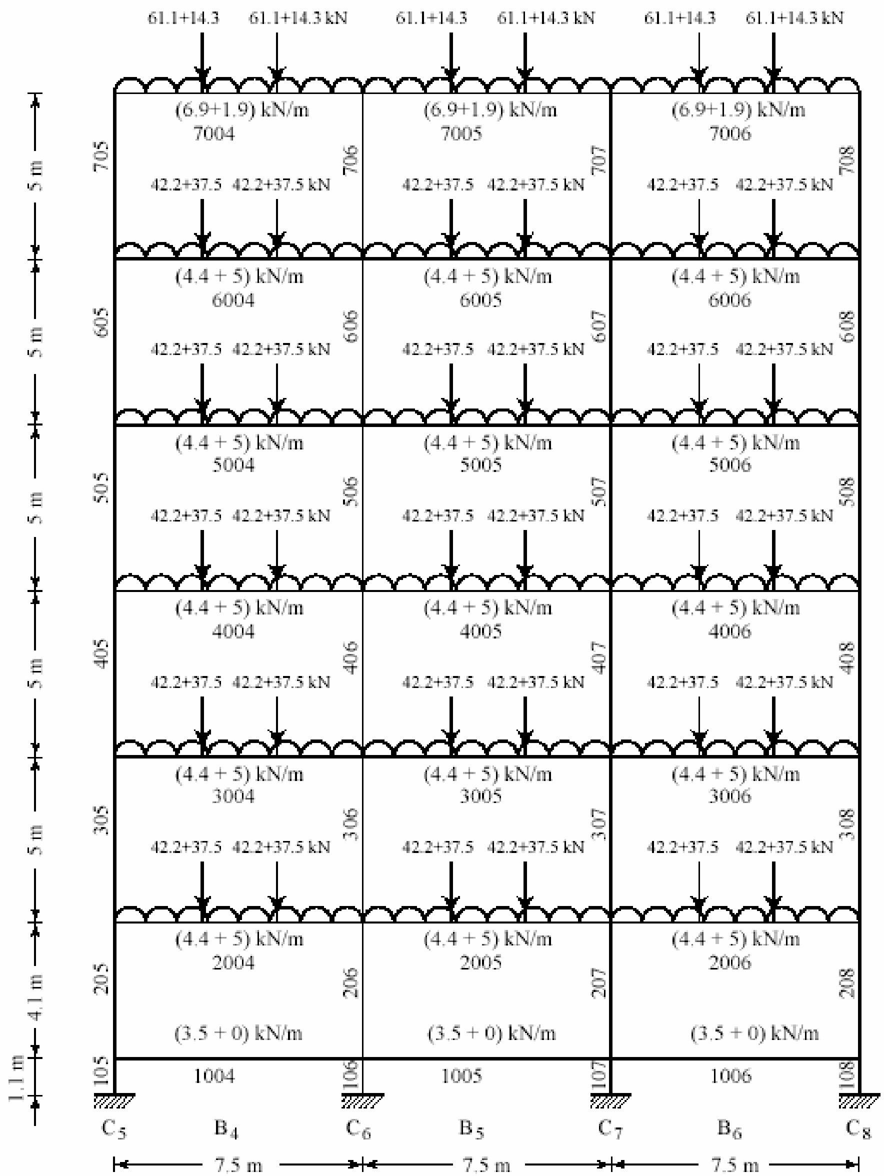


Figure 2(b) Gravity Loads: Frame BB

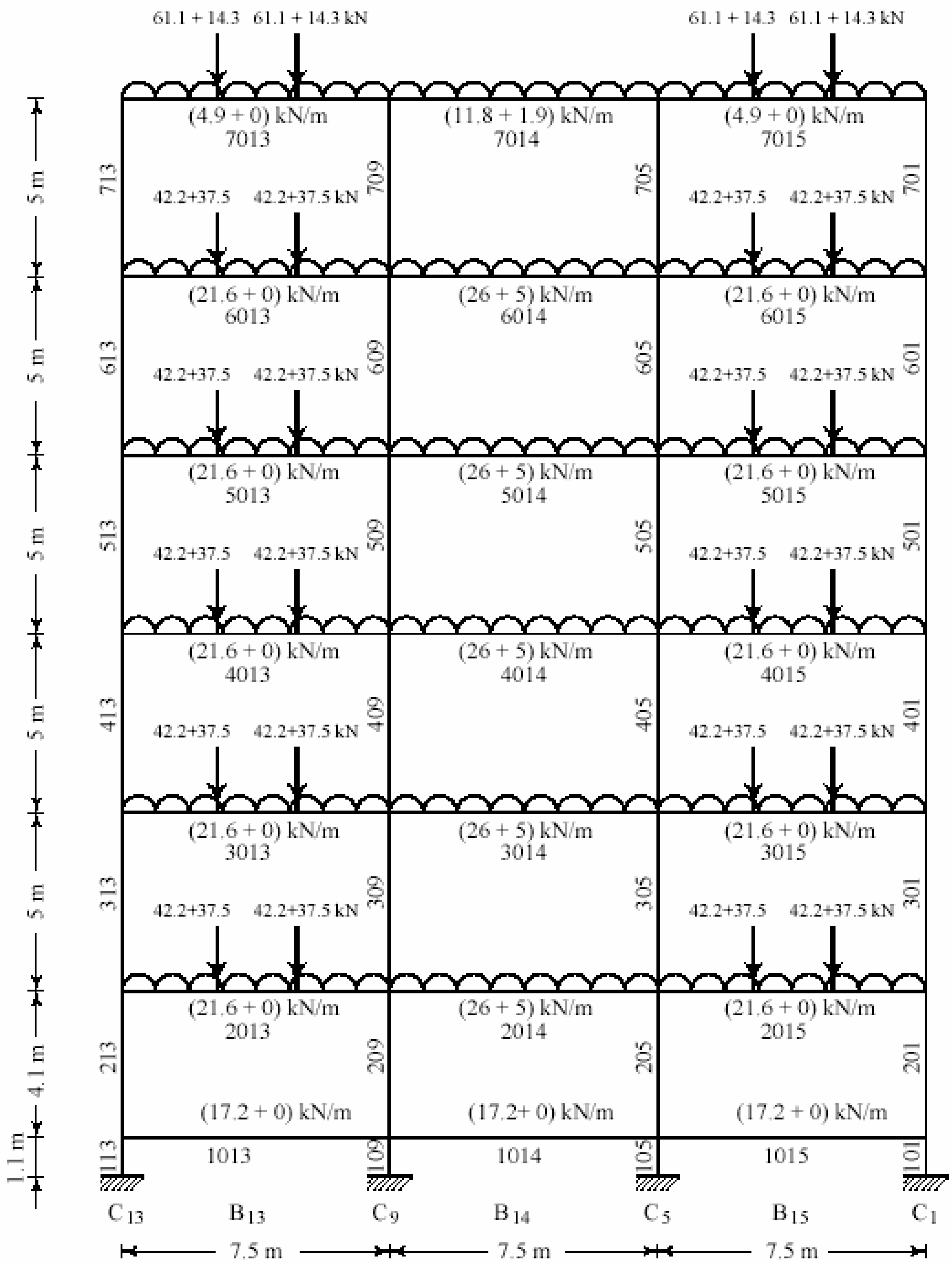


Figure 2(c) Gravity Loads: Frame 1-1

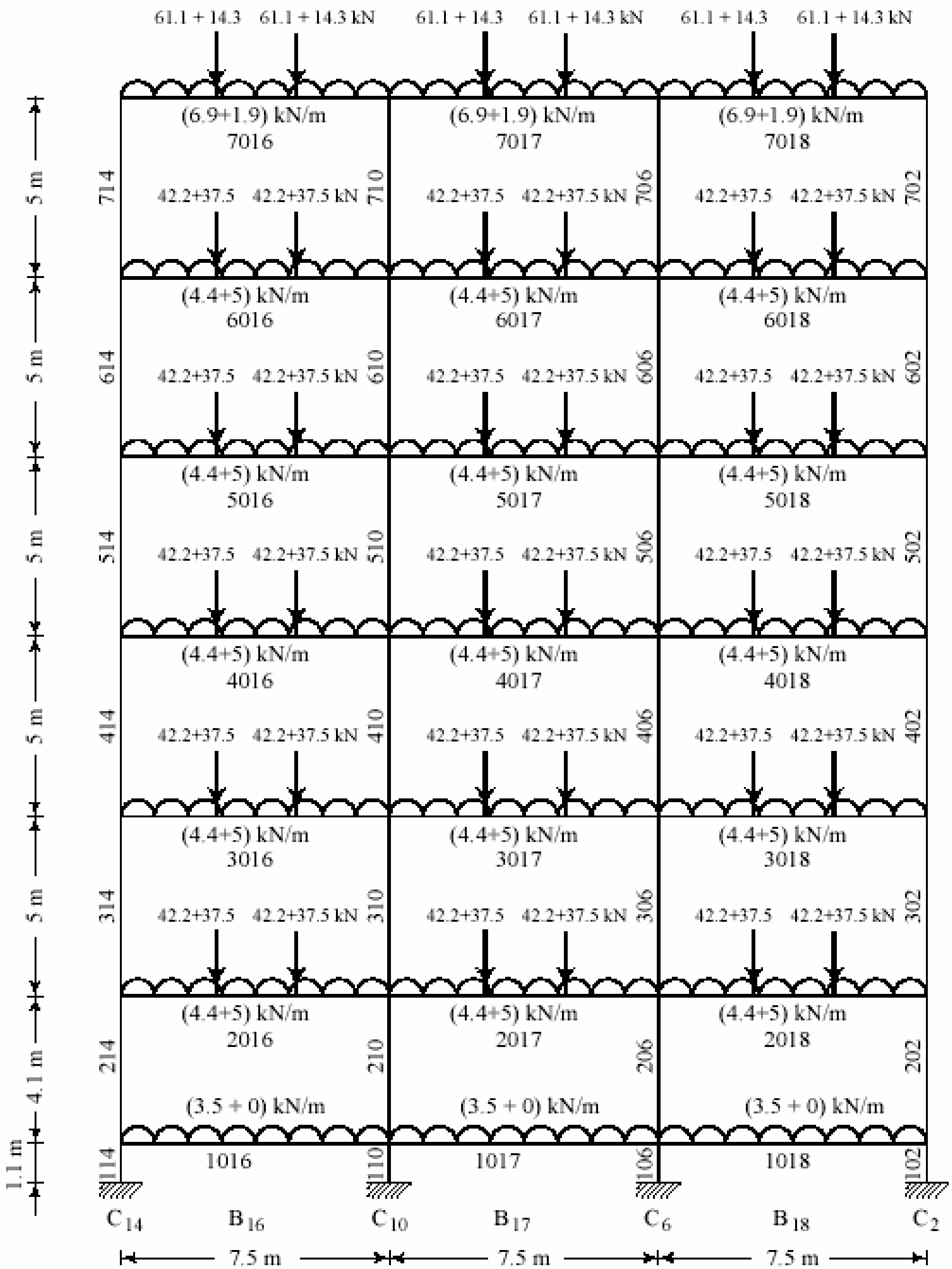
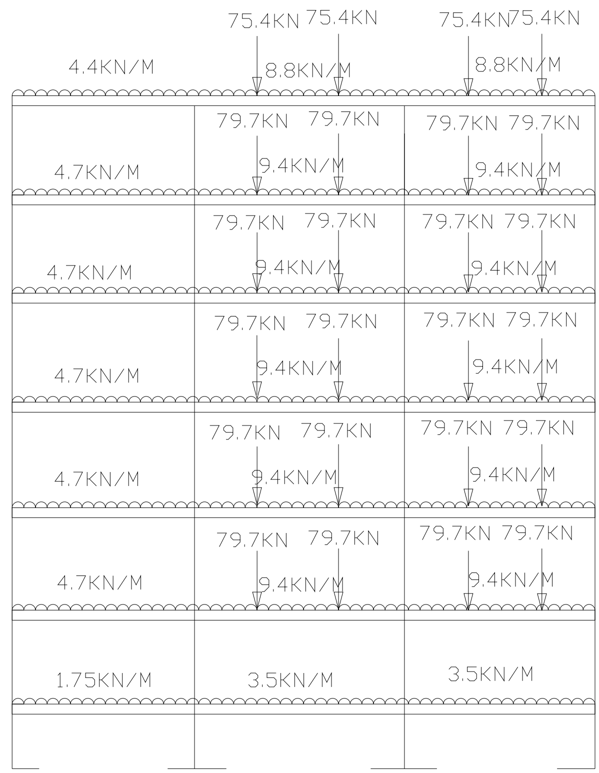


Figure 2(d) Gravity Loads: Frame 2-2


75.4KN 75.4KN 75.4KN 75.4KN
4.4KN/M
8.8KN/M
8.8KN/M
79.7KN 79.7KN 79.7KN 79.7KN
4.7KN/M
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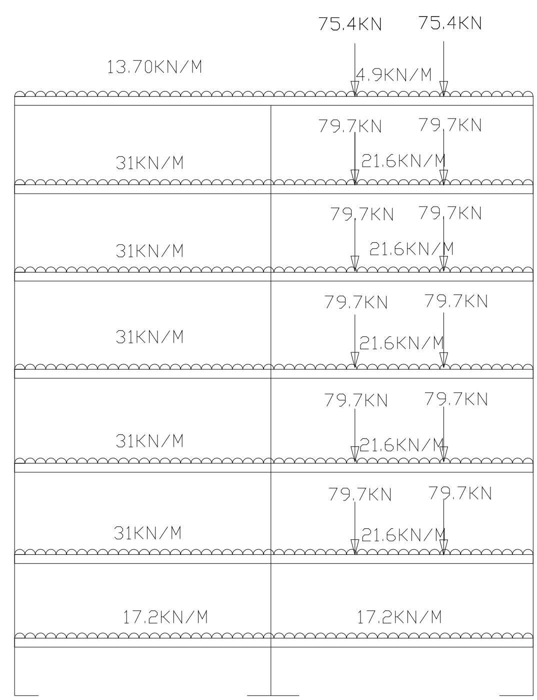
79.7KN 79.7KN 79.7KN 79.7KN
4.7KN/M
9.4KN/M
9.4KN/M
1.75KN/M
3.5KN/M
3.5KN/M

FRAME-3-3


75.4KN
75.4KN
13.70KN/M
4.9KN/M
79.7KN
79.7KN
31KN/M
21.6KN/M
79.7KN
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31KN/M
21.6KN/M
79.7KN
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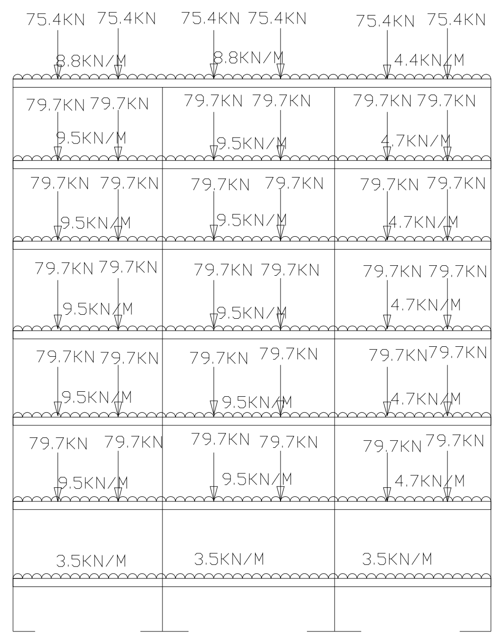
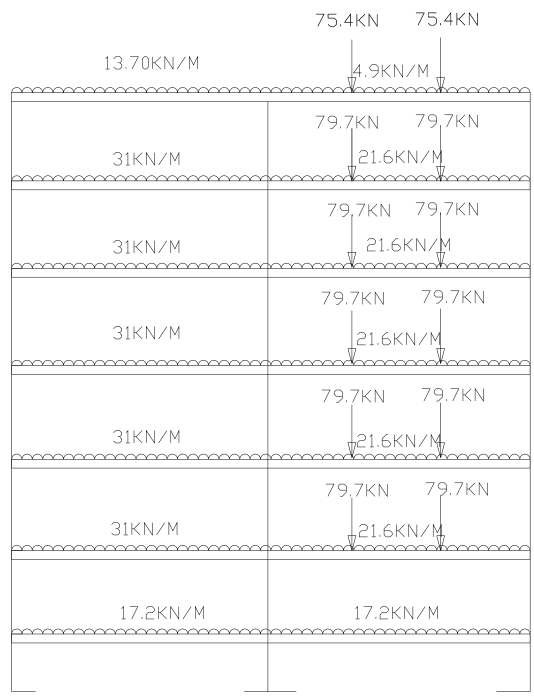
31KN/M
21.6KN/M
79.7KN
79.7KN
31KN/M
21.6KN/M
79.7KN
79.7KN
31KN/M
21.6KN/M
17.2KN/M
17.2KN/M

FRAME-4-4


75.4KN 75.4KN
75.4KN 75.4KN
75.4KN 75.4KN
8.8KN/M
8.8KN/M
4.4KN/M
79.7KN 79.7KN
9.5KN/M
4.7KN/M
3.5KN/M
3.5KN/M
3.5KN/M

FRAME-C-C



75.4KN
75.4KN
13.70KN/M
4.9KN/M
79.7KN
79.7KN
31KN/M
21.6KN/M
79.7KN
79.7KN
31KN/M
21.6KN/M
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79.7KN
31KN/M
21.6KN/M
79.7KN
79.7KN
31KN/M
21.6KN/M
17.2KN/M
17.2KN/M

FRAME-D-D

CHAPTER-4

SEISMIC WEIGHT CALCULATION

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	450x (5.5+0)	2 475 + 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	16 x 7.5 x (2.5 + 0)	300 + 0
Main beams	7.5 x 22 x (4.5 + 0)	743 + 0
Columns	0.5 x 5 x 15 x (6.3 + 0)	236 + 0
Total		5 167 + 0 = 5 167 kN

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

		DL + LL
From slab	450 x (3.5 + 0.5 x 4)	1 575+900
Walls	4 x 22. 5 x (21.6 + 0)	1 944 + 0
Secondary beams	16 x 7.5 x (2.5 + 0)	300+0
Main beams	7.5 x 22 x (4.5 + 0)	743 + 0
Columns	15 x 5 x (6.3 + 0)	473+0
Total		5035 +900

= 5935 kN

(3) Storey 2:

		DL + LL
From slab	450x (3.5 + 0.5 x 4)	1 575 +900
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	16 x 7.5 x (2.5 + 0)	300 + 0
Main beams	7.5 x 22 x (4.5 + 0)	743 + 0
Columns	15 x 0.5 x (5 + 4.1) x (6.3 + 0)	430 + 0
Total		5 794 +900
		= 5694 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	7.5 x 22 x (4.5 + 0)	743 + 0
Column	15 x 0.5 x 4.1 x (6.3 + 0)	194 + 0
	15 x 0.5 x 1.1 x (9.0 + 0)	74 + 0
Total		1943 + 0
		=1943 kN

Seismic weight of the entire building

= 5167 + 4 x 5935 + 5694 + 1943

= 36544 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.

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 x (30.5)0.75

= 0.97 sec.

Zone factor, *Z* = 0.36 for Zone V

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

Importance factor, *I* = 1.5 (public building) Medium soil site and 5% damping

Sa/g=1.36/0.97=1.402

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

Distribution of total horizontal load to different floor levels:

TABLE 1:

Storey	W_i kN	h_i m	$W_i h_i^2 \times 10^{-3}$	$Qi = W_i h_i^2 \times V_B / \sum W_i h_i^2$ kN	Vi kN
7	5167	30.2	4713	999	999
6	5935	25.2	3769	799	1798
5	5935	20.2	2422	514	2312
4	5935	15.2	1372	291	2603
3	5935	10.2	618	131	2734

2	5694	5.2	154	33	2766
1	1943	1.1	2	0	0
Total			13050		

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,

$$A_h = (Z/2) \times (I/R) \times (S_a/g)$$

$$= (0.36/2) \times (1.5/5) \times (1.402)$$

$$= 0.0757$$

$$W = 36544 \text{ kN}$$

$$V_B = 0.0757 \times 36544$$

$$= 2766 \text{kN}$$

Base shear, *V*B = *A*h *W* = 2766 kN. The total horizontal load of 2766 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.

Accidental eccentricity:

Design eccentricity is given by

$$e_{di} = 1.5\, e_{si} + 0.05\, b_i \quad \text{or}$$
$$e_{si} - 0.05\, b_i$$

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

For the present case, static eccentricity, esi=0.19m

edi=0.19+ 0.05 x 22.5 = 1.41 m.

Thus the load is eccentric by 1.41 m from mass centre. For the purpose of our calculations, eccentricity from centre of stiffness shall be calculated. 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., +1.41 *Q*i kNm or -1.41 *Q*i kNm).

Forces Qi 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.

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

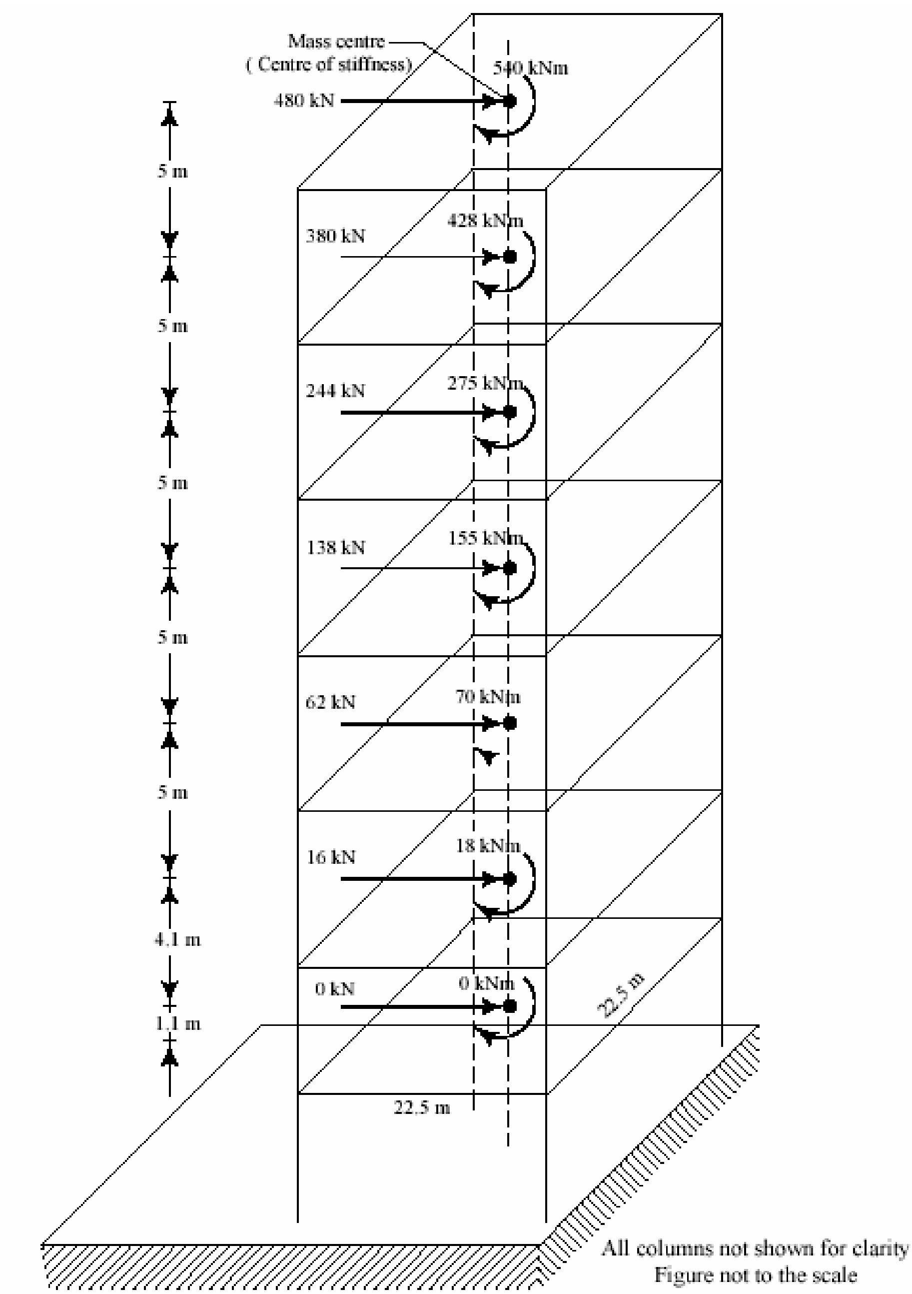


Figure 3 Accidental Eccentricity Inducing Torsion in the Building

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.

CHAPTER-5

LOAD COMBINATIONS

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. . 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, experience shows 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 obtain.

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

CHAPTER-6

STOREY DRIFT AND STABILITY INDICES

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.

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.

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 storey columns, Q = 0.

Q_{si} = stability index of i^{th} storey

$\sum P_u$ = sum of axial loads on all columns in the i^{th} 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 Qsi . 0.04, otherwise, it is a sway column. It may be noted that both sway and nonsway columns are unbraced columns. For braced columan 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

CHAPTER-7

DESIGN OF BEAM

Design of Selected Beams

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

General requirements

The flexural members shall fulfil the following general requirements.

(IS 13920; Clause 6.1.2)

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

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

Hence, ok.

(IS 13920; Clause 6.1.3)

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

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

Hence, ok.

(IS 13920; Clause 6.1.4)

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

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

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

Hence, ok.

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.

:

END MOMENTS FOR BASIC LOAD CASES:

TABLE:6

Sl No.	LOAD CASE.	B2001		B2002		B2003	
		LEFT.	RIGHT.	LEFT.	RIGHT.	LEFT.	RIGHT.
1.	DL.	127.95	-177.80	189.96	-198.05	150.95	-127.95
2.	IL/LL.	20.18	-30.85	59.81	-59.81	31.85	-20.18
3.	EXTP.	-265.96	-243.66	-217.41	-217.4	-238.9	-263.90
4.	EXTN.	-218.03	-198.34	-182.83	-182.73	-197.2	-218.05
5.	EZTP.	-41.28	-40.25	-38.32	-38.2	-39.38	-43.37
6.	EZTN.	36.39	34.61	30.69	31.8	32.00	31.99

Sign Convention:Anti-Clockwise Moment.(+).

Clockwise Moment.(-).

END SHEARS FOR BASIC LOAD CASES:

TABLE:7

Sl 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	20.23	37.5	37.5	20.23	17.19
3	EXTP	-90.75	90.75	-82.64	82.64	-90.76	90.76
4	EXTN	-80.7	80.7	-73.95	73.95	-80.7	80.7
5	EZTP	-34.74	34.74	-34.34	34.34	-35.3	35.3
6	EZTN	34.8	-34.8	33.90	-33.90	34.24	-34.24

Sign Convention:Upward Force(+)'

Downward Force(-).

FACTORED END MOMENTS(KNM):

TABLE:8

COMB.NO.	LOAD COMBINATIONS.	B2001		B2002		B2003	
		LEFT	RIGHT	LEFT	RIGHT	LEFT	RIGHT
1	1.5(DL+IL)	204.20	- 281.70	371.66	- 371.66	281.71	- 204.21
2	1.2(DL+IL+EXTP)	- 148.34	- 508.42	36.43	-558.2	-57.72	-475.1
3	1.2(DL+IL+EXTN)	-94.68	-459.6	81.53	- 513.12	-8.9	-421.4
4	1.2(DL+IL-EXTP)	475	57.7	558.2	-36.44	508.44	148.38
5	1.2(DL+IL-EXTN)	421.4	8.87	513.12	-81.53	459.6	94.7

6	1.2(DL+IL+EZTP)	117.42	-270.1	253.74	-340.76	178.8	-213
7	1.2(DL+IL+EZTN)	204.6	-187.4	332.82	-261.68	261.9	-125.8
8	1.2(DL+IL-EZTP)	209.3	-180.66	340.9	-253.9	271.42	-113.7
9	1.2(DL+IL-EZTN)	122.1	-263.29	261.83	-332.96	188.8	-200.93
10	1.5(DL+EXTP)	-212.7	-590.7	-42.68	-609.54	-116.9	-566.6
11	1.5(DL+EXTN)	-145.62	-529.71	13.69	-553.18	-55.9	-499.5
12	1.5(DL-EXTP)	566.5	116.9	609.5	42.66	590.8	212.7
13	1.5(DL-EXTN)	499.47	55.86	553.2	-13.7	529.7	145.65
14	1.5(DL+EZTP)	119.5	-292.8	228.96	-337.74	179.36	-238.98
15	1.5(DL+EZTN)	228.51	-189.51	327.81	-238.9	282.63	-129.96
16	1.5(DL-EZTP)	234.35	-181.05	337.92	-229.14	294.5	-114.87
17	1.5(DL-EZTN)	125.34	-284.34	239.07	-328	191.22	-223.89
18	(0.9DL+1.5EXTP)	-283.47	-496	-156.05	-496.16	-211.7	-495.8
19	(0.9DL+1.5EXTN)	-216.39	-434.94	-99.68	-439.8	-150.6	-428.73

20	(0.9DL-1.5EXTP)	495.78	211.66	496.18	156.03	496	283.5
21	(0.9DL-1.5EXTN)	428.7	150.63	439.8	99.68	435	216.42
22	(0.9DL+1.5EZTP)	48.05	-198.03	115.58	-224.36	84.6	-168.21
23	(0.9DL+1.5EZTN)	157.74	-94.74	214.43	-125.5	187.86	-59.19
24	(0.9DL-1.5EZTP)	163.6	-86.28	224.54	-115.76	199.7	-44.1
25	(0.9DL-1.5EZTN)	54.57	-189.57	125.7	-214.6	96.45	-153.12

FACTORED END SHEARS:

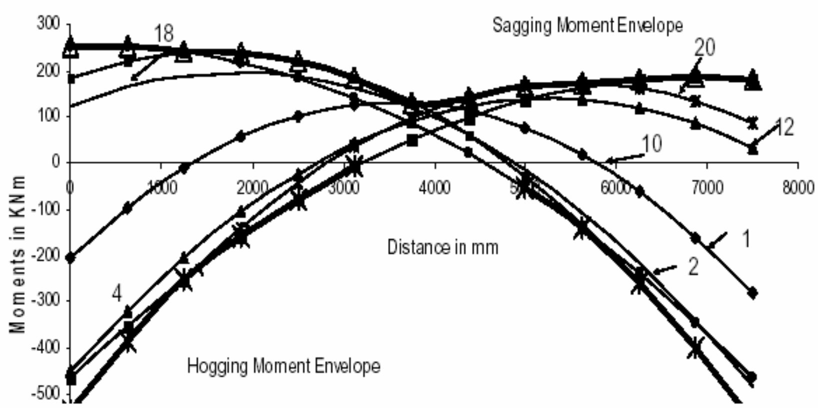
TABLE:9

COMBINATION NO:	LOAD COMBINATION	B2001		B2002		B2003	
		LEFT	RIGHT	LEFT	RIGHT	LEFT	RIGHT
1	1.5(DL+IL)	189.35	210.03	267.3	266.34	210	189.35
2	1.2(DL+IL+EXTP)	42.58	276.92	113.92	312.25	59.12	260.39
4	1.2(DL+IL-EXTP)	260.38	59.12	312.25	113.92	276.94	42.56
10	1.5(DL+EXTP)	27.44	315.69	86.15	334	43.43	299.7

12	1.5(DL-EXTP)	299.7	43.44	334	86.15	315.7	27.4
18	(0.9DL+1.5EXTP)	-38	243.86	2.1	250	-28.4	234.28
20	(0.9DL-1.5EXTP)	234.26	-28.39	250	2.1	243.9	-38

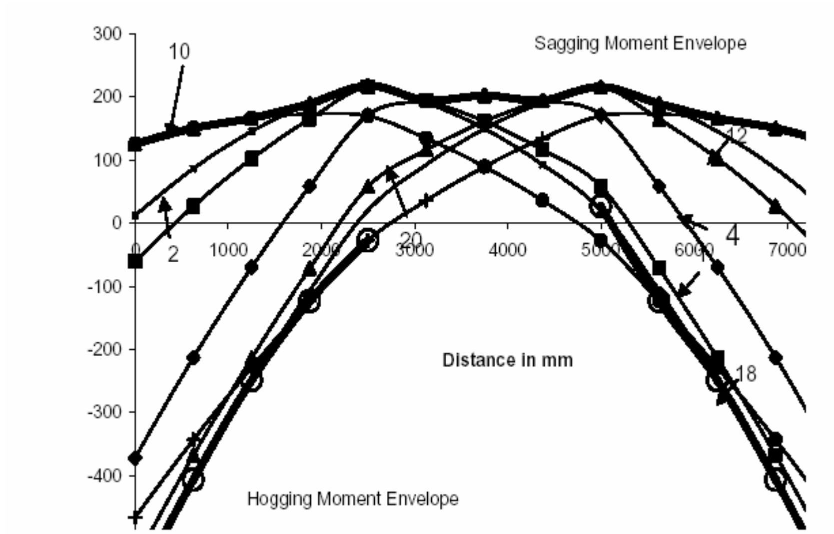
Sign Convention:Upward Force(+)'

Force
Downward (-).



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

DESIGN MOMENTS AND SHEARS AT VARIOUS LOCATIONS:

Table-10

BEAM	B2001		B2002		B2003	
DISTANCE FROM LEFT END	MOMENT KNM	SHEAR KN	MOMENT KNM	SHEAR KN	MOMENT KNM	SHEAR KN
0	-567 284	300	-610 156	334	-591 212	316
625	-416 283	270	-440 181	358	-430 218	280
1250	-248 238	180	-240 167	245	-280 185	248
1875	-160 240	189	-153 199	223	-181 180	198
2500	-80 224	160	-57 238	205	-85 180	166
3125	-18 190	122	0 215	128	0 150	133
3750	0 130	-111	0 225	95	0 140	109
4375	0 140	-138	0 222	-117	-38 201	-122
5000	-80 180	-166	-37 258	-135	-98 236	-156
5625	-151 178	-190	-133 200	-235	-170 245	-179
6250	-278 190	-224	-279 197	-260	-284 256	-208
6875	-450 200	-270	-457 181	-280	-412 265	250
7500	-591 212	-316	-610 156	-334	-567 284	-300

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

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	300	532	6.02	D	1.99	0.84	3 176	744
	+136	300	540	1.55	S	0.466	-	755	,-
1 250	-249	300	532	2.93	S	0.97	-	1 548	-
	+167	300	540	1.91	S	0.59	-	956	-
2 500	-27	300	557	0.29	S	0.09	-	150	-
	+218	300	540	2.49	S	0.80	-	1 296	-
3 750	0	300	557	0	S	0	-	0	-
	+202	300	560	2.15	S	0.67	-	1 126	-
5 000	-27	300	557	0.29	S	0.09	-	150	-
	+218	300	540	2.49	S	0.80	-	1 296	-
6 250	-249	300	532	2.93	S	0.97	-	1 548	-
	+167	300	540	1.91	S	0.59	-	956	-
7 250	-511	300	532	6.02	D	1.99	0.84	3 176	744
	+136	300	540	1.55	S	0.466	-	755	,-

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	486	486	486	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	486	486	486	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

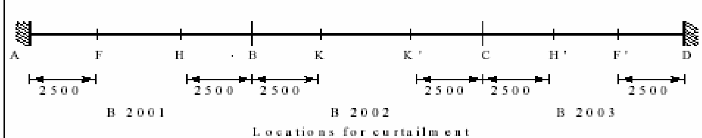


Figure 5 Critical Sections for the Beams

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.

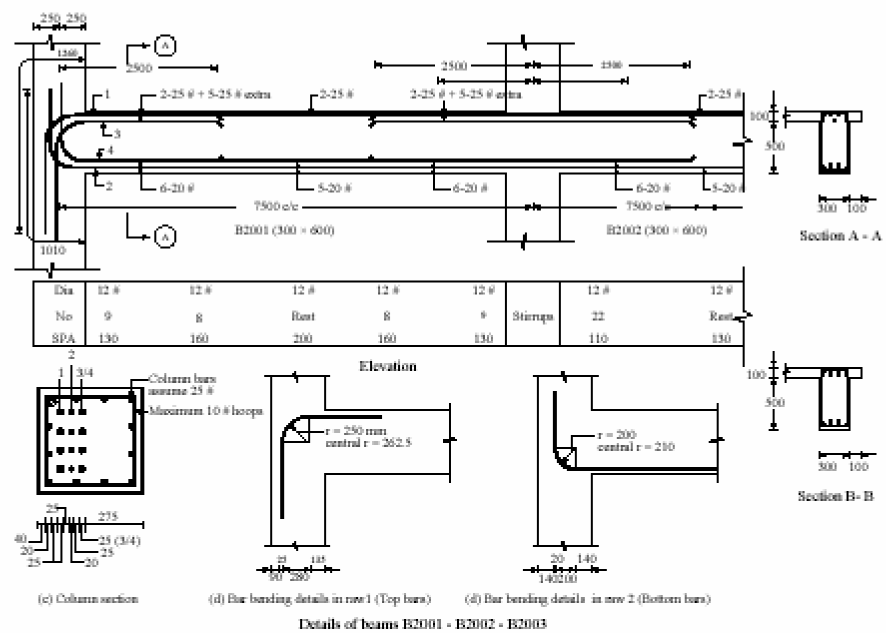


Figure 6 Details of Beams B2001, B2002 and B2003

Check for reinforcement

(a) Minimum two bars should be continuous at top and bottom. Here, 2–25 mm # (982 mm2) are continuous throughout at top; and 5–20 mm # (1 570 mm2) are continuous throughout at bottom. Hence, ok.

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

$$= 0.00289, \quad \text{i.e., } 0.289\%.$$

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

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 = $\frac{3437}{2}$ 1718 mm2

Positive steel = 1884 mm2 > 1718 mm2

Hence, ok.

Joint B

Half the negative steel = $\frac{3437}{2}$ 1718 mm2

Positive steel = 1 884 mm2 > 1 718 mm2

Hence, ok.

(IS 13920; Clause 6.2.4)

Along the length of the beam,

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

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

≥ Ý 859 mm2

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.

Web reinforcements

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

Hoop diameter ≥Ý 6 mm

≥ Ý 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:

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

$M_u^{AH} = 568 kNm$ $\quad\quad$ $M_u^{Bh} = 568 kNm$

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

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

$M_u^{AH} = 585 kNm$ $\quad\quad$ $M_u^{Bh} = 585 kNm$

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:

$$V_a^{D+L} + V_b^{D+L} = 5.0\ x\ 7.5\ x\ 42.6 = 159.7kN$$

For beam 2002:

$$V_a^{D+L} + V_b^{D+L} = 5.0x7.5x31.3 + 95.6 = 213kN$$

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

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

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

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

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

Beam 2002
Sway to right

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

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

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

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

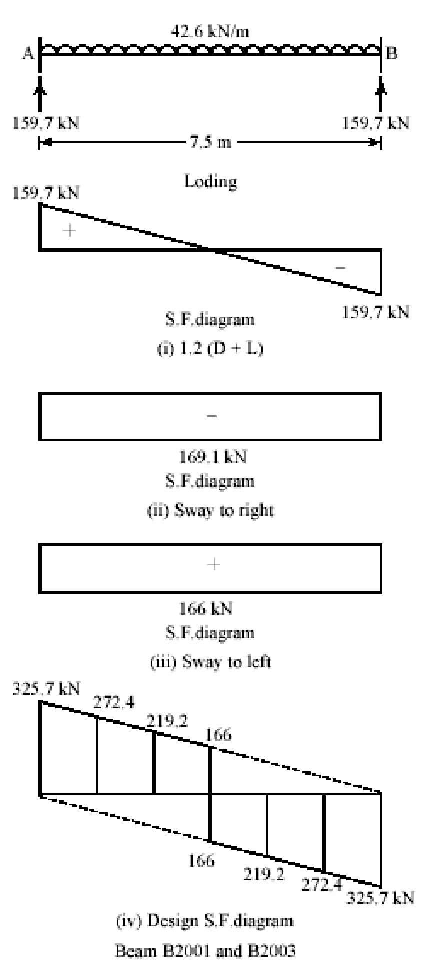


Figure 7 Beam Shears due to Plastic Hinge Formation for Beams B2001 and B2003

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.

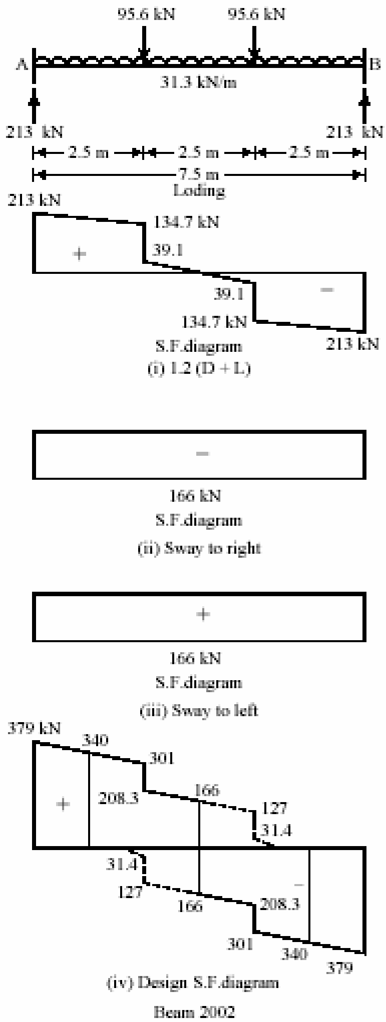


Figure 8 Beam Shears due to Plastic Hinge Formation for Beam B 2002

Design Example of a Building

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) 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,

$$s \le \frac{d}{2} = \frac{532}{2} = 266 \text{ mm.}$$

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/mm², $x_{u,max}$ = 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 mm²	7-25 # = 3 437 mm²	7-25 # = 3 437 mm²	7-25 # = 3 437 mm²
Bottom bars	6-20 # = 1 884 mm²	6-20 # = 1 884 mm²	6-20 # = 1 884 mm²	6-20 # = 1 884 mm²
A_{st} (mm²)	1 884	3 437	1 884	3 437
A_{sc} (mm²)	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
$M_{uc1} = (0.36 f_{ck} b x_u) \times (d - 0.42 x_u)$	-	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.

CHAPTER-8

DESIGN OF COLUMN

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.

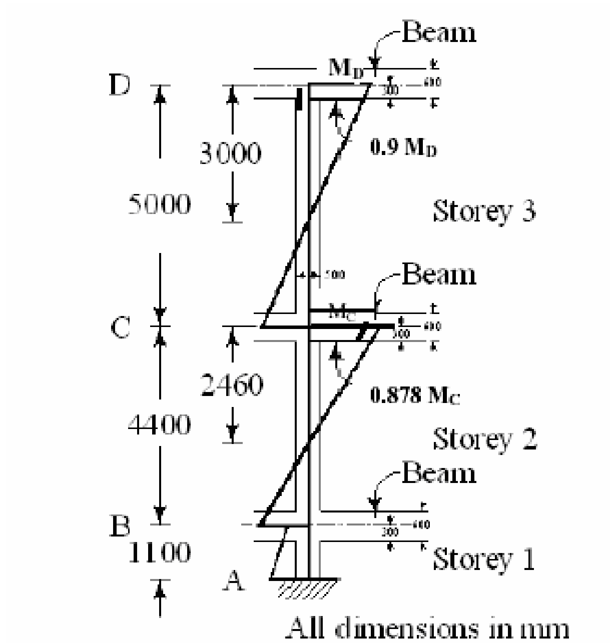


Figure 9 Determining moments in the column at the face of the beam.

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.

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.

Table 21 Stiffness factors for Selected Members

Member	Size (mm)	I (mm^4)	*l* (mm)	Stiffness Factor $(I/l)\times10^{-3}$
All Beams	300 x 600	5.4×10^9	7 500	720
Columns				
C101, C102	600 x 600	1.08×10^{10}	1 100	9 818
C201, C202	500 x 500	5.2×10^9	4 100	1 268
C301, C302	500 x 500	5.2×10^9	5 000	1 040
C401, C402	500x 500	5.2×10^9	5 000	1 040

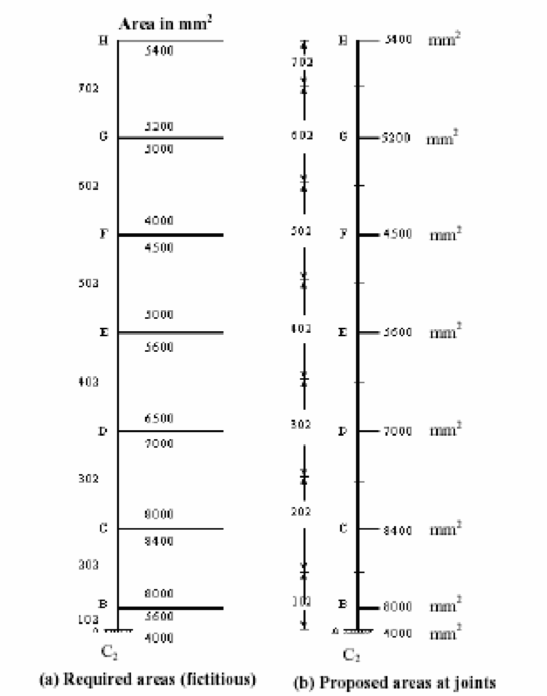


Figure 10 Description of procedure to assume reinforcement in a typical column

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

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

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

(IS 456:2000, Clause 25.4)

For lower height of column, L = 4,100 – 600

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

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

Similarly, for all the columns in first and second storey, ex,min = ey,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.

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

For column C2 in all floors, i.e., columns C102,

C202, C302, C402, C502, C602 and C702,

$f$ck = $25 \text{ N/mm}^2, f_y = 415 \text{ N/mm}^2, \text{ and } \frac{d'}{d} = \frac{50}{500} = 0.1.$

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., $p = 0.105 \times 25 = 2.625$, 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., $p = 0.11 \times 25 = 2.75$, 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 mm2, equally distributed on all faces.

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

$$P_{uz} = [0.45 \times 25(500 \times 500 - 6968) + 0.75 \times 415 \times 6968] \times 10^{-3} = 4902\ kN.$$

Calculations given in Tables 24 to 27 are self-explanatory

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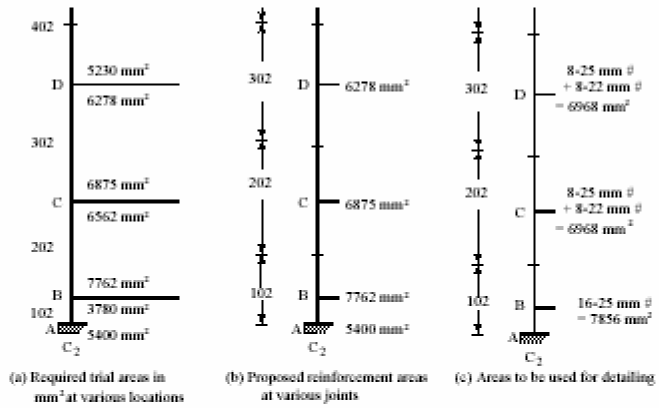

Figure 11 Required Area of Steel at Various Sections in Column

TABLE 24 TRIAL SECTION BELOW JOINT C

Com b. No.	Pu, kN	Centreline moment		Moment at face		Cal. Ecc.,mm		Des. Ecc.,mm		Mux, kNm	Muz, kNm	P'u	M'uz	$\frac{P'_u}{f_{ck}bD}$	$\frac{M'_u}{f_{ck}bD}$	$\frac{p}{f_{ck}}$
		Mux, kNm	Muz, kNm	Mux, kNm	Muz, kNm	ex	ez	edx	edz							
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}	$\frac{P'_u}{f_{ck}bD}$	$\frac{M'_x}{f_{ck}bD^2}$	$\frac{p}{f_{ck}}$
		M_{uxr} kNm	M_{uzr} kNm	M_{uxr} kNm	M_{uzr} 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

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.

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 φmin (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 mm2.

The spacing should not exceed

(i) $\frac{0.87\, f_y\, A_{sv}}{0.4b}$

(requirement for minimum shear reinforcement)

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

= mm 451.3

(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 . b/2 of column

= 500 / 2 = 250 mm ………………………………………………………. **(3)**

From **(1), (2)** and **(3)**, maximum spacing of

stirrups is 250 mm c/c.

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 C202, lower height, Vu = 161.2 kN, for load combination 12.

For C202, upper height, Vu = 170.0 kN, for load combination 12.

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

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

$M^{bL}_{u,lim}$ = 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.

Than

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

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

Design shear is maximum of (a) and (b). Then, design shear Vu = 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, Pu = 3,027 kN for lower length and Pu = 2,547 kN for upper length.

Than

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

$$= 1 + \frac{3\times3027\times1000}{500\times500\times25} = 2.45, \text{ for lower length, and}$$

$$= 1 + \frac{3\times2547\times1000}{500\times500\times25} = 2.22, \text{ for upper length.}$$

≤ 1.5

Table δ =1.5

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

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

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

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

$A_{sv} = 250\text{mm}^2$, using 8mm # 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 l0, 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 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 ≤ 0.25 *D* = 0.25 x 500 = 125 mm≤ 75 mm ≤. 100mm
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:

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

Here average *h* referring to fig 12 is

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

$$A_{sh} = 50.26 \text{ mm}^2$$

$$A_k = 428 \text{ mm x } 428 \text{ mm}$$

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

$$50.26 = 0.4232 \text{ s}$$

$$\text{s} = 118.7 \text{ mm}$$

$$\leq 100 \text{ mm} \qquad \ldots \qquad \ldots \qquad (2)$$

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

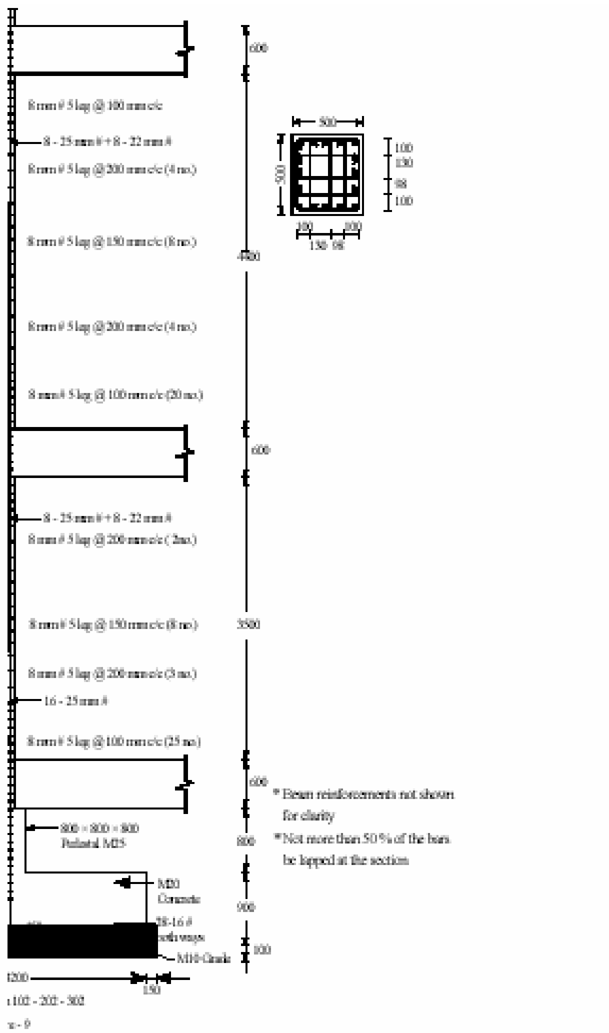

Figure 12 Reinforcement Details

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 = Ld in tension = 40.3 db. 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 db, the spacing of hoops is reduced to 150 mm. Refer to Figure 12.

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.

CHAPTER-9

DESIGN OF FOOTING

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

Hx = 12 kN, Hz = 16 kN

Mx = 12 kNm, Mz = 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).

Mx1 = Mx + Hy .D

= 12 + 16 . 0.9 = 26.4 kNm

Mz1 = Mz +Hy .D

= 6 + 12 . 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 . 4.2 = 17.64 m2

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

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

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

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

Maximum soil pressure

= 189 + 2.14 + 1.52

= 192.66 kN/m2 < 200 kN/m2

Minimum soil pressure

= 189 – 2.14 – 1.52

= 185.34 kN/m2 > 0 kN/m2.

Case 2:

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

Permissible soil pressure is increased by 25%.

i.e., allowable bearing pressure = 200 .1.25

P = (2291 - 44) = 2247 kN

Hx = 92 kN, Hz = 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 .*D*

= 3 + 13 . 0.9 = 14.7 kNm

*M*z1 = *M*z +*H*y .*D*

= 216 + 92 . 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/m2 < 250 kN/m2.

Minimum soil pressure

= 152.04 - 1.19 – 24.2

= 126.65 kN/m2 > 0 kN/m2.

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 +LI

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

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

For *M*x

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

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

For M_z

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

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

Since there is no much difference in the values, the footing shall be designed for *Mz* for an upward pressure of 250 kN/m2 on one edge and 167 kN/m2 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.

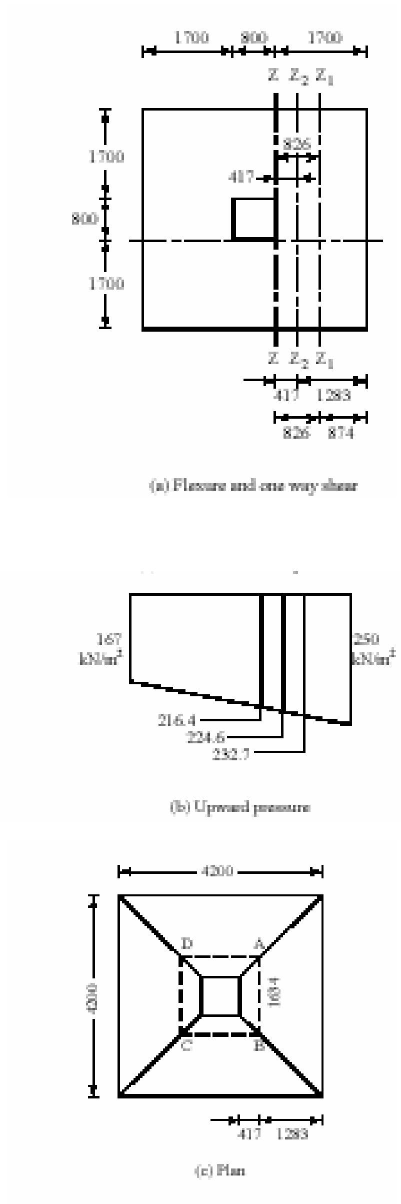

Figure 13

(b) Size of pedestal: A pedestal of size 800 mm . 800 mm is used. For a pedestal

A = 800 . 800 = 640000 mm2

Z = 3 2 mm 85333333 800 800

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

For case 1

$$q_{01} = \frac{2899 \times 1000}{800 \times 800} + \frac{(26.4 + 18.8) \times 10^6}{85333333}$$
$$= 4.53 + 0.53 = 5.06 \text{ N/mm}^2 \ldots \quad (1)$$

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

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

From (1) and (2) consider $q_0 = 5.4$ N/mm2.
For the pedestal

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

This gives

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

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.

$$M_{xz} = \left[\frac{1}{2} \times 216.4 \times 1.7 \times \frac{1}{3} \times 1.7 + \frac{1}{2} \times 250 \times 1.7 \times \frac{2}{3} \times 1.7\right] \times 4.2$$
$$= 1449 \text{ kNm}$$

For the pad footing, width b = 4200 mm

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

Balanced depth required

$$= \sqrt{\frac{1449\times10^6}{2.76\times4200}} = 354 \text{ mm}$$

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

dx = 900 – 50 – 8 = 842 mm

dz = 842 – 16 = 826 mm.

Average depth = 0.5(842+826) = 834 mm. Design for z direction.

$$\frac{M_{uz}}{bd^2} = \frac{1449\times10^6}{4200\times826\times826} = 0.506$$

$$p_t = 0.145, \text{ from table 2, SP :16}$$

$$A_{st} = \frac{0.145}{100}\times4200\times900 = 5481 \text{ mm}^2$$

$$A_{st,min} = \frac{0.12}{100}\times4200\times900 = 4536 \text{ mm}^2$$

(Clause 34.5, IS: 456)

Provide 28 no. 16 mm diameter bars.

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

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

$$< 3\times826 \text{ mm} \ldots\ldots \ldots. \text{ (o.k.)}$$

(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 z1-z1

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

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

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

$\tau_c = 0.289$ 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 + 2.417 = 1634 mm.

Two-way shear along linr *AB*

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

$$\tau_v = \frac{V_u}{bd} = \frac{883 \times 1000}{1634 \times 834} = 0.648 \text{ N/mm}^2$$

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/mm².

Here $\tau_v < \tau_c$ … … … (o.k.)

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

Design bearing pressure at the base of pedestal

= 2 N/mm 25 . 11 25 45 . 0 45 . 0 = . = *ck f*

Design bearing pressure at the top of the footing

$$= \sqrt{\frac{A_1}{A_2}} \times 0.45 f_{ck} = 2 \times 0.45 \times 20 = 18 \text{ N/mm}^2$$

Thus design bearing pressure = 11.25 N/mm2.

Actual bearing pressure for case 1

= 1.5 .*q*01= 1.5 . 5.06 = 7.59 N/mm2.

Actual bearing pressure for case 2

= 1.2 .*q*02= 1.2 . 7.18 = 8.62 N/mm2.

Thus dowels are not required.

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

= 3200 mm2.

Area of column bars = 7856 mm2

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 = Ld 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 midheight of the column.

(h) Weight of the footing:

= 4.2 . 4.2 . 0.9 . 25 = 396.9 KN(assumed)

< 4 35KN.

Conclusions

1. The tasks of providing absolute seismic safety for the residents inhabiting the most earthquake-prone regions are far from being solved. However, new regulations on construction that contribute to earthquake disaster mitigation have been introduced and implemented in accordance with world practice. These regulations are based on experience of past earthquakes and results of special researches, summarized in adequate documents of many states.

The regulations of each country depend on local experience in seismic design, however they have some common guidance. In the regulations adopted for implementation in India the following factors have been found to be critically important in the design and construction of seismic resistant buildings:

- To select sites for construction that are the most favorable in terms of the frequency of occurrence and the likely severity of ground shaking and ground failure;
- To apply structural-spatial solutions that provide symmetry and regularity in the distribution of mass and stiffness in plan and in elevation;
- To implement the design of building elements and joints between them in accordance with analysis that take into account the structural requirements;
- To provide high quality of construction to ensure good performance during future earthquakes.

Researchers indicates that compliance with the above-mentioned requirements will contribute significantly to disaster mitigation, regardless of the intensity of the seismic loads and specific features of the earthquakes.

The modifications in construction and design that have been introduced increase seismic reliability of the buildings and seismic safety for human life.

At the same time, current regulations cannot be considered as final. As new data become available they have to be defined more precisely and new provisions have to be added to meet the needs.

The conducted analysis illustrated that the main reasons of tragic earthquake consequences are to be considered as follows:

- The violations of code requirement on design and construction of structures, and in some countries – low normative basis of earthquake engineering, in part of legislative force;
- Selection for construction sites if multi-storied buildings – the sites with unfavorable in terms of seismic design soil conditions;
- The Application of simplified procedure for co-ordination and licensing on launching of construction;

- Lack of due quality control of performance construction.

In order to improve seismic safety of residents and to prevent huge property damage caused of earthquake it is necessary:

- To strengthen role of design an construction control by State body;
- To design and construct on seismic unfavorable sites only by authority of appropriate experts’ commission;
- To implement widely the test practice and quality control of building material and elements, used in construction; The buildings of new construction types must be put into operation only after conducting model and real tests;
- To develop guidance on design for buildings of some types (e.g. frame and frame buildings with stiffening core) and to specify series of current seismic code provisions;
- To allow implementation of construction activity only for companies with appropriate license and documents proved the right on conducting certain kinds of works;
- To conduct full-scale investigation of built-up area developments which are located in seismic active regions, and then in accordance with systematization and analysis of investigation results to carry out the measures in order to mitigate consequences of disaster.

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