Purbanchal University

PU SCHOOL OF ENGINEERING & TECHNOLOGY

Biratnagar

A

PROJECT REPORT

ON

STRUCTURAL ANALYSIS AND DESIGN OF RCC FRAMED OFFICE BUILDING

SUBMITTED BY:- **SYMBOL NO:-**

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Submitted To the Faculty of Science and Technology

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CERTIFICATE

This is to certify that the project entitled "ANALYSIS & DESIGN OF OFFICE BUILDING FOR EARTHQUAKE RESISTANCE", submitted by , group members Santosh Agrahari (382265), Sandesh Chaulagai (382262), Milan kumar Lohar (382255), Kunta Sharma (382252), Dibya Dutta (382248), Sant kumar Mahato (382264), Shiva Nandan Shah (382267) Yogendra Thakur (382271) students of BCE 2070, is accepted as their final project report for the award of degree in Bachelors in Civil Engineering under Purwanchal University.

The project is carried under my supervision and the materials they have included in this report are the outcome of three months authentic work.

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ABSTRACT

The report presents the analysis and design of a municipal office building located in the Itahari Sub Metropolitan City, at Aitabare-5 undertaken as a partial fulfillment of Bachelor in Civil Engineering (BCE) 8^{th} semester of course requirement. In the view of seismic vulnerability of the country, seismic load on the building was given due consideration, in addition to the wind load. Load calculation was done as according to IS: 875 – 1987 (Code of practice for Design Loads for building & structure) and IS 1893 – 2002 (Criteria for Earthquake resistant – Design and Structure). Modeling & Analysis was done with the help of SAP 2000 structural analysis software. Design is based on Limit State of Method. Detailing has been carried out as according to SP 34 (S & T) – 1987 (Handbook on concrete Reinforced & Detailing) and IS 13920 – 1993 (Ductile Detailing of RC Structure subjected to seismic forces).

LIST OF SYMBOLS

A_{st} = Area of steel

b_f = Breadth of slab/compression face/flange

b_w = Breadth of web or rib

D = Overall depth of beam or slab

D_f = Thickness of flange

DL = Dead Load

d = Effective depth of beam or slab

d' = Depth of compression reinforcement

E_c = Modulus of elasticity of concrete

EL = Earthquake load

E_s = Modulus of elasticity of steel

e = Eccentricity

f_{ck} = Characteristic cube compressive strength of concrete

f_y = Characteristic strength of steel

I = Moment of inertia

k = Constant or coefficient or factor

L_d = Development Length

LL = Live Load or imposed load

l_{eff} = Effective span of beam or slab or effective length of column

l_x = Length of shorter side of slab

l_y = Length of longer side of slab

α_x, α_y = Bending moment Coefficients

l_0 = Distance between points of zero moments in a beam

M = Bending moment

P = Axial load on a compression member

q_0 = Calculated maximum bearing pressure of soil

A_{sv} = Total cross-sectional area of stirrup legs or bent-up bars within a distance s_v

A_c = Area of concrete

A_{sv} = Area of longitudinal reinforcement for columns

s_v = Spacing of stirrups

V_{us} = Strength of shear reinforcement

T = Torsional moment

T	=	Thickness of wall
V	=	Shear force
W	=	Total Load
WL	=	Wind Load
w	=	Uniformly distributed load
w_u	=	Ultimate load
M_u	=	Ultimate bending moment
M_{lim}	=	Limiting bending moment
x	=	Depth of neutral axis
x_{lim}	=	limiting neutral axis depth
τ_{bd}	=	Design bond stress
τ_c	=	Shear stress in concrete
$\tau_{c,max}$	=	Maximum shear stress in concrete with shear reinforcement
τ_v	=	Nominal shear stress
ϕ	=	Diameter of bar

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CHAPTER-1

INTRODUCTION

1.1 GENERAL

Nowadays, for the development of any nation, it is essential to construct the infrastructures regarding various constructions works. To carry out such works engineers and various technicians play an important role entirely through the governmental or private sector. Government having inadequate resources, nowadays, private sectors is immensely activated towards the construction works on the contract basis.

In this scenario, regarding several projects which were offered to us, we have chosen the project entitled "Analysis and Design of Office Building of Earthquake Resistance". Comparing various models of building structures, we found public building as the most challenging project. Thus, we have given preference to the construction of the frame structured municipal building which is a frame structure of three-stories with basement. There are total 17 frames. These frames are analyzed for various vertical (Live, Dead) and horizontal (Earthquake, wind) loads. The site is located in Itahari-5 at Aitabare. According to IS 1893-2002 (Criteria for Earthquake Resistant design of structures), Itahari lying on v^{th} zone, earthquake load was found to be pre-dominant in case of lateral load.

Earthquake is a natural phenomenon as old as the history of the earth itself. It occurs rarely at any particular location. However the effects of these events are very destructive. Amongst the other natural disasters, earthquake is the most unpredictable one with maximum risk. Massive loss of lives and properties occurs. Now a days, designers and engineers are giving more emphasis towards the earthquake resistance while analyzing and designing any structure to minimize the seismic impact.

Various types of loads during the design phase are considered. While analyzing the structure, internal forces in various structural members are discussed. Structural design deals with the designing various members of the structure to resist these internal forces to which they might be subjected during the life period of the structure.

While analysis, frames are analyzed for earthquake as lateral or horizontal load. During the earthquake, structural and non-structural damages occur in which both of them are hazardous to occupants. When earthquake shaking occurs, a building gets thrown from side to side or up and down. I.e. ground moves violently side by side and building stay at rest. Thus the building gets thrown back and forth by the motion of the ground with some part of building lagging behind and then moving in the opposite direction. The level of damage depends upon how well the building has been designed and constructed.

In our context, the analysis of earthquake is based on seismic coefficient design method as described in IS 1893: 2002. The lumped masses are calculated in floor level and with the help of horizontal base shear; the lateral load (earthquake load) is calculated as well as distributed.

The project report has been prepared in complete conformity with various provisions in Indian Standards as Code of practice for plain & reinforced concrete IS 456-2000; Code of practice for Design loads IS 875 (Part 2)-1987: Design Aids for Reinforcement Concrete (SP 16) IS 456-1978; Handbook on Concrete Reinforcement and Detailing SP 34 (1987) are thoroughly referred for proper analysis, design and detailing of structural elements viz. beam, slab, column, staircase, foundation, basement wall with respect to safety, strength, stability, ductility & economy in addition to adequate serviceability requirements of cracking and deflection in concrete structures. All the codes are based on the principles of limit state of design.

The project report posses modeling output (SAP 2000), analysis results, load calculations, architectural drawings, structural drawings and sample calculations of various structural elements and their detailing as well.

1.2 OBJECTIVES

The main objective is to achieve a practical knowledge on structural analysis, design and detailing of a building

- To estimate the various loads on the building
- To determine the dimensions of the structural elements of the building
- To design the structural elements of the building
- To make detailing of the structure system
- To design the structure on the basis of allowable material stresses, the section sizes, concrete strengths & amounts, reinforcements
- To estimate the load effects that can be exerted to each critical section in the structure under design loads (BM, SF, thrust)
- To know the methodology for carrying out the structural analysis and detailing in a real practice

1.3 SALIENT FEATURES

Building type	:	Office/Public building
Structural system	:	RCC Frame structure
No. of Storey	:	3 (excluding staircase covering) + Basement
Floor Height	:	Floor – 3.302 m (10'10")
		Basement – 3.606m (11'10")
Length	:	36.576 m
Breadth	:	21.336 m
Plinth area	:	569.205 sq. m
Type of staircase	:	Open – well staircase
Type of foundation	:	Mat foundation
Type of sub-soil	:	Medium Sub-soil (Bearing Capacity = 130 KN/m^2)

FRAME SYSTEM

Direction	Frame Naming
Transverse	1-1, 2-2, 3-3, 4-4, 5-5, 6-6, 7-7, 8-8, 9-9, 10-10
Longitudinal	A-A, B-B, C-C, D-D, E-E, F-F, G-G,

1.4 METHODOLOGY

a) Study of the Architectural Drawing

Initially, the architectural drawing of the building was studied. Rooms within this office building were allocated to various purposes such as staff room, administration, seminar hall, storage etc.

b) Preliminary Design

Estimation of various structural elements such as beam and slab were designed and checks were done with the help of deflection criteria and moment criteria. For the column, vertical axial capacity was taken for the design and percentage of steel was checked.

c) Load Calculation

After the study architectural drawing and preliminary design, load calculation was done. In vertical, dead load was obtained by the sized determined in preliminary design and live load was determined by using code for design loads (IS 875 part 2) for various types and purposes of rooms. In horizontal load, earthquake load was determined by calculating lumped mass at floor level and horizontal base shear (IS 1893). It was done by Seismic Coefficient Method. Wind load (IS 875 Part 3) was obtained by design wind speed and design wind pressure. Earthquake load being the pre-dominant one between the two lateral loads, hence its effect was only considered.

d) Modeling & Analysis

For the purpose of Seismic analysis of our building we have used the structural analysis program SAP 2000. It has a special option for modeling horizontal rigid floor diaphragm system. A floor diaphragm is modeled as a rigid horizontal plane parallel to global X-Y plane, so that all points on any floor diaphragm cannot displace relative to each other in X-Y plane. Initially, the characteristics of the materials used were defined such as concrete – M20 and reinforcement – Fe415. Then, the load cases as well as their combinations with load factors were introduced. Next, structures were analyzed for different load combinations and the final output was determined in the form of SF, BM and AF etc.

e) Design

Design was done on the basis of limit state of design for collapse and serviceability. The sample calculations of various structural elements were done with numerous checks and with the help of MS-excel, the formulation was done for each and every structural member in the building.

f) Detailing

Detailing was to done by determining number, size, layout and location of reinforcement, given the element dimensions and areas of steel required. Certain details such as lap and development lengths, hook requirements, cut-off points etc. were covered by the code.

CHAPTER-2

ANALYSIS OF BUILDING

The analysis of the building was done by the estimation of dimensions of various structural members such as slab, beam, column, staircase, foundation, and basement wall with the help of preliminary design. And different types of loads such as vertical load (Dead + finishes, and Live) and Lateral Load (earthquake and wind load) were calculated. Earthquake being pre-dominant, only its effect was taken for lateral loads. Also combinations of such loads were taken into consideration. With the help of SAP 2000, element stresses in beams and column were calculated in the provision of rigid diaphragm with.

2.1 PRELIMINARY DESIGN

Preliminary design is carried out to estimate approximate size of the structural members. Grid diagram is taken as basic guideline for analysis. Preliminary design of flexural members of the structural system i.e. for beam and slab are done as per the limit state of serviceability. Work out is done from deflection criteria and moment criteria. And for column, it was done from net vertical axial load capacity assuming suitable percentage of steel.

SLAB DESIGN

Reference	Step	Calculation	Remarks
		Slab (Seminar Hall)	
	1	**From deflection criteria**, we have,	
		$d \geq \frac{l_x}{\alpha\beta\gamma\lambda\delta}$, l_x = 15' = 4.572m	
IS 456-2000 Clause 23.2 (fig. 4)		where, α = 26 (for continuous slab two way) δ = 1 (for compression steel) β = 1 (for span less than 10m) λ = 1 (for no web flange)	
		$\gamma = 0.55 + \frac{477 - \sigma_s}{120\left(0.9 + \frac{Mu}{bd2}\right)} \leq 2.0$	

BS 8110-1985 Clause 3.4.5.6		$\sigma_s = \sigma_{y/\eta}\left(\frac{A_{reqd}}{A_{taken}}\right)\left(\frac{1}{\beta_b}\right)$ $= (415/1.8) * 1$ $= 230.55\ N/mm^2$	
A.K. Jain Pg. 187)	2	**For Tension side:** $M_r = 0.87\ f_y\ A_{st}\ (d - 0.42x_{u,l})$	
IS 456-2000 Annex G		$= 0.87*415*(P/100)*bd\ (d-0.42*0.48d)$ $= 0.87*415*(0.5/100)*bd\ (d-0.42*0.48d)$ $= 1.44\ bd^2$ **For Compression side:**	
IS 456-2000 Annex G		$M_c = Kf_{ck}bd^2$ Equating both, we get, $Kf_{ck} = 1.44$ $M_u = Kf_{ck}bd^2$ $M_u/bd^2 = 1.44$ $\gamma = 0.55 + \frac{477 - 230.55}{120(0.9 + 1.44)}$ **$\gamma = 1.427 < 2.0$ (OK)** $\therefore d = \frac{4.572*10^3}{26*1.427} = 123.23\ mm \approx 124\ mm$ Adopt overall depth (D) = **150 mm** $\therefore$ Effective depth (d) = 150-15-6 =129 mm	
	3	**Check by Moment Criteria** Calculation of Load For slab (floor) 1. Dead load i) R.C.C Slab = γ*t = 25*0.15 = 3.75 KN/m² ii) 15 mm marble = γ_m*t = 27*0.015 = 0.405 KN/m² iii) 25 mm screed = γ_s*t = 21*0.025 = 0.525 KN/m² $\therefore$ Total dead load = 4.68 KN/m² Live load = 5 KN/m² $\therefore$ Total load (W) = 9.68 KN/m² per m $\approx$ 9.68 KN/m $\therefore$ Ultimate load (W_u) = 1.5*9.68 = 14.52 KN/m For roof slab 1. Dead Load I. R.C.C Slab = γ*t = 25*0.15 = 3.75 KN/m² II. 25 mm plaster = γ*t = 22*0.025 = 0.55 KN/m²	

		Total dead load (W) = 4.3 KN/m^2	
		Ultimate load (W_u) = 4.3*1.5	
		= 6.45 KN/m^2	
		Let us take	
		Maximum Moment (Fig.)	
		$M_x = \alpha_x w_u l_x^2$	
		= 0.039*14.52*4.572^2	
		= 11.837 KN–m	
IS 456-2000 Annex D Table 26		$M_y = \alpha_y w_u l_x^2$	
		= 0.032*14.52*4.572^2	
		= 9.72 KN-m	
		∴ M = 11.837 KN-m	
		∵ $M_u = 0.138 f_{ck} bd^2$	
		or, 11.837*10^6 = 0.138*20*1000*d^2	
		⇨ **d = 65.48 mm < 129 mm (OK)**	
		<u>Next Slab:</u>	
		Effective length (l_x) = 14' = 4.267 m.	
	4	**From deflection criteria**	
		$d \geq \frac{lx}{\alpha\beta\gamma\lambda\delta}$	
IS 456-2000 Clause 23.2 (fig. 4)		Where,	
		α = 26 (for continuous slab two way)	
		δ = 1 (for compression steel)	
		β = 1 (for span less than 10m)	
		λ = 1 (for no web flange)	
		$\gamma = 0.55 + \frac{477-\sigma_s}{120\left(0.9+\frac{Mu}{bd2}\right)} \leq 2.0$	
BS 8110-1985 Clause 3.4.5.6			
		$\sigma_s = \sigma_{y/n}\left(\frac{A_{reqd}}{A_{taken}}\right)\left(\frac{1}{\beta_b}\right)$	
(A.K. Jain Pg. 187)		= (415/1.8) *1	
		= 230.55 N/mm^2	
	5	**<u>For Tension side:</u>**	
		$M_r = 0.87 f_y A_{st} (d - 0.42x_{u,l})$	
		= 0.87*415*(P/100)*bd (d-0.42*0.48d)	
IS 456-2000 Annex G		= 0.87*415*(0.5/100)*bd (d-0.42*0.48d)	
		= 1.44 bd^2	

		For Compression side:	
		$M_c = Kf_{ck}bd^2$	
IS 456-2000 Annex G		Equating both, we get,	
		$Kf_{ck} = 1.44$	
		$M_u = Kf_{ck}bd^2$	
		$M_u/bd^2 = 1.44$	
		$\gamma = 0.55 + \dfrac{477 - 230.55}{120(0.9 + 1.44)}$	
		$\boldsymbol{\gamma = 1.427 < 2.0}$ **(OK)**	
		$\therefore d = \dfrac{4.267}{26 * 1.427}$ => $d \geq 0.115$ m.	
		∴ Overall depth (D) = 115+15+5 = **135 mm.**	
		Effective depth (d) = 135-15-6 = 114 mm	
	6	**Check by Moment Criteria**	
		Calculation of load:	
		For Slab	
		1. Dead load	
		I. R.C.C Slab = γ*t = 25*0.135 = 3.375 KN/m^2	
		II. 15 mm Marble = γ*t = 27*0.015 = 0.405 KN/m^2	
		Total dead load = 4.305 KN/m^2	
		Live load = 4 KN/m^2	
		Total load (W) = 8.305 KN/m^2 per m	
		Ultimate load (W_u) = 1.5*8.305 = 12.458 KN/m	
		Max. Moment	
		$M = \alpha_x w_u l_x^2$	
		$= 0.032*12.4575*4.267^2$	
		= 7.258 KN-m.	
		Now,	
		$M_u = 0.138\ f_{ck}\ bd^2$	
IS 456-2000 Annex D Table 26		Or, $7.258*10^6 = 0.138*20*1000*d^2$	
		d = 51.281 mm < 114 mm (OK)	

BEAM DESIGN

Reference	Step	Calculation	Remarks
	1	**Beam: Seminar Hall**	
IS 456-2000 Clause 23.1.2.a		$b_f = \frac{l_0}{6} + b_w + 6D_f = \frac{0.7*5.182*10^3}{6} + 230 + 6*150$ $= 1735$ mm $(l_0 = l_e * 0.7)$ & $(l_e = 5.182$ m$)$	
IS 456-2000 Clause 23.2 (fig. 4)		$d \geq \frac{l}{\alpha\beta\lambda\delta\gamma}$ Where, $\alpha = 26$ (for continuous) $\beta = 1$ (span < 10m)	
BS 8110-1985 Clause 3.4.5.6		$\gamma = 0.55 + \frac{477 - \sigma_s}{120\left(0.9 + \frac{M_u}{bd^2}\right)} \leq 2$ $\gamma = 0.82$ for 2% tensile steel $\sigma = 1.08$ for Pc = 0.25% $\lambda = 0.8$ for $\frac{b_w}{b_f} = \frac{230}{1735} = 0.133$ $\therefore d \geq \frac{5.183*10^3}{26*0.82*1.08} = 225.097$ mm $\therefore$ Adopt **D = 300 mm**	
	2	**Check by Moment Criteria:** Total area of trapezoid = ½(a+b)*h = 0.5(1.22 + 5.182)*1.98 $\therefore A_1 = 6.338$ m^2 $\quad A_2 = 6.62$ m^2 Total Area = 12.958 m^2 Slab: Live Load = 5 KN/m^2 Dead Load = γ*A*t = 25*12.958*0.15 = 48.6 KN $\therefore$ Live Load = 5*12.958 = 64.79 KN Total Load = 100.432 KN $\therefore$ UDL = (113.39/5.182) = 21.88 KN/m Floor Finish: 1. Screed (25mm) = 12.958*21*0.025 = 6.8 KN 2. Marble (15mm) = 27*12.958*0.015 = 5.25 KN Total floor finish load = 12.05 KN $\therefore$ UDL = 2.325 KN/m	

		Self wt. of beam = 25*0.23*(0.3-0.15)*5.182 = 4.469 KN ∴UDL = 4.469/5.182 = 0.863 KN/m Wall load on beam = 19*0.23*4.88*(3.302-0.3) = 64.019 KN ∴UDL = 12.354 KN/m Deduct 30% for opening ∴UDL = 12.354*0.7 = 8.648 KN/m Hence, total UDL = 25.068 KN/m And Ultimate UDL (W_u) = 25.068*1.5 = 37.602 KN/m Ultimate Moment (M_u) = $W_u l^2/10 = \frac{37.602 * 5.182^2}{10}$ = 100.973 KN-m Also, we have $M_u = 0.138 f_{ck} b d^2$ $\therefore d = \sqrt{\frac{M_u}{0.138 f_{ck} b}} = \sqrt{\frac{100.973 * 10^6}{0.138 * 20 * 230}}$ = 398.82 mm ≈ 435 mm	
IS 456-2000 Annex G			
	3	**Beam No. 2** For l_e = 14' = 4.267m $B_f = \frac{l_0}{6} + b_w + 6D_f = \frac{0.7 * l_e}{6} + 230 + 6 * 135$ = 1537.8 mm = 1.538 m $D = \frac{l}{10}$ to $\frac{l}{15}$ = 284.467 mm $d \geq l/26$ = 164.115 mm According to deflection criteria; $l/d \leq \alpha\beta\gamma\delta\lambda$ where; α = 26 (for continuous) β = 1 (span < 10m) γ = 0.82 for 2% tensile steel σ =1.08 for Pc = 0.25% $\lambda = 0.8$ for $\frac{b_w}{b_f} = \frac{230}{1735} = 0.133$ $\therefore d \geq \frac{4.267 * 10^3}{26 * 0.82 * 1.08 * 0.8}$ = 232.644 mm ≈ 232 mm ∴Adopt D = (232+25+8) mm = **265 mm**	
IS 456-2000 Clause 23.1.2			
IS 456-2000 Clause 23.2.1			

4 **Check by Moment Criteria:**

Total area of trapezoid = ½(a+b)*h

$\therefore A_1$ = ½*(4.267) * (1/2)*(4.267) = 4.552 m^2 (triangular)

& A_2 = 4.342 m^2 (trapezoidal)

Total Area = 8.894 m^2

Slab:

Live Load = 4 KN/m^2*8.894 m2 = 35.576 KN

Dead Load = 25*8.894*0.135 = 30.017 KN

Total Load = 65.6 KN

$\therefore$ UDL = 65.6/4.267 KN/m = 15.374 KN/m

Floor Finish:

Marble (15 mm) = 27*8.894*0.015 = 3.602 KN

Screed (25 mm) = 21*8.894*0.025 = 4.669 KN

Total floor finish load = 8.271 KN

$\therefore$ UDL = 8.271/4.267 = 1.938 KN/m

Self wt. of Beam = 25*0.23*(0.265-0.135)*4.267

= 3.190 KN

$\therefore$ UDL = 0.748 KN/m

Wall Load on beam = 19*0.23*3.962*(3.302-0.265)

= 52.582 KN

$\therefore$ UDL = 52.582/4.267 = 12.323 KN/m

Effective UDL (Deducting 30%) = 8.626 KN/m

Hence, total UDL = 26.686 KN/m

And Ultimate UDL (W_u) = 26.686*1.5 = 40.029KN/m

Ultimate Moment (M_u) = $W_u l^2 / 10$ = 72.882 KN-m

Also, we have $M_u = 0.138 f_{ck} b d^2$

IS 456-2000 Annex G

$$\therefore d = \sqrt{\frac{M_u}{0.138 f_{ck} b}} = 338.838 \text{ mm} \approx 340 \text{ mm}$$

Hence, Adopt **D = 450 mm**

COLUMN DESIGN

Reference	Step	Calculation	Remarks
	1	**Column Design**	
		Here,	
		Area = 19.509 m^2	
		Load Calculation	
		Slab	
		Live Load = 5 KN/m^2 8 19.509 m^2 = 97.545 KN	
		Dead Load = 19.509*25*0.15 + 73.159 KN	
		Dead Load of Beam = (4.267+4.572)*25*(0.435-0.15)*0.23 = 14.48 KN	
		Wall Load = 19*(4.267+4.572)*.23*(3.302-0.48)	
		= 109.004 KN	
		After deducting 30% + 0.7*109.004 = 76.303 KN	
		Floor Finish	
		15 mm Marble = 27*19.509*0.015 = 7.901 KN	
		25 mm Screed = 21*19.509*0.025 = 10.242 KN	
		Hence, Total Load (P) = 279.634 KN	
		Total load including self wt. = 1.1*279.634 = 307.6 KN	
		And Ultimate load (P_u) = 1.5*307.6 = 461.400 KN	
		For four storey (P_u) = 4*461.400 = 1845.6 KN	
		Then,	
		We have,	
IS 456-2000		$P_u = 0.4f_{ck}A_c + 0.67f_yA_{st}$ (for Fe415)	
Clause 39.3		$=0.4*20*(A_g-A_{st}) + 0.67*415*A_{st}$	
		$1845.6*10^3 = 0.4*20*(A_g-\frac{2}{100}A_g) +0.67*415*2/100*A_g$	
		$\therefore A_g = 137721.065$ mm^2	
		= 371.1078*371.1078	
		Hence, Adopt = **450 mm*450 mm**	
	2	**Check for % of Steel**	
		$1845.6*10^3 = 0.4*20\ (450*450 - A_{st}) + 0.67*415*A_{st}$	
		$\therefore A_{st} = 1927.8$ mm^2	
		% of steel = $\frac{1927.8}{3752}$ * 100 % = 1.37 %	
		$\therefore$0.8% < 1.37% < 4% **So, OK**	

STAIRCASE DESIGN

Reference	Step	Calculation	Remarks
	1	**Staircase Design** General considerations: For office buildings: Width of Stairs (b) = 1800 to 2400 mm Trade (T) = 270 to 300 mm Then, we have, 2*R + T = 600 to 640 mm And R*T = 40000 to 42000 mm^2 Now, We have, Floor height = 10'10" = 3.302 m No. of Riser = 18 ∴ Riser (R) = Floor ht./No. of Riser = 3.302/18 = 183 mm Trade (T) = 12" = 304.8 mm For span AB (L_e) = 11' = 3.353 m Let, Waist slab thickness (D) = Span/20 = 3.353/20 = 167.65 mm Hence, **Adopt D = 175 mm** Also $\sqrt{R^2 + T^2} = \sqrt{183^2 + 304.8^2} = 356$ mm	
	2	**For Inclined portion** 1) Self wt. of slab = (D*0.356)*25 = (0.175*0.356)*25 = 1.56 KN/m = 1.56/0.3048 = 5.12 KN/m^2 2) Self wt. of steps = ½ 0.183*0.3048*25 = 0.7 KN/m = 0.7/0.3048 = 2.3 KN/m^2 3) Finishes = 1 KN/m^2 4) Live Load = 5.0 KN/m^2 (for office & public building) Hence, Total load = 13.42 KN/m^2 Taking 1 m width of flight W = 1*13.42 = 13.42 KN/m W_u = 1.5*13.42 = 20.13 KN/m (Per m width)	

3 **For Landing zone**

1) Self wt. = 0.175*1/1*25 = 4.375 KN/m^2
2) Finishes = 1 KN/m^2
3) Live load = 5.0 KN/m^2

Total load = 10.375 KN/m^2

W = 10.375 KN/m (per m width)

Ultimate load (W_u) = 1.5*10.375 = 15.56 KN/m

Now,

$R_A + R_B$ = 15.56*1.524 +20.13*1.183

$R_A + R_B$ = 47.53 KN …………(1)

Taking Moment about B (clockwise positive)

R_A (1.524+1.183)-15.56*1.524(1.183+1.524/2) – 20.13*1.183*1.183/2 = 0

∴ R_A = 22.24 KN

$R_A + R_B$ = 47.53

∴ R_B = 47.53-22.24 = 25.288 KN

For the point of Maximum Bending Moment,

R_A –15.56*1.524 – 20.13(x=1.524) = 0 ⇒ (x>1.524m)

22.24 – 15.56*1.524 – 20.13(x – 1.524) = 0

∴ x = 1.45 m

Moment at (x = 1.45)

M_x = 22.24*1.45 = 15.56*1.45*1.45/2

∴ M_x = 15.89 KN-m

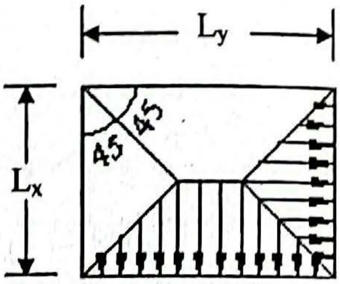
$M_u = 0.138\ f_{ck}\ bd^2$

$18.89*10^6 = 0.138*20*1000*d^2$

∴ d = 75mm < 175 mm **So, OK**

OUTPUTS

Slab (Seminar hall) thickness	=	150 mm
Slab (Others) thickness	=	135 mm
Beam	=	230 mm x 450 mm
Column	=	450 mm x 450 mm
Staircase (waist slab)	=	175 mm



2.2 VERTICAL LOADS

DEAD LOAD

- Dead load from slab is trapezoidal acts on beams
- Dead load from walls are UDL also acts on beams
- Self wt. of beam is UDL and acts on the same beam
- Self wt. of columns are considered point loads action on joints
- Dead load intensity of floor slab
  = Unit wt. of concrete*thickness [RCC + floor finishes (Marble +screed)]
  = 4.68 KN/m^2 (Seminar hall)
  = 4.305 KN/m^2 (Others)
- Dead load intensity of roof slab
  = Unit wt. of concrete*thickness [RCC + floor finishes (Plaster)]
  = 4.3 KN/m2
  - Unit wt. of different materials (γ)
    - $\gamma_{Reinforced\ Concrete}$ = 25 KN/m^3
    - γ_{marble} = 27 KN/m^3
    - γ_{screed} = 21 KN/m^3
    - $\gamma_{plaster}$ = 22 KN/m^3
    - γ_{brick} = 19 KN/m^3
    - $\gamma_{Cement\ Concrete}$ = 24 KN/m^3

LIVE LOAD

- Live load is acted directly on slab
- In structural modeling it is considered trapezoidal loads in beams supporting the slab
- Intensity of Live load in slab differ with type of room

For e.g.	Office room	=	2.5 KN/m^2
	Cafeteria	=	3.0 KN/m^2
	Seminar hall	=	3.0 KN/m^2
	Staircase	=	4.0 KN/m^2
	Storage room	=	5.0 KN/m^2
	Toilet	=	2.0 KN/m^2

IS 875 (Part 2) – 1987 (Code of practice for Design Loads for building & structure)

2.3 HORIZONTAL LOADS

While analyzing, we are concerned with various types of vertical and horizontal/lateral loads. Earthquake load as a lateral load should be considered during design in order to meet the acceptable level of responses. The main objective of design is to emphasize the probable deformation or response to their considerable limit within their lifetime. The structural elements should sustain entire possible loads within the suitable degree of safety, upgrade the durability of structure, and be resistive towards misuse and fire.

(A) EARTHQUAKE LOAD

For the lateral load calculation, the ***lumped weight*** of each floor (including basement) is determined. Due to the ***moderate building (<40 m), Seismic Coefficient method*** is followed for an ease (Referring IS: 1893 – 1975)

LUMPED WEIGHT CALCULATION (Floor-wise)

) GROUND FLOOR:

1) BEAM:

Grid No.	*Length (m)*	*Grid No.*	*Length (m)*
A-A	32.613	1-1	8.149
B-B	32.163	2-2	19.042
C-C	32.163	3-3	19.042
D-D	36.576	4-4	19.042
Inclined	4.572	5-5	19.042
E-E	4.915	6-6	13.48
F-F	16.154	7-7	13.48
G-G	16.154	8-8	10.967
		9-9	10.967
		10-10	10.967
Total	**176.21**		**144.178**

Total length of beam: 176.21 + 144.178 = 320.388

Dead Load of Beam: 0.375*0.23*320.388*25 = **690.836 KN**

(Unit wt. of concrete = 25 KN/m^3)

2) **COLUMN:**

Dead Load of Column: 49*0.375*0.375*3.054*25 = **526.10 KN**

3) **WALL:**

Ht. of lower half of ground floor = 1.651m

a) Wall having opening (10' wall) = 72.23m

Reducing 30% opening = 72.23*0.7 = 50.57

b) Wall without opening (including interior wall)

i) 5' wall = 25.908m

ii) 10' wall = 173.73m

Dead Load (DL):

a) DL = 50.57*0.254*1.651*19 = 402.92 KN

b) i) DL = 25.908*0.127*1.651*19 = 103.21 KN

ii) DL = 173.73*0.254*1.651*19 = 1384.23 KN

Total = 402.92+103.21+1384.23

= 1890.36 KN

Ht. of upper half of basement = 1.643m

a) Wall having opening (10' wall) = 63.08 m

Reducing 30% opening = 63.08*0.7 = 44.16

b) Wall without opening (10' wall) = 139m

Total length = 139+44.16 = 183.16 m

Dead Load (DL):

(a &b) DL = 183.16*0.254*1.643*19 = 1452.29 KN (Due to 10' wall of both cases)

Hence, total wall load = **1890.36+1452.29 = 3342.65 KN**

4) **SLAB:**

a) **Dead Load** = Total area*Depth*Unit wt.

= 586.527*0.135*25

= **1979.52 KN**

b) **Live Load** = Σ (Intensity of individual room as per code * Internal area of slab excluding area of beam & column) [Take 50% of Live Load for value ≥3 KN/m^2 and 25% of Live Load for the value < 3KN/m^2]

= **836.668 KN**

5) **STAIRCASE:**

Dead Load of Staircase = [(UDL of inclined portion which is converted into its horizontal equivalent*Width*2) + (UDL of landing portion*Width*2)] * (No. of staircase well)

= [(20.13*1.524*2) + (15.56*1.524*2)]*2 = **217.566 KN**

6) **FLOOR FINISH:**

a) Marble = Total area*thickness*Unit wt. of marble (γ_{marble} = 27 KN/m^3)

= 526.206*0.015*27

= 213.11 KN

b) Screed = Total area*thickness*Unit wt. of screed (γ_{screed} = 21 KN/m^3)

= 526.206*0.025*21

= 276.25 KN

Total Floor finish (DL) = 213.11 + 276.25 = **489.36 KN**

HENCE, TOTAL LOAD ON GROUND FLOOR (W_0) = 8082.72 KN

B) FIRST FLOOR:

1) **BEAM:**

Grid No.	*Length (m)*	*Grid No.*	*Length (m)*
A-A	32.613	1-1	8.149
B-B	32.163	2-2	19.042
C-C	32.163	3-3	19.042
D-D	36.576	4-4	19.042
Inclined	4.572	5-5	19.042
E-E	4.915	6-6	13.48
F-F	16.154	7-7	13.48
G-G	16.154	8-8	10.967
		9-9	10.967
		10-10	10.967
Total	**176.21**		**144.178**

Total length of beam: 176.21 + 144.178 = 320.388

Dead Load of Beam: 0.375*0.23*320.388*25 = **690.836 KN** (Unit wt. of concrete = 25 KN/m^3

2) COLUMN:

Dead Load of Column: 49*0.375*0.375*(1.276+1.651)*25 = **504.22 KN**

3) WALL:

Ht. of lower half of first floor = 1.651m

a) Wall having opening (10' wall) = 77.72 m

Reducing 30% opening = 77.72*0.7 = 54.41m

b) Wall without opening (including interior wall)

i) 5' wall = 25.908 m

ii) 10' wall = 179.22 m

Dead Load (DL):

a) DL = 54.41*0.254*1.651*19 = 433.523 KN

b) DL = (5' wall) = 25.908*1.651*0.127 *19 = 103.21 KN

= (10' wall) = 179.22*0.254*1.651*19 = 1427.97 KN

Total = 433.523+103.21+1427.97

= 1964.703 KN

Ht. of upper half of basement ⇒ For 10' wall = 1.276 m

For 5' wall = 1.516 m

a) DL = 50.57*1.276*0.254*19 = 311.41 KN

b) DL = (5' wall) = 25.908*1.516*0.127*19 = 94.77 KN

= (10' wall) = 173.73*1.276*0.254*19 = 1069.83 KN

Total = 311.41+94.77+1069 = 1475.94 KN

Hence, total wall load = 1964.703+1475.94 = **3440.713 KN**

4) SLAB:

a) Dead Load = (Total area + slope area)*Depth*Unit wt.

= (1979.52*0.135*25) + (20.41*0.1*25) [Depth of slope portion = 0.1 m = 4']

= **2030.63 KN**

b) Live Load = Σ (Intensity of individual room as per code * Internal area of slab excluding area of beam & column) [Take 50% of Live Load for value ≥3 KN/m^2 and 25% of Live Load for the value < 3KN/m^2]

= **767.70 KN**

5) **STAIRCASE:**

Dead Load of Staircase = [(UDL of inclined portion which is converted into its horizontal equivalent*Width*2) + (UDL of landing portion*Width*2)] * (No. of staircase well)

= [(20.13*1.524*2) + (15.56*1.524*2)]*2 = **217.566 KN**

6) **FLOOR FINISH:**

a) Marble = Total area*thickness*Unit wt. of marble (γ_{marble} = 27 KN/m^3)

= 524.735*0.015*27

= 212.52 KN

b) Screed = Total area*thickness*Unit wt. of screed (γ_{screed} = 21 KN/m^3)

= 524.735*0.025*21

= 275.486 KN

Total Floor finish (DL) = 212.52 + 275.486 = **488.006 KN**

HENCE, TOTAL LOAD ON FIRST FLOOR (W_1) = 8139.671KN

:) **SECOND FLOOR:**

1) **BEAM:**

Length of Seminar hall = 121.890 m

Length of Others (except Seminar hall) = 320.388-121.89 = 198.498 m

Total Dead Load of Beam = (121.890*0.435*0.23*25) + (198.498*0.375*0.23*25)

= **732.888 KN**

2) **COLUMN:**

Dead Load of below Seminar hall column = 18*0.375*0.375*(1.651-0.435)*25

= 76.95 KN

Dead Load of Others (except Seminar hall) = 31*0.375*0.375*(1.651-0.375)*25

= 139.064 KN

Dead load for upper part of second floor = 45*0.375*0.375*1.651*25 = 261.193 KN

Total Dead Load of Column = 201.58 +322.88 = **477.207 KN**

3) **WALL:**

Ht. of upper half of first floor = For Seminar hall = 1.216 m

For Others = 1.276 m

Dead Load = (102.96*0.254*1.216*19) + (156.58*0.254*1.276*19)

= 1568.42 KN

Ht. of lower half of second floor =1.651 m (same for both seminar hall and others)

Dead Load for others (except seminar hall) = (259.528-56)*19*0.254*1.651

= 1621.65 KN

Hence, total wall load = 1568.42+1621.65 = **3190.07KN**

4) **SLAB:**

a) **Dead Load** = (Total area + slope area)*Depth*Unit wt.

= (1979.52*0.135*25) + (20.41*0.1*25) [Depth of slope portion = 0.1 m = 4']

= **2030.63KN**

b) **Live Load** = Σ (Intensity of individual room as per code * Internal area of slab excluding area of beam & column) [Take 50% of Live Load for value ≥3 KN/m^2 and 25% of Live Load for the value < 3KN/m^2]

= **988.47 KN**

5) **STAIRCASE:**

Dead Load of Staircase = [(UDL of inclined portion which is converted into its horizontal equivalent*Width*2) + (UDL of landing portion*Width*2)] * (No. of staircase well)

= [(20.13*1.524*2) + (15.56*1.524*2)]*2 = **217.566 KN**

6) **FLOOR FINISH:**

a) Marble = Total area*thickness*Unit wt. of marble (γ_{marble} = 27 KN/m^3)

= 525.139*0.015*27

= 212.68 KN

b) Screed = Total area*thickness*Unit wt. of screed (γ_{screed} = 21 KN/m^3)

= 525.139*0.025*21

= 275.70 KN

Total Floor finish (DL) = 212.68 +275.70 = **488.378 KN**

HENCE, TOTAL LOAD ON SECOND FLOOR (W_2) = 8125.209 KN

D) THIRD FLOOR:

1) BEAM:

Grid No.	*Length (m)*	*Grid No.*	*Length (m)*
A-A	32.613	1-1	8.149
B-B	32.163	2-2	19.042
C-C	32.163	3-3	19.042
D-D	36.576	4-4	19.042
Inclined	4.572	5-5	19.042
E-E	4.915	6-6	13.48
F-F	16.154	7-7	13.48
G-G	16.154	8-8	10.967
		9-9	10.967
		10-10	10.967
Total	**176.21**		**144.178**

Total length of beam: 176.21 + 144.178 - 50.902 = 269.486 m (50.902 is the cumulative length below steel truss)

Dead Load of Beam: 0.375*0.23*269.486*25 = **581.079 KN**

(Unit wt. of concrete = 25 KN/m^3)

2) COLUMN:

Total dead load of column = (49-4)*0.375^2*25*Height of upper half of second floor

= 45*0.375^2*(3.302/2 – 0.375)*25

= **201.86 KN**

3) WALL:

Ht. of upper half of second floor = 1.276 m

Dead Load = (259.528-56)*19*0.254*1.276 = <u>1253.32 KN</u>

Parapet Wall = Length*height*thickness*unit wt.

= 55.78*0.914*0.23*19 = <u>222.79 KN</u>

Slope portion of parapet wall = 38.70*0.1*2.2*25 = <u>212.903 KN</u>

Hence, total wall load = **1689.013 KN**

4) **SLAB:**

a) **Dead Load:** Total area (deducting Seminar hall area and staircase well)*thickness*unit wt.

= 332.489*0.135*25 = **1122.150 KN**

b) **Live Load:** Total area (deducting Seminar hall area and staircase well)*intensity in KN/m^2

= 1.5*332.489 = **498.73 KN**

5) **STAIRCASE:**

Dead Load of Staircase = [(UDL of inclined portion which is converted into its horizontal equivalent*Width) + (UDL of landing portion*Width)] * (No. of staircase well)

= [(20.13*1.524) + (15.56*1.524)]*2

= **108.78 KN**

6) **FLOOR FINISH:**

Terrace: Screed = Area of screed*unit wt.*Thickness ($\gamma_{screed} = 21\ KN/m^3$)

= 332.489*21*0.025 = 174.55 KN

Panning (10mm) = 0.001*22*332.489 = 7.314 KN ($\gamma_{panning} = 21\ KN/m^3$)

Total Floor finish (DL) = **181.864 KN**

7) **STAIRCASE WELL:**

Dead load of column = 8*2*0.327*0.375*2.6924/2*25 = **37.86 KN**

Dead load of wall (including reduction) = [(4*3.048*0.7)+(4*3.048)]*19*0.254*2.6924/2

= **134.65 KN**

8) **STEEL TRUSS:**

Total load of steel truss = (live load + dead load) intensity * area

= (0.15 + 0.75) * 176.887

≈ 1 *176.887 = **176.887 KN**

HENCE, TOTAL LOAD ON THIRD FLOOR (W_3) = 4234.14 KN

C) STAIRCASE WELL:

Total dead load of column = 4*2*0.375*0.375*(2.6924/2 – 0.33)*25 = **28.58 KN**

Total dead load of wall = (2.6924/2 – 0.2286)*0.23*20.72*19 = **101.19 KN**

Total dead load of beam = 0.2286*0.2286*13.41*25 = **17.52 KN**

Total dead load of slab = 26.756*2*0.135*25 = **180.603 KN**

TOTAL LOAD (W_4) = 327.89 KN

Since, $W_4 < 25\%$ of W_3, So W_4 is lumped within W_3

HENCE, TOTAL LOAD ON THIRD FLOOR (W_3) = (4234.14 + 327.89) = 4562.03 KN

BASEMENT (Lower portion):

COLUMN:

Dead Load = 49*1.829*0.375*0.375*25

= **315.07 KN**

WALL:

a) **Wall (external)** = 57.408*1.829*0.254*25 = 664.92 KN ($\gamma_{\text{concrete wall}} = 25\ KN/m^3$)

b) **Wall (internal)** = 139*1.824*0.254*19 = 1223.56 KN ($\gamma_{\text{brick wall}} = 19\ KN/m^3$)

Hence, total wall load = **1888.48 KN**

TOTAL LOAD ON LOWER PORTION OF BASEMENT ($W_{Basement}$) = 2203.55 KN

TABLE-1 RESULT TABLE

<table>
<tr><th rowspan="2">Floor</th><th rowspan="2">Beam</th><th rowspan="2">Column</th><th rowspan="2">Wall</th><th colspan="2">Slab</th><th rowspan="2">Staircase</th><th colspan="2" rowspan="2">Floor finish</th><th rowspan="2">TOTAL</th></tr>
<tr><th>DL</th><th>LL</th></tr>
<tr><td>Ground Floor</td><td>690.836</td><td>526.10</td><td>3342.65</td><td>1979.52</td><td>836.668</td><td>217.566</td><td colspan="2">489.36</td><td>8082.72</td></tr>
<tr><td>First Floor</td><td>690.836</td><td>504.22</td><td>3440.713</td><td>2030.63</td><td>767.70</td><td>217.566</td><td colspan="2">488.006</td><td>8139.671</td></tr>
<tr><td>Second Floor</td><td>732.888</td><td>477.207</td><td>3190.07</td><td>2030.63</td><td>988.47</td><td>217.566</td><td colspan="2">488.378</td><td>8125.203</td></tr>
<tr><td>Third Floor</td><td>581.079</td><td>201.86</td><td>1689.013</td><td>1122.150</td><td>498.73</td><td>108.78</td><td>181.864</td><td>Staircase well:
Column = 37.86 KN
Wall = 134.65 KN
+
Steel
truss = 176.887 KN
+
W_4 = 327.89 KN</td><td>4562.033</td></tr>
<tr><td colspan="9">Mass lumped in Basement</td><td>2203.55</td></tr>
<tr><td colspan="9">TOTAL (W)</td><td>31113.17</td></tr>
</table>

HORIZONTAL BASE SHEAR CALCULATION

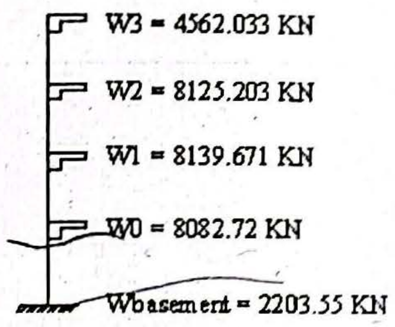


Fig. MASS LUMPED AT FLOOR LEVEL

Then, the ***horizontal base shear*** is determined,

It is given by

$V_b = A_h * W$

Where, V_b = Horizontal Base Shear

A_h = Design horizontal seismic Coefficient

W = Total lumped weight of building

Again, $A_h = \frac{ZI}{2R} * \frac{S_a}{g}$

Where, Z = Zone factor (Table 2) = Zone 5 = 0.36 (= very severe)

I = Importance factor (Table 6) = 1.5 (=Community building)

R = Response Reduction factor (Table)

= 5.0 (= special RC moment resisting frame)

S_a/g = Average Response Acceleration Coefficient

= Depends upon Time period (T)

= T = Fundamental natural period

Also we have, $T = 0.75h^{0.75}$ where, h = ht. of building = 13.512 m

$\therefore$ T = 0.52859 (lies between 0.55 and 4.00)

Hence, $S_a/g = 2.5$

And, $A_h = \frac{0.36 * 1.5}{2 * 5} * 2.5 = 0.135$

Lastly, Horizontal base shear (V_b) = A_h*W = 0.135*31113.17 = **4200.28 KN**

After determining the Horizontal base shear, the distribution of lateral (earthquake) load on different floor is determined as follows:

$$Qi = \frac{W_i h_i^2}{\sum W_i h_i^2} * V_b$$

where, Q_i = Horizontal/Lateral force acting at any i^{th} floor

W_i = Lumped weight of i^{th} floor

h_i = Ht. of each floor above base

V_b = Horizontal base shear

TABLE-2 CALCULATION OF SEISMIC LOAD (EQL)

Base Shear $V_b = A_h * W = 0.135*31113.17 =$ **4200.28 KN** **and** $Qi = (W_i h_i^2 / \Sigma W_i h_i^2) * V_b$

Floor	Mass of each floor (W_i)	Cum. Height from ground (h_i)	$W_i h_i^2$	Q_i (KN)	Remarks
Ground Floor	8082.72	3.606	105101.52	**203.11**	
First Floor	8139.671	6.908	388428.88	**750.66**	
Second Floor	8125.203	10.21	847004.47	**1636.87**	
Third Floor	4562.033	13.512	832909.27	**1609.64**	
			Σ = 2173444.14		

(All values are in KN)

SEISMIC LOAD DISTRIBUTION

The lateral seismic load obtained from the calculation of lumped mass at floor level and horizontal base shear is then distributed within the building in both X and Y direction. While distributing in each grid, the lateral load in each column is obtained by dividing total lateral load in a floor to the no. of column in that floor. Then, at the time of distribution grid wise, no. of column is counted in each grid and it is multiplied by lateral load in each column. The magnitude of seismic lateral load is same in both positive and negative direction.

Lateral Load on each column = (Qi/No. of column)

Ground floor = 203.11/49 = 4.145 KN

First Floor = 750.66/49 = 15.32 KN

Second Floor = 1636.87/49 = 33.40 KN

Third Floor = 1609.64/49 = 35.77 KN

TABLE-3 SEISMIC LOAD DISTRIBUTION ON GROUND FLOOR

X-Direction				Y-Direction			
Grid No.	**No. of Column**	**Load on each Column**	**Lateral Load**	**Grid No.**	**No. of Column**	**Load on each Column**	**Lateral Load**
A-A	9	4.145	37.305	1-1	3	4.145	12.435
B-B	9	4.145	37.305	2-2	6	4.145	24.87
C-C	9	4.145	37.305	3-3	6	4.145	24.87
D-D	10	4.145	41.45	4-4	6	4.145	24.87
E-E	2	4.145	8.29	5-5	6	4.145	24.87
F-F	5	4.145	20.725	6-6	5	4.145	20.725
G-G	5	4.145	20.725	7-7	5	4.145	20.725
				8-8	4	4.145	16.58
				9-9	4	4.145	16.58
				10-10	4	4.145	16.58
Total	**49**		**203.11**		**49**		**203.11**

(For both EQL-Positive and EQL-Negative)

TABLE-4 SEISMIC LOAD DISTRIBUTION ON FIRST FLOOR

X-Direction				Y-Direction			
Grid No.	No. of Column	Load on each Column	Lateral Load	Grid No.	No. of Column	Load on each Column	Lateral Load
A-A	9	15.32	137.88	1-1	3	15.32	45.96
B-B	9	15.32	137.88	2-2	6	15.32	91.92
C-C	9	15.32	137.88	3-3	6	15.32	91.92
D-D	10	15.32	153.20	4-4	6	15.32	91.92
E-E	2	15.32	30.64	5-5	6	15.32	91.92
F-F	5	15.32	76.60	6-6	5	15.32	76.60
G-G	5	15.32	76.60	7-7	5	15.32	76.60
				8-8	4	15.32	61.28
				9-9	4	15.32	61.28
				10-10	4	15.32	61.28
Total	**49**		**750.36**		**49**		**750.66**

(For both EQL-Positive and EQL-Negative)

TABLE-5 SEISMIC LOAD DISTRIBUTION ON SECOND FLOOR

X-Direction				Y-Direction			
Grid No.	No. of Column	Load on each Column	Lateral Load	Grid No.	No. of Column	Load on each Column	Lateral Load
A-A	9	33.40	300.60	1-1	3	33.40	100.20
B-B	9	33.40	300.60	2-2	6	33.40	200.40
C-C	9	33.40	300.60	3-3	6	33.40	200.40
D-D	10	33.40	334.0	4-4	6	33.40	200.40
E-E	2	33.40	66.80	5-5	6	33.40	200.40
F-F	5	33.40	167	6-6	5	33.40	167
G-G	5	33.40	167	7-7	5	33.40	167
				8-8	4	33.40	133.60
				9-9	4	33.40	133.60
				10-10	4	33.40	133.60
Total	**49**		**1637.87**		**49**		**1637.87**

(For both EQL-Positive and EQL-Negative)

TABLE-6 SEISMIC LOAD DISTRIBUTION ON THIRD FLOOR

X-Direction				Y-Direction			
Grid No.	No. of Column	Load on each Column	Lateral Load	Grid No.	No. of Column	Load on each Column	Lateral Load
A-A	9	35.77	321.93	1-1	3	35.77	107.31
B-B	9	35.77	321.93	2-2	5	35.77	178.85
C-C	8	35.77	286.16	3-3	3	35.77	107.31
D-D	9	35.77	321.93	4-4	6	35.77	214.62
E-E	2	35.77	71.54	5-5	6	35.77	214.62
F-F	3	35.77	107.31	6-6	5	35.77	178.85
G-G	5	35.77	178.85	7-7	5	35.77	178.85
				8-8	4	35.77	143.08
				9-9	4	35.77	143.08
					4	35.77	143.08
Total	**45**		**1609.64**		**45**		**1609.64**

(For both EQL-Positive and EQL-Negative)

CHAPTER-3

MODELING

3.1 INTRODUCTION

For the purpose of Seismic analysis of our building we have used the Structural Analysis Program (SAP 2000). It has a special option for modeling horizontal rigid floor diaphragm system. A floor diaphragm is modeled as a rigid horizontal plane parallel to global X-Y plane, so that all points on any floor diaphragm cannot displace relative to each other in X-Y plane. Initially, the characteristics of the materials used were defined such as concrete – M20 and reinforcement – Fe415 Then, the load cases as well as their combinations with load factors were introduced. Next, structures were analyzed for different load combinations and the final output was determined in the form of SF, BM and AF etc.

The concept of Finite Element Method is also used thoroughly. This method deals with the elementary analysis of any structure. While analyzing the building, it was divided into no. of structural elements. For example, during calculation of lumped mass, the whole building mass was calculated as the summation of masses at floor level. Similarly, various structural elements such as beam, slab, column, and foundations were analyzed in the elementary basis.

The building was restrained at the basement. During seismic analysis, while calculating base shear, the time period was dependent upon the height of the building. In our context, the height of basement storey was included due to presence of ventilation in the basement wall. If the whole basement storey was of RCC wall without ventilation, then it was possible to restrain at the ground level.

3.2 SAMPLE INPUT

Input figures (unreformed shape from SAP 2000)

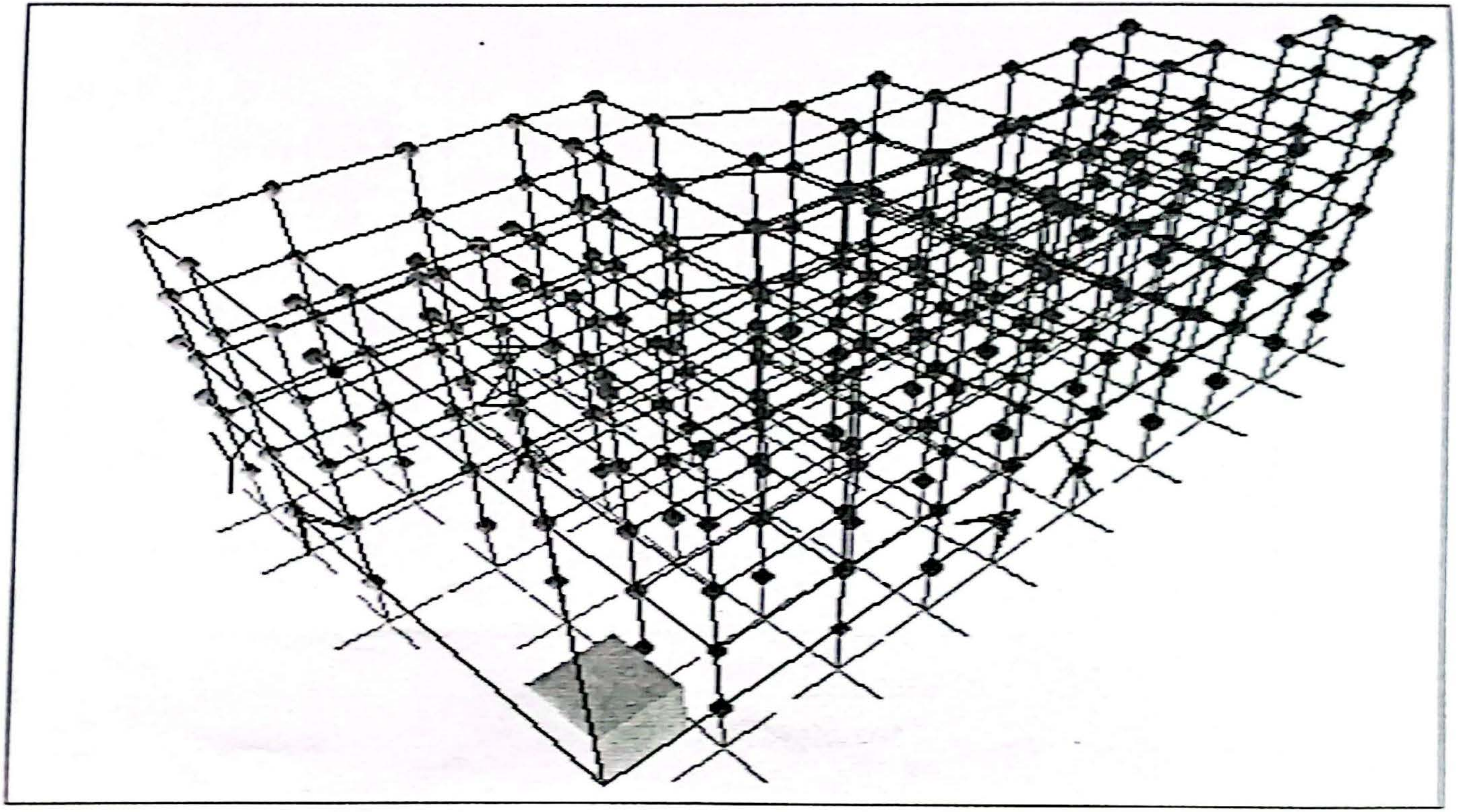


FIG 1. 3D FRAME STRUCTURE (UNDEFORMED)

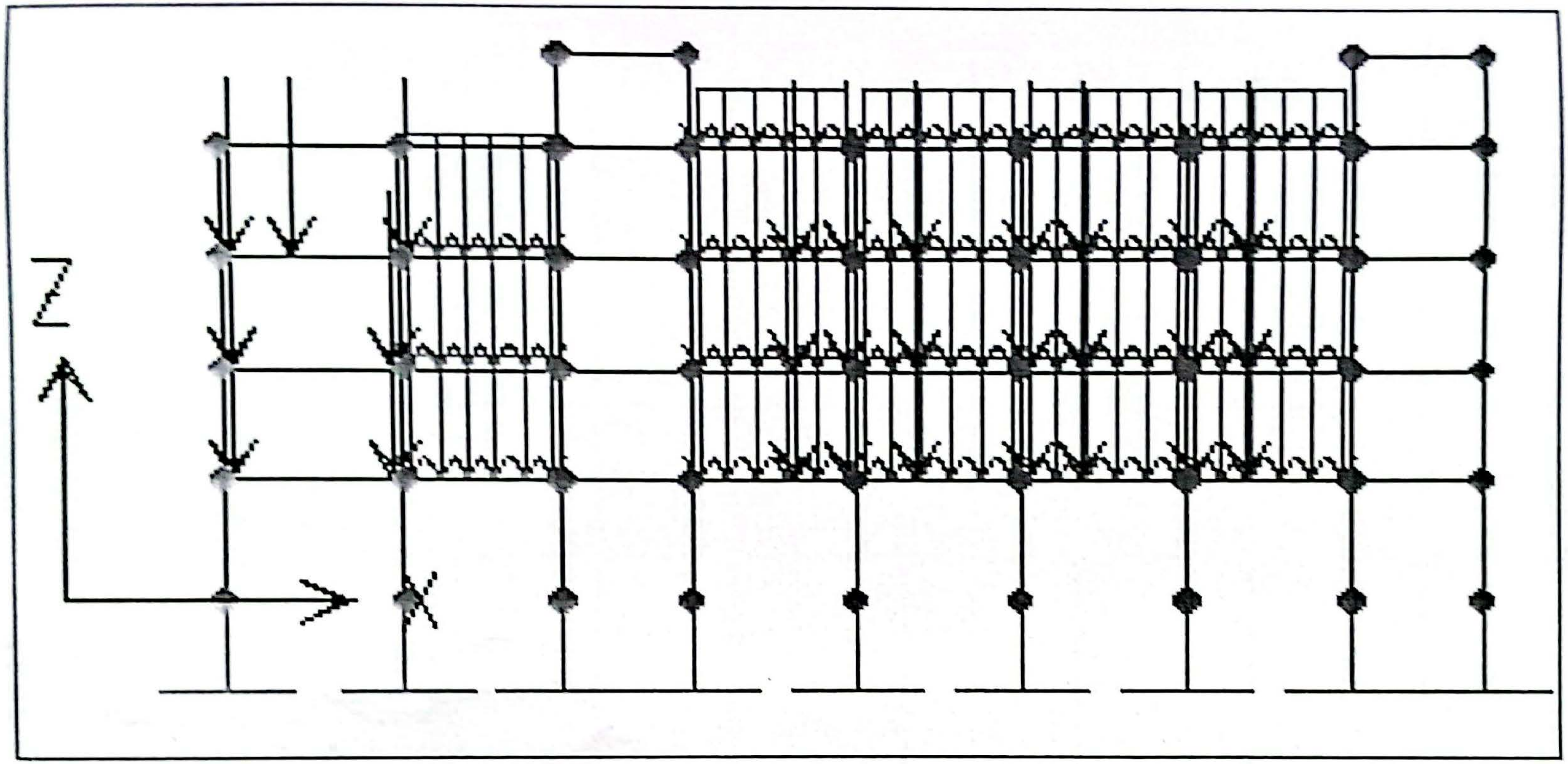


FIG 2. FRAME A-A (WALL LOAD)

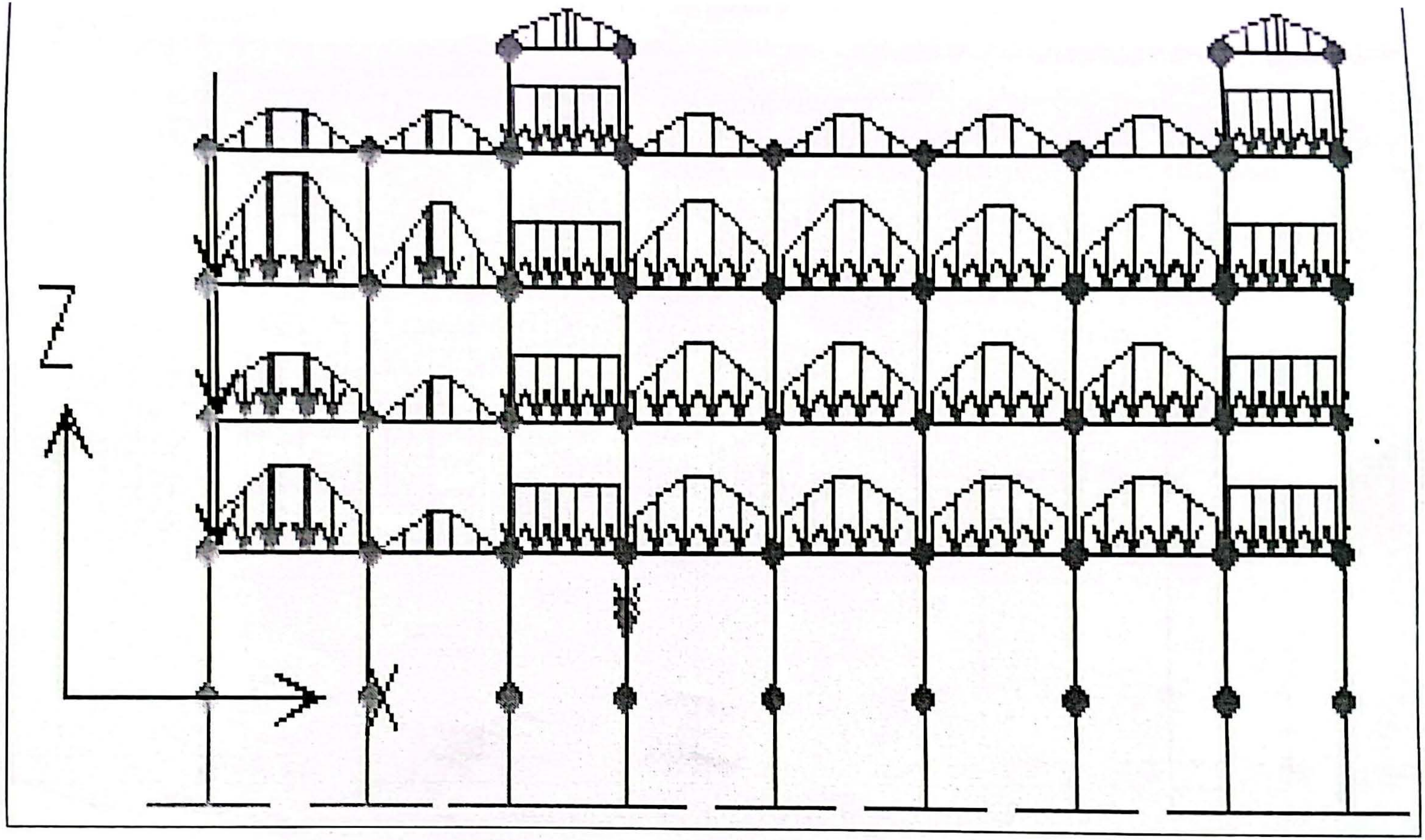


FIG 3. FRAME A-A (LIVE LOAD)

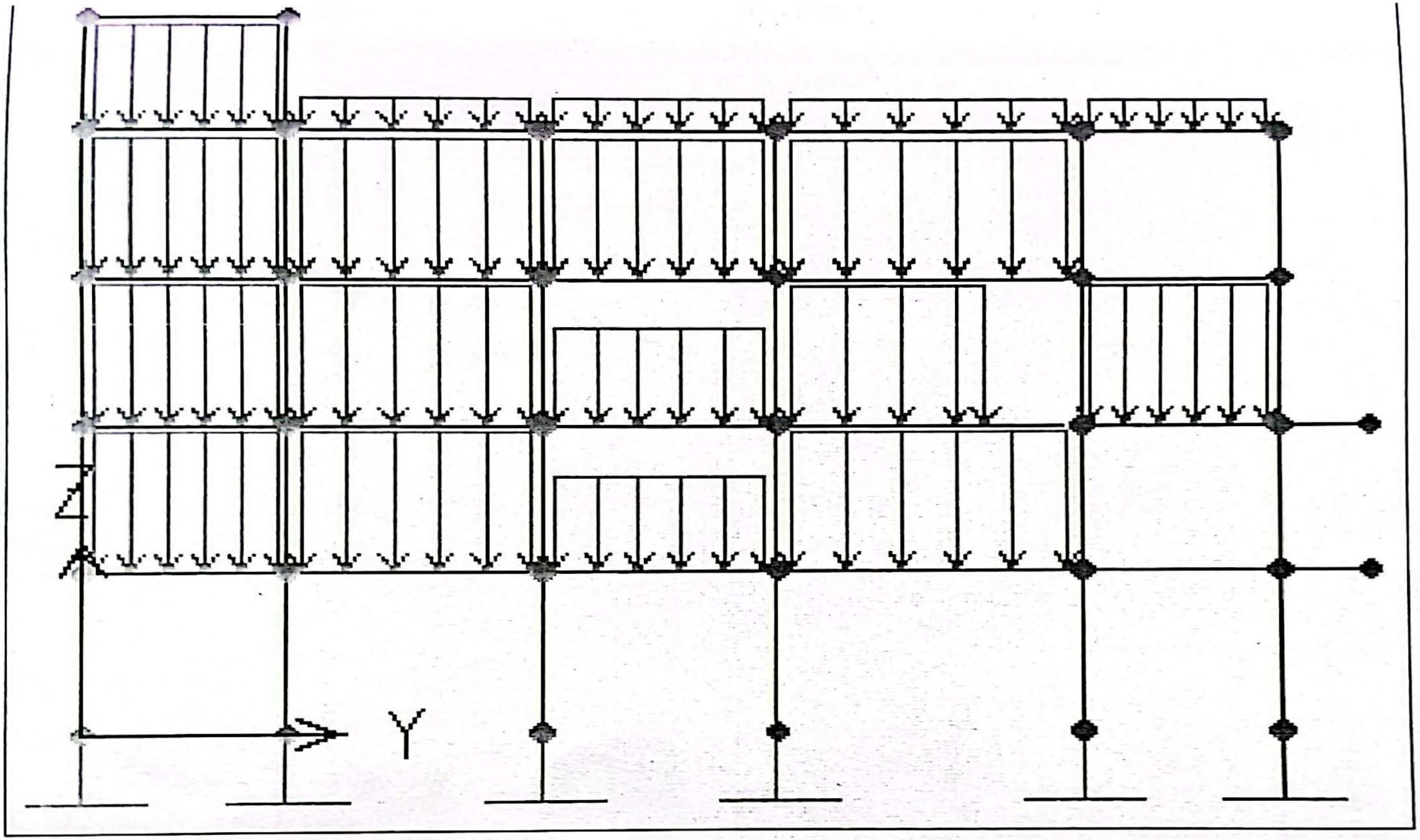


FIG 4. FRAME 4-4 (WALL LOAD)

3.2 SAMPLE OUTPUT

Deformed shape due to load cases

1) Dead Load

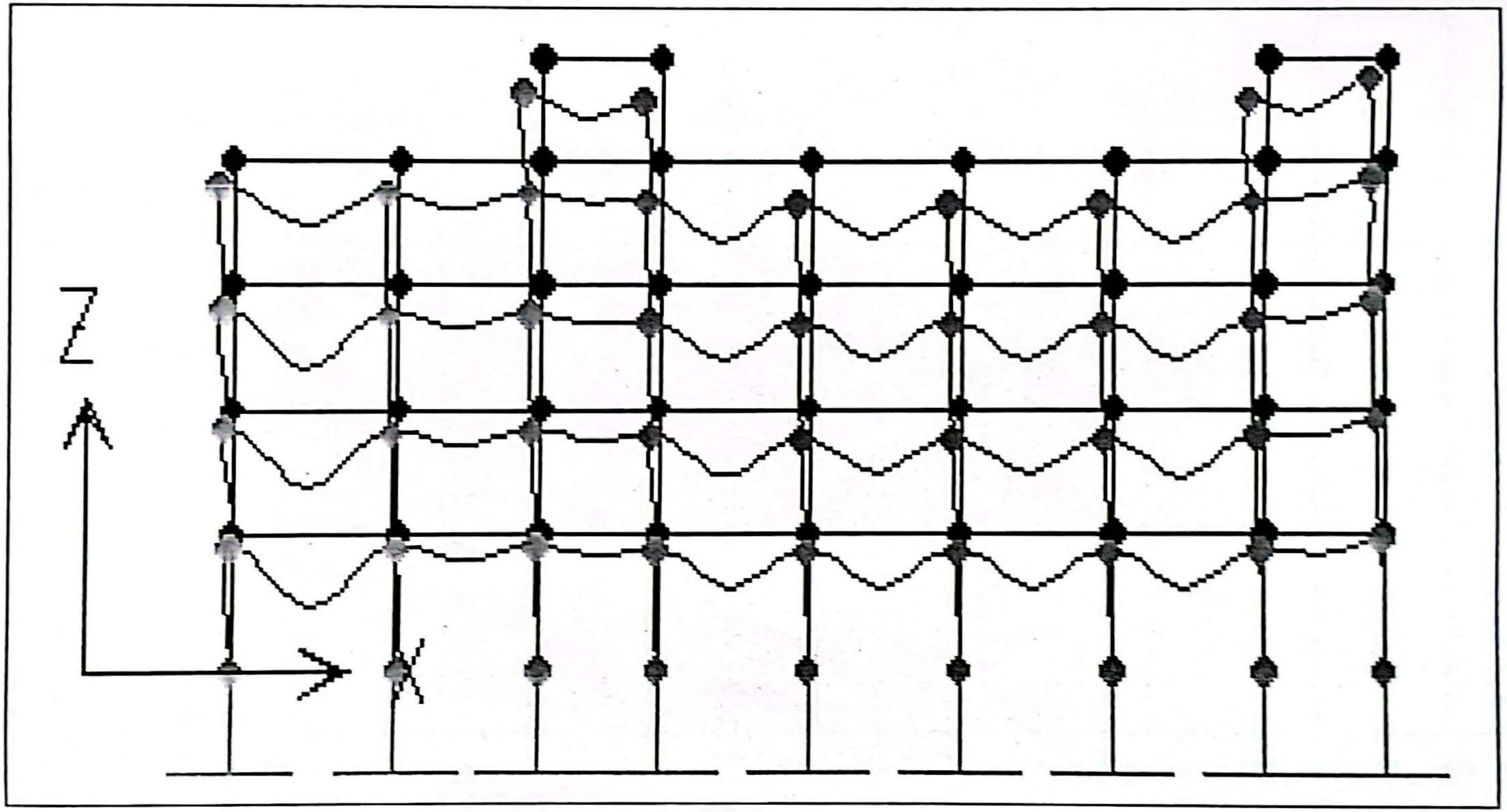


FIG 5. SHOWING DEFOMED SHAPE IN FRAME A-A DUE TO DEAD LOAD

2) Live Load

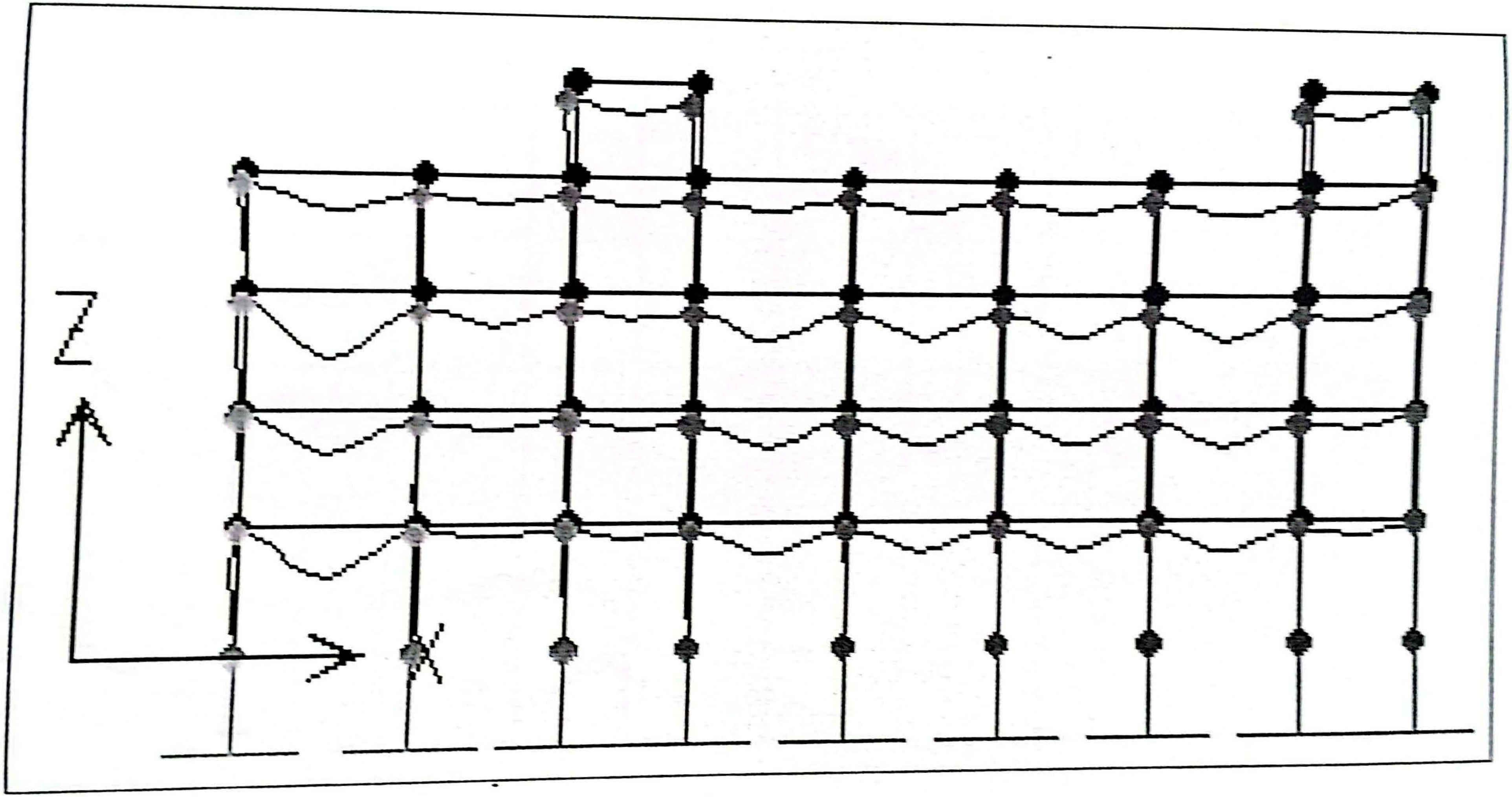


FIG 6. SHOWING DEFORMED SHAPE IN FRAME A-A DUE TO LIVE LOAD

3) EQPX

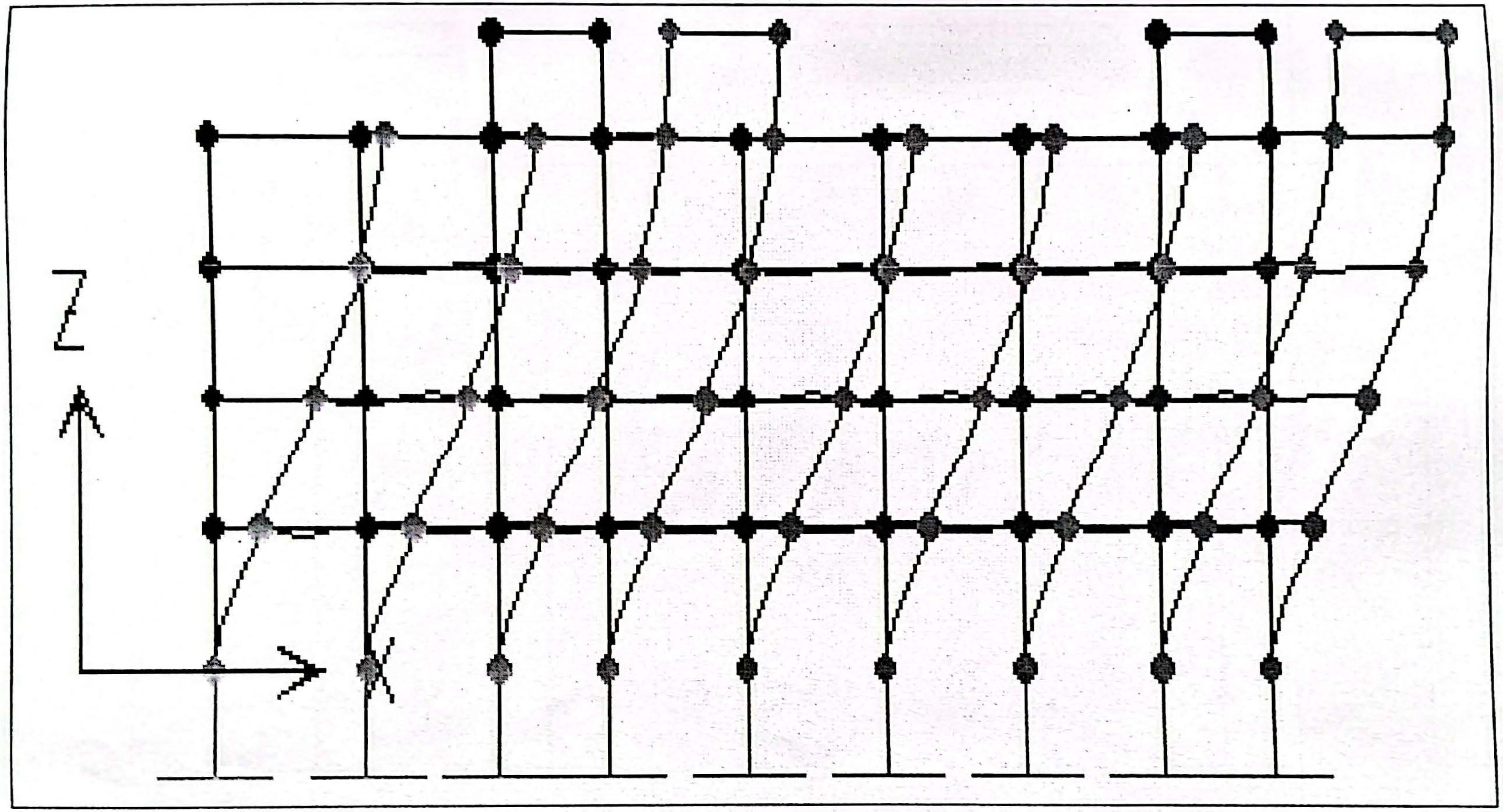


FIG 7. SHOWING DEFORMED SHAPE IN FRAME A-A DUE TO EPQX

4) EQPY

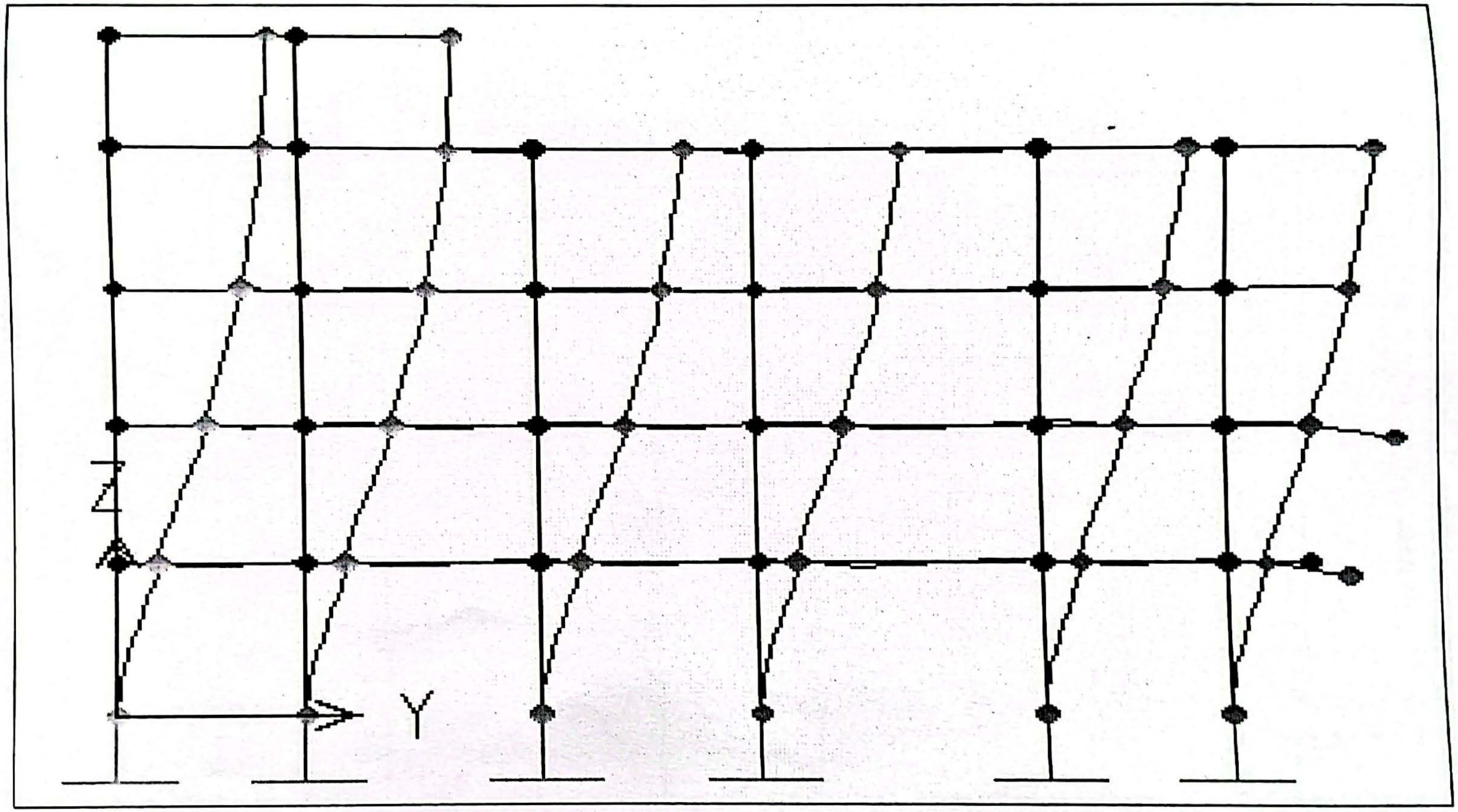


FIG 8. SHOWING DEFORMED SHAPE IN FRAME 4-4 DUE TO EQPY

Deformed shape due to load combinations

1) Combination 1 = 1.5 (DL + LL)

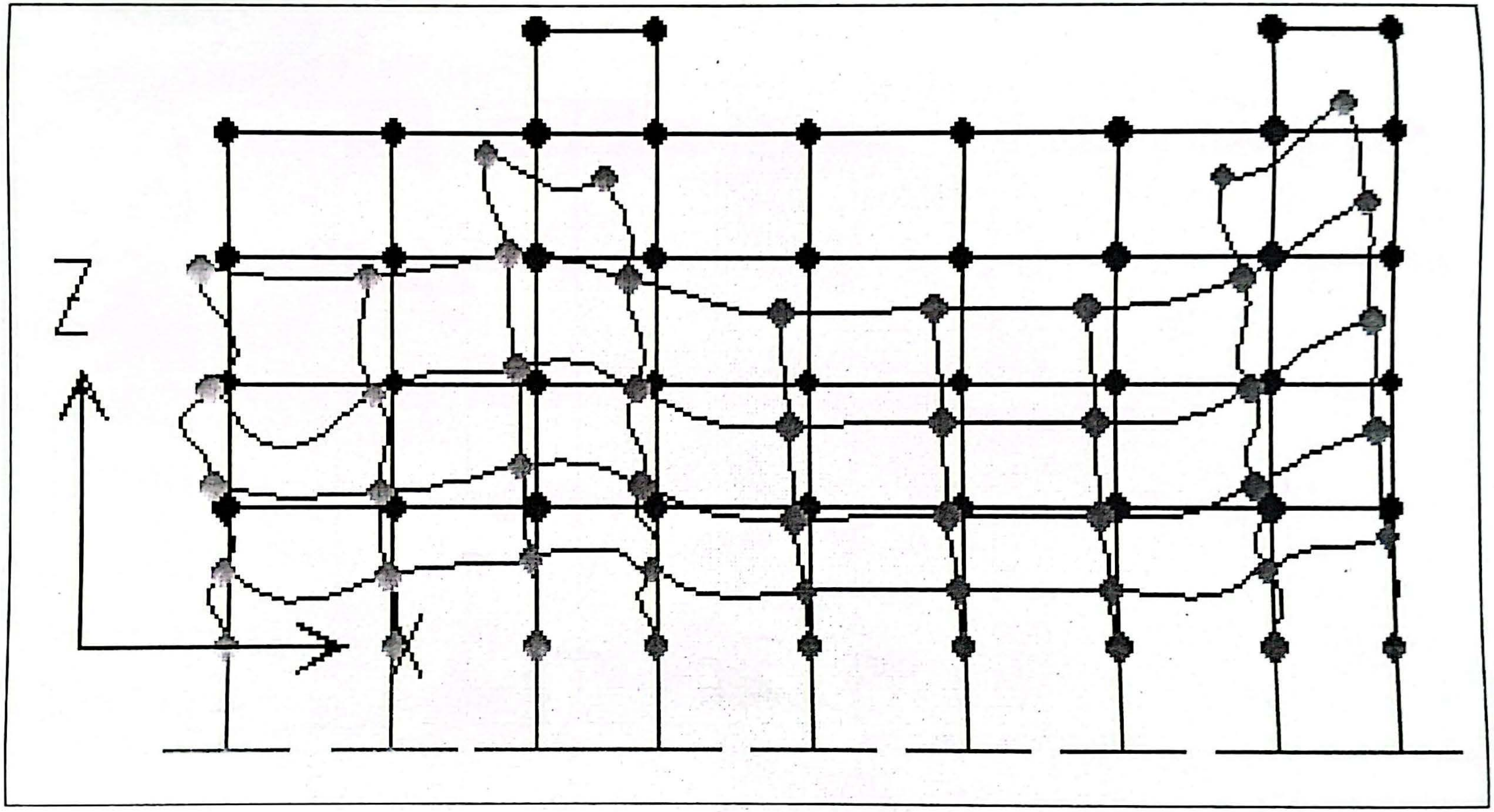


FIG 9. SHOWING DEFORMED SHAPE IN FRAME A-A DUE TO COMBINATION 1

2) **Combination 2 = 0.9 DL + 1.5 EQPX**

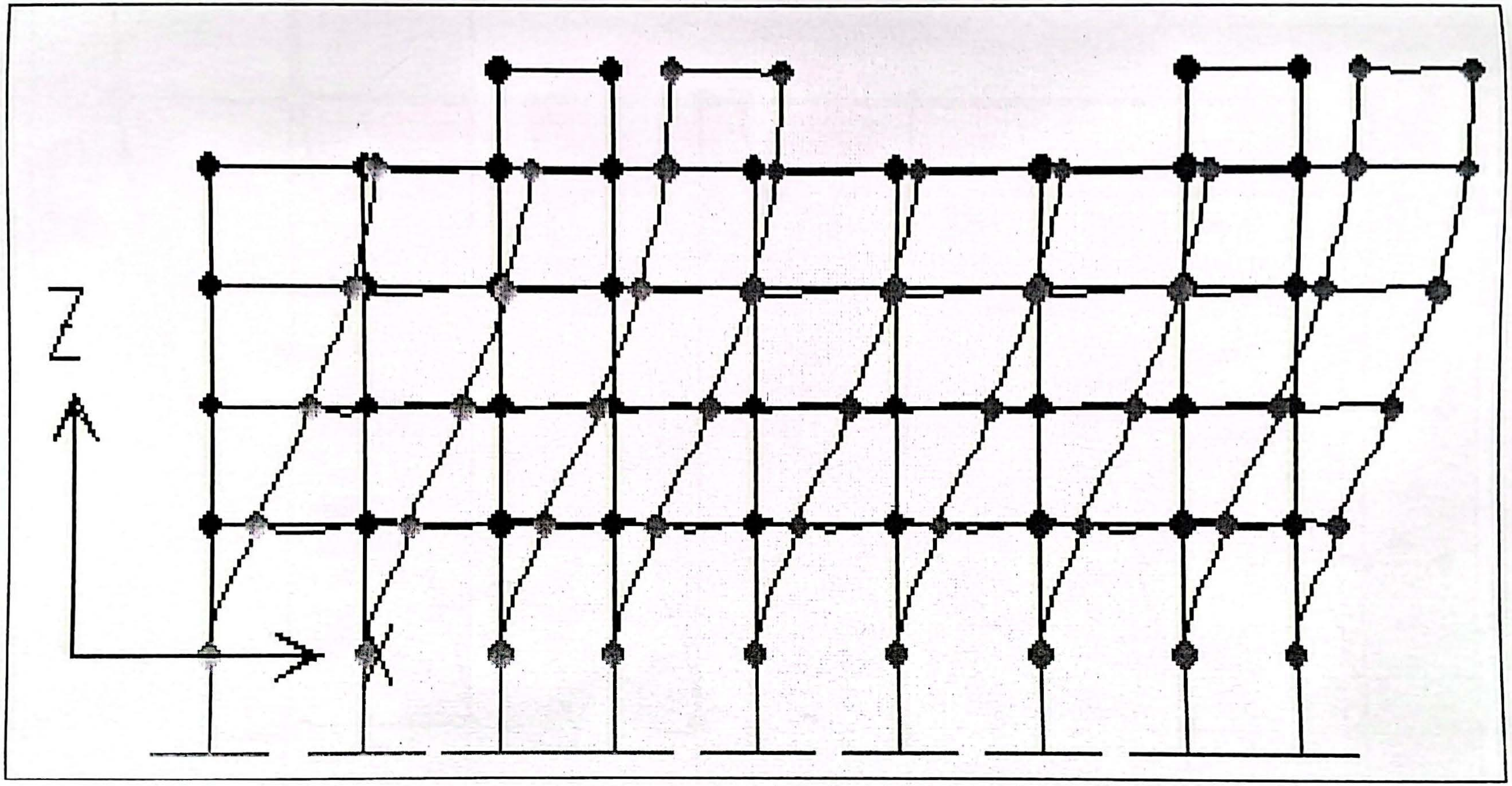


FIG 10. SHOWING DEFORMED SHAPE IN FRAME A-A DUE TO COMBINATION 2

3) Combination 9 = 1.2 (DL + LL + EQNY)

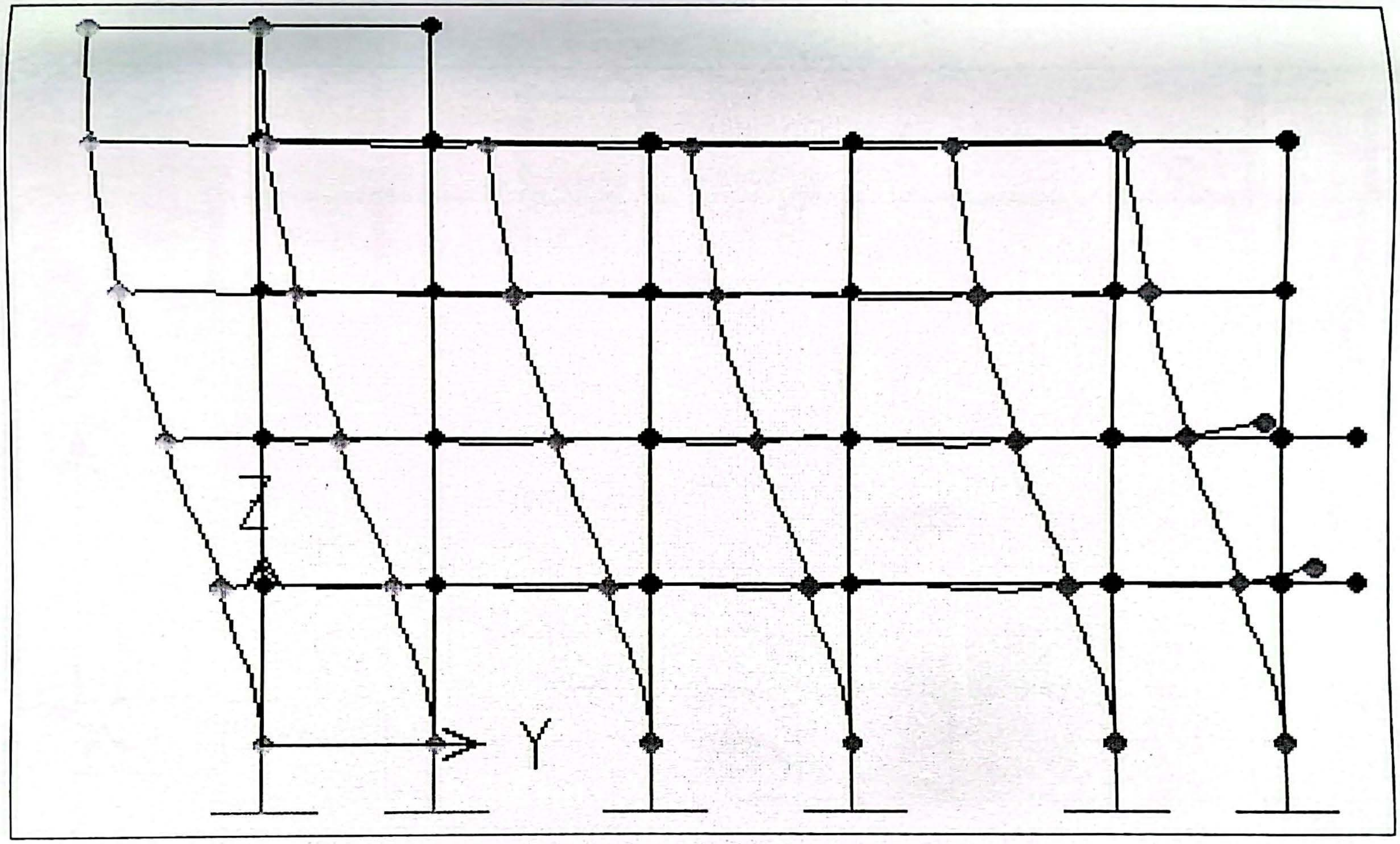


FIG 11. SHOWING DEFORMED SHAPE IN FRAME 4-4 DUE TO COMBINATION 9

1) **Axial Load**

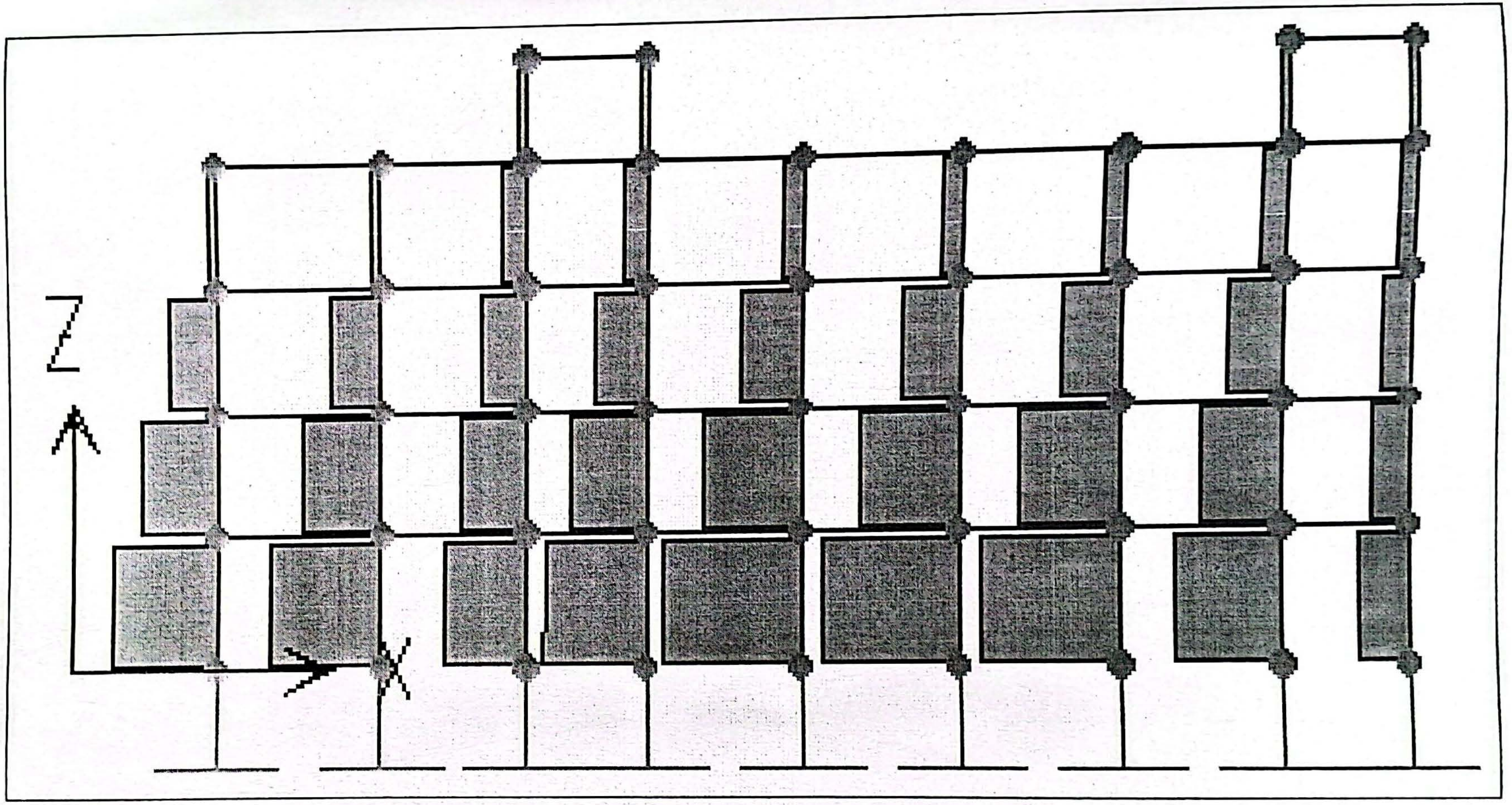


FIG 12. SHOWING AXIAL LOAD ON THE FRAME A-A DUE TO COMBINATION 1

2) Shear force

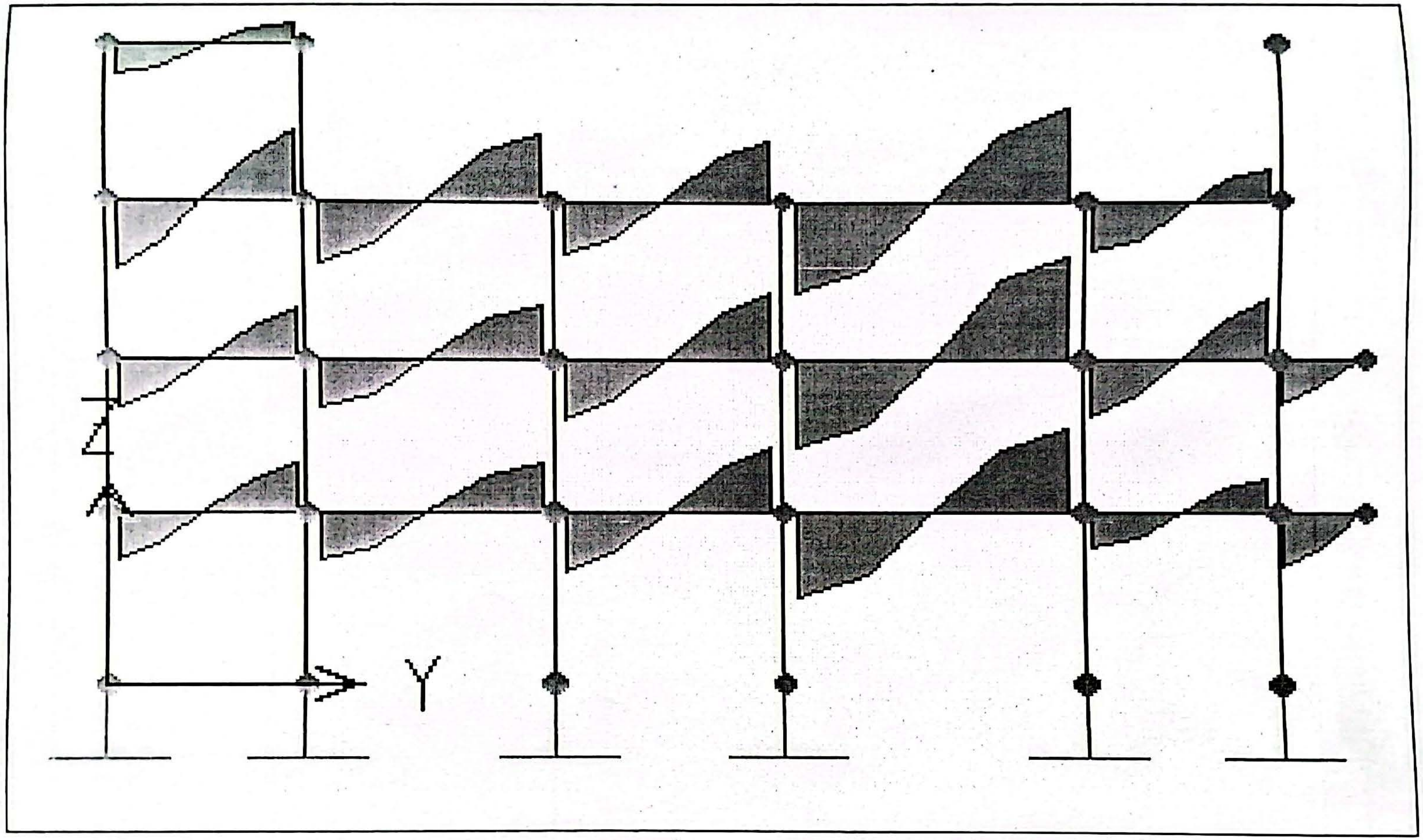


FIG 13. SHOWING SHEAR FORCE 2-2 ON THE FRAME 3-3 DUE TO COMBINATION 1

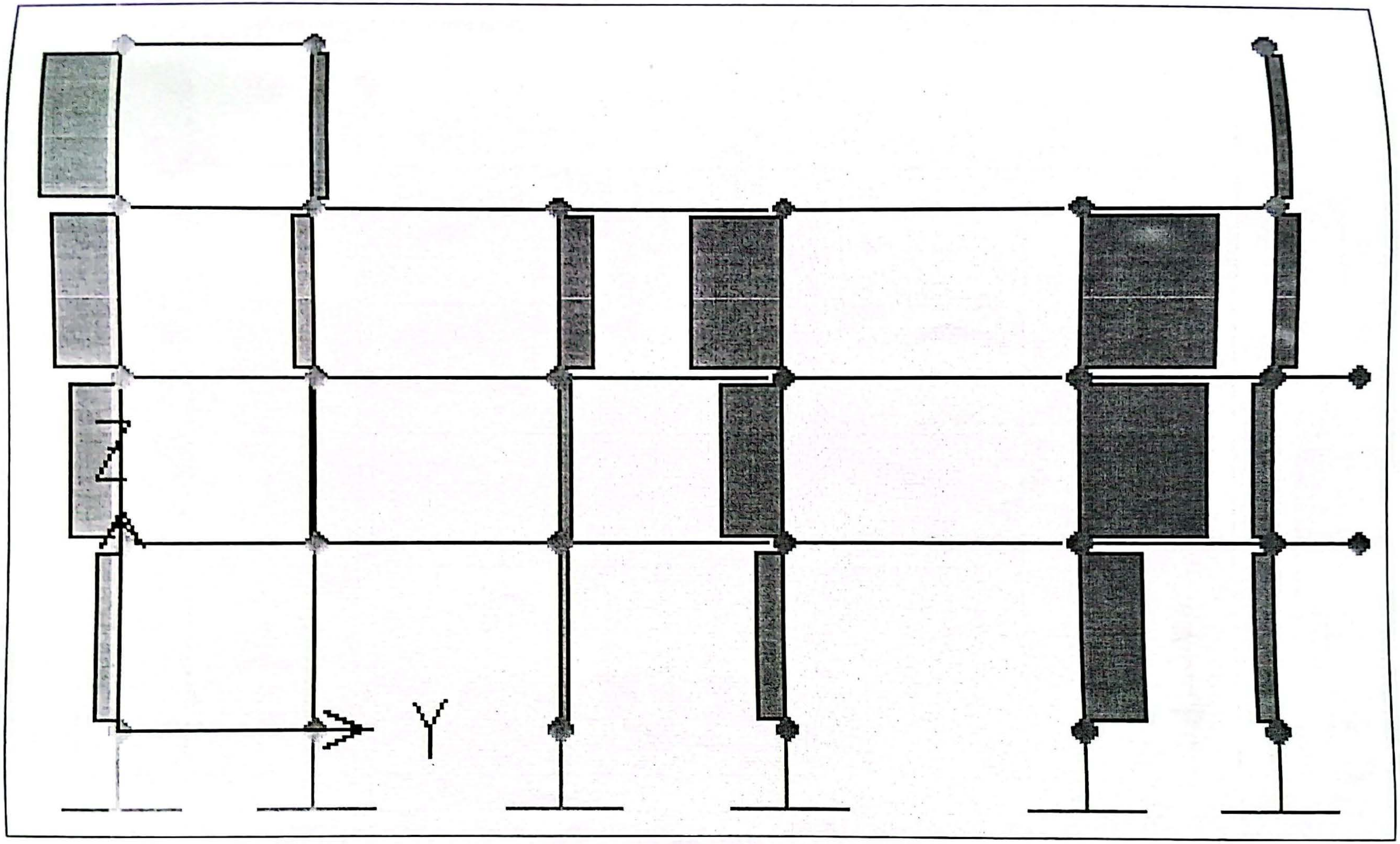


FIG 14. SHOWING SHEAR FORCE 3-3 ON THE FRAME 3-3 DUE TO COMBINATION 1

3) **Bending Moment**

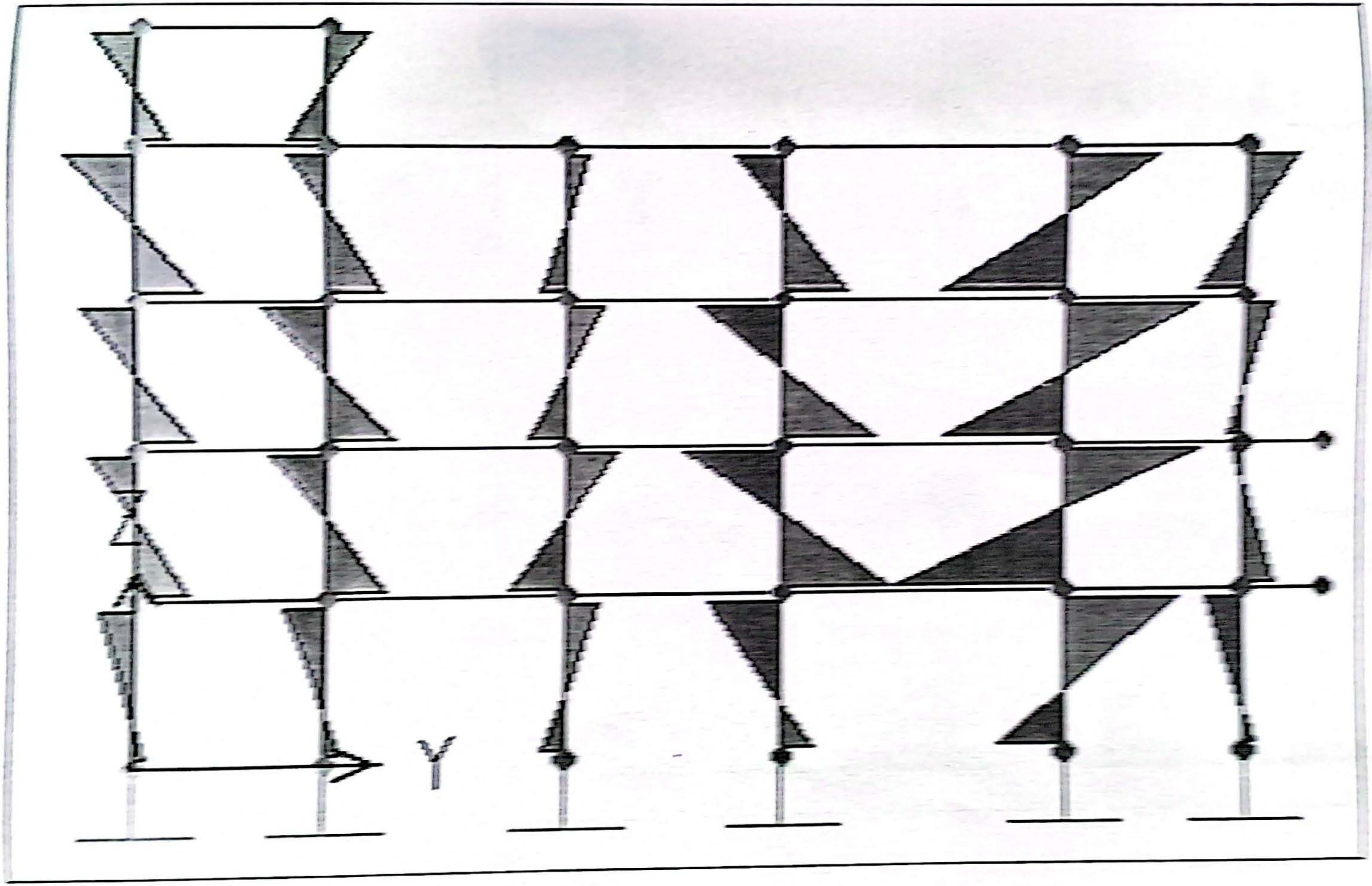


FIG 15. SHOWING BENDING MOMENT 2-2 ON THE FRAME 3-3 DUE TO COMBINATION 1

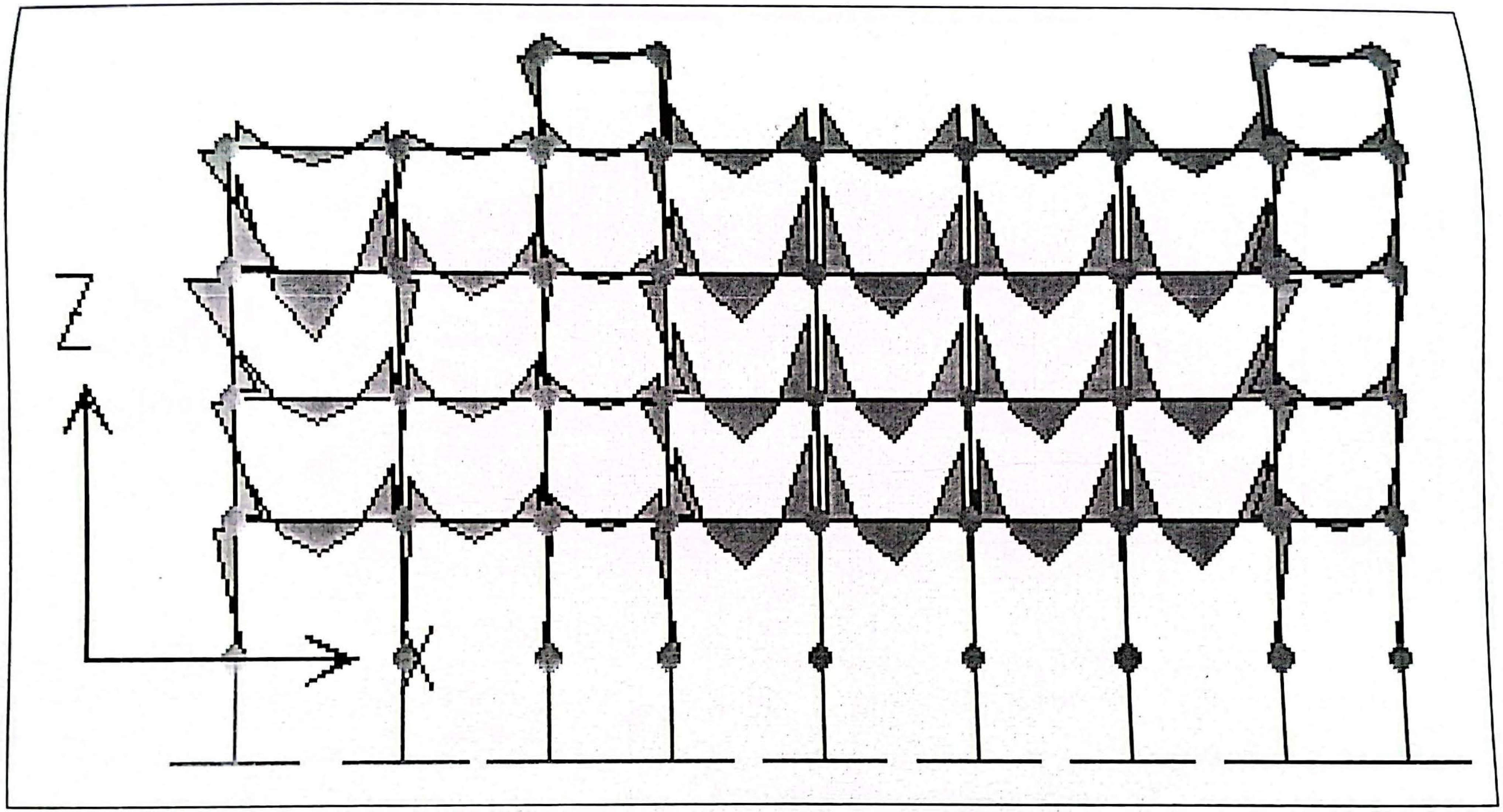
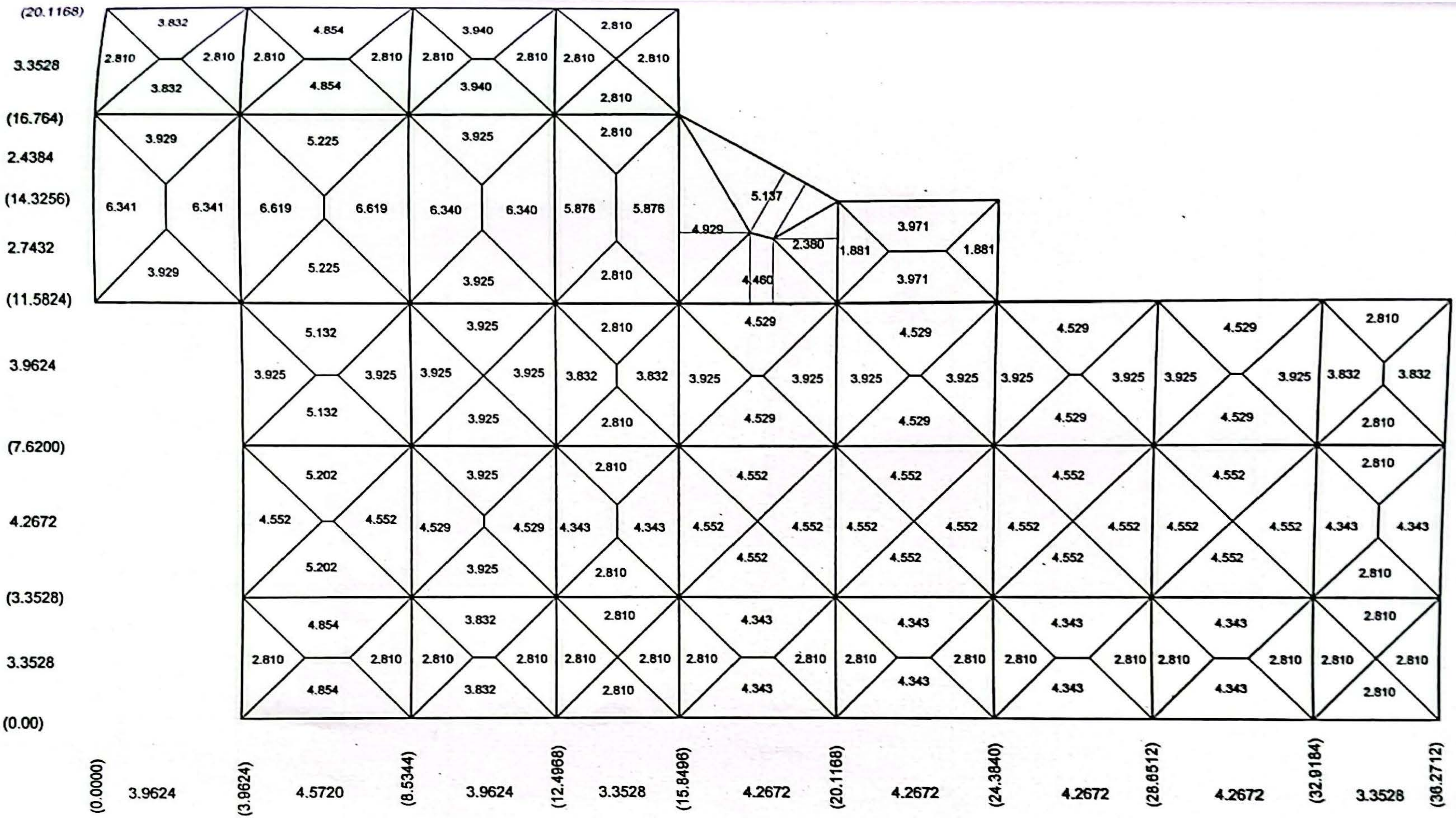
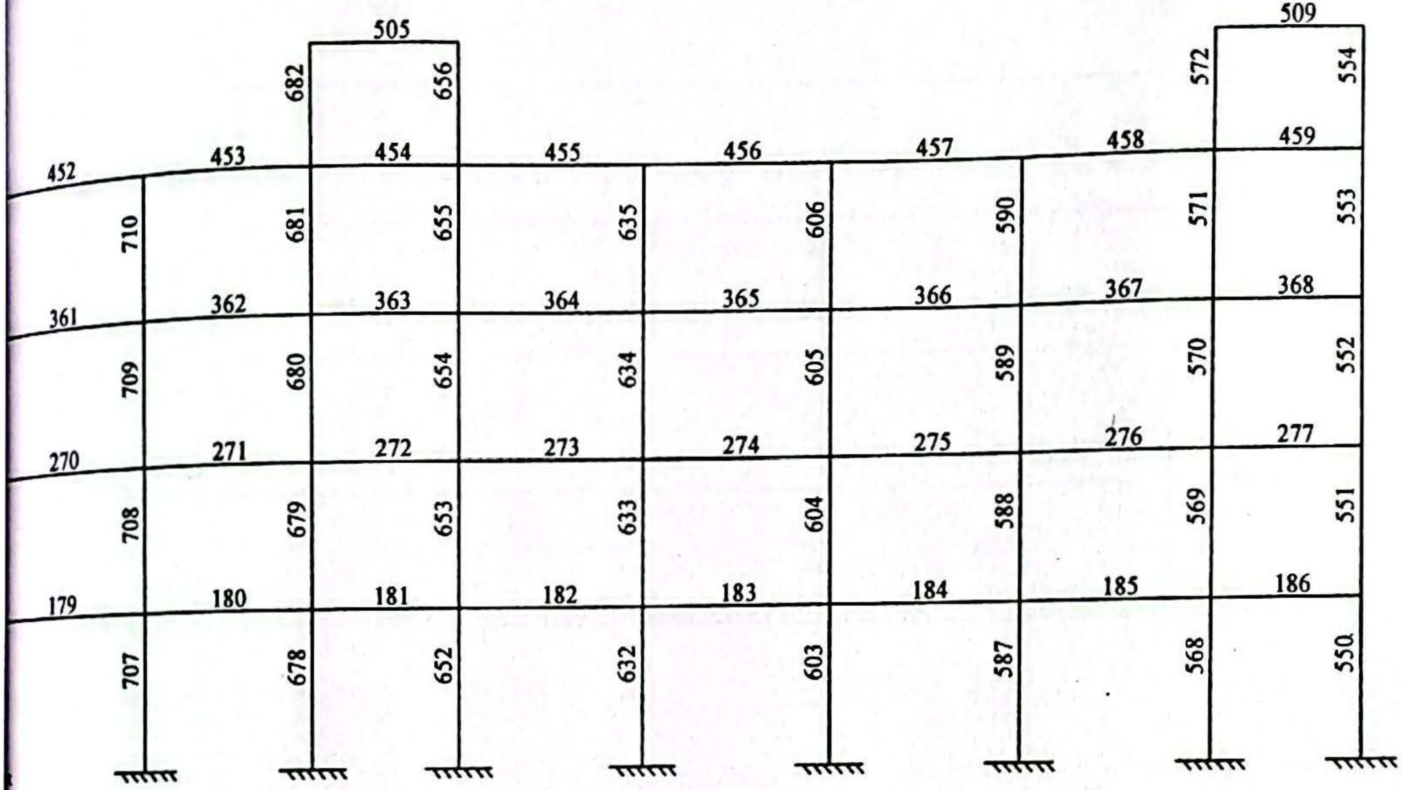


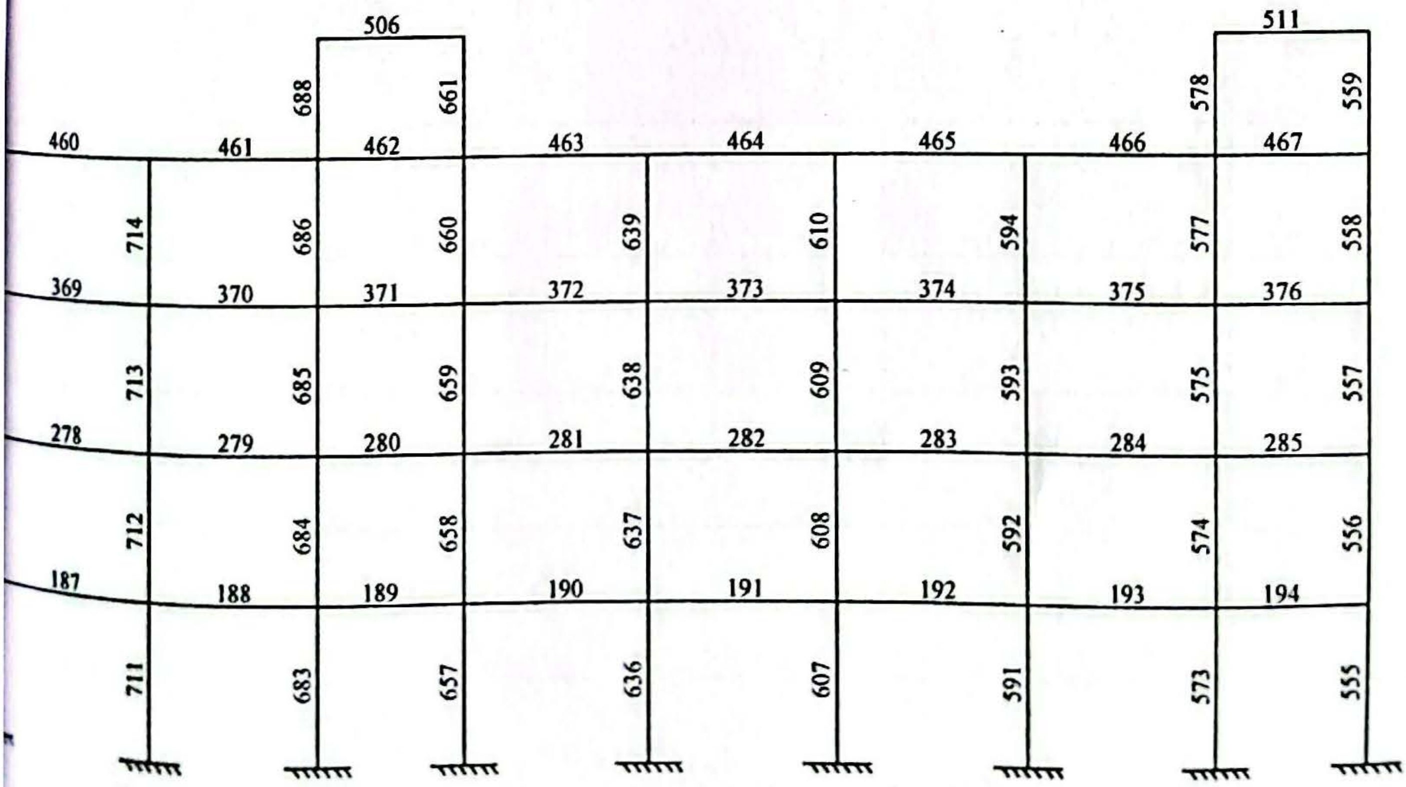
FIG 16. SHOWING BENDING MOMENT 3-3 ON THE FRAME A-A DUE TO COMBINATION 1



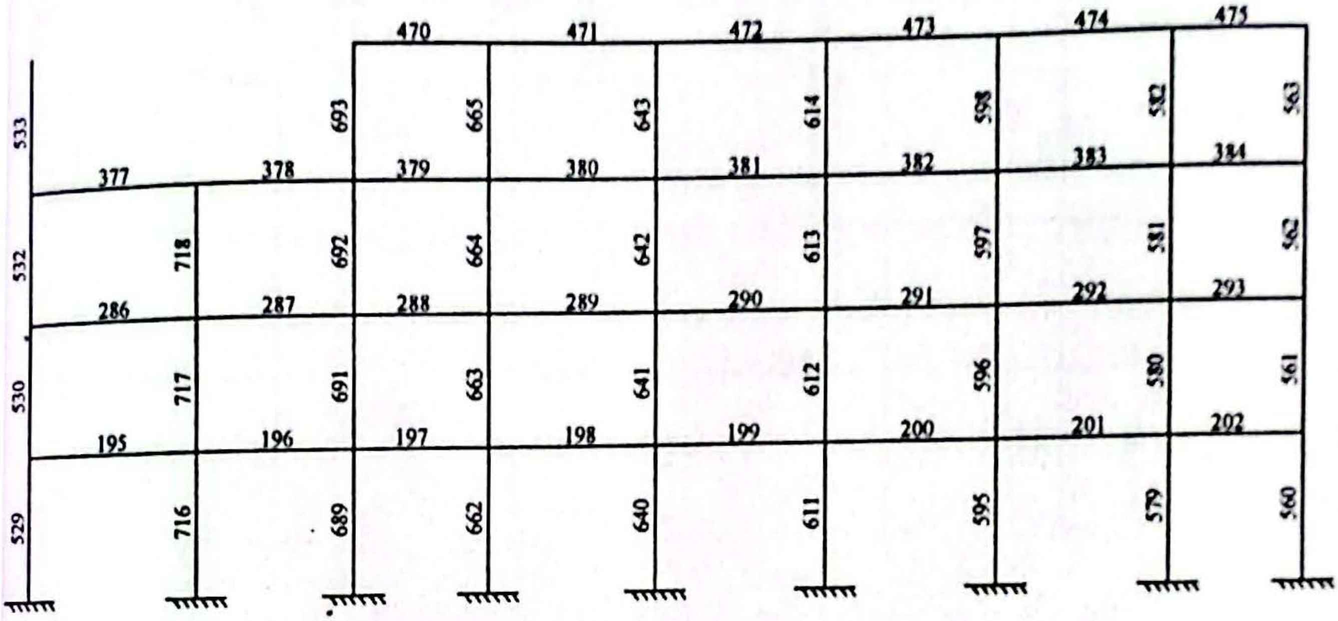
PLAN SHOWING YIELD LINE



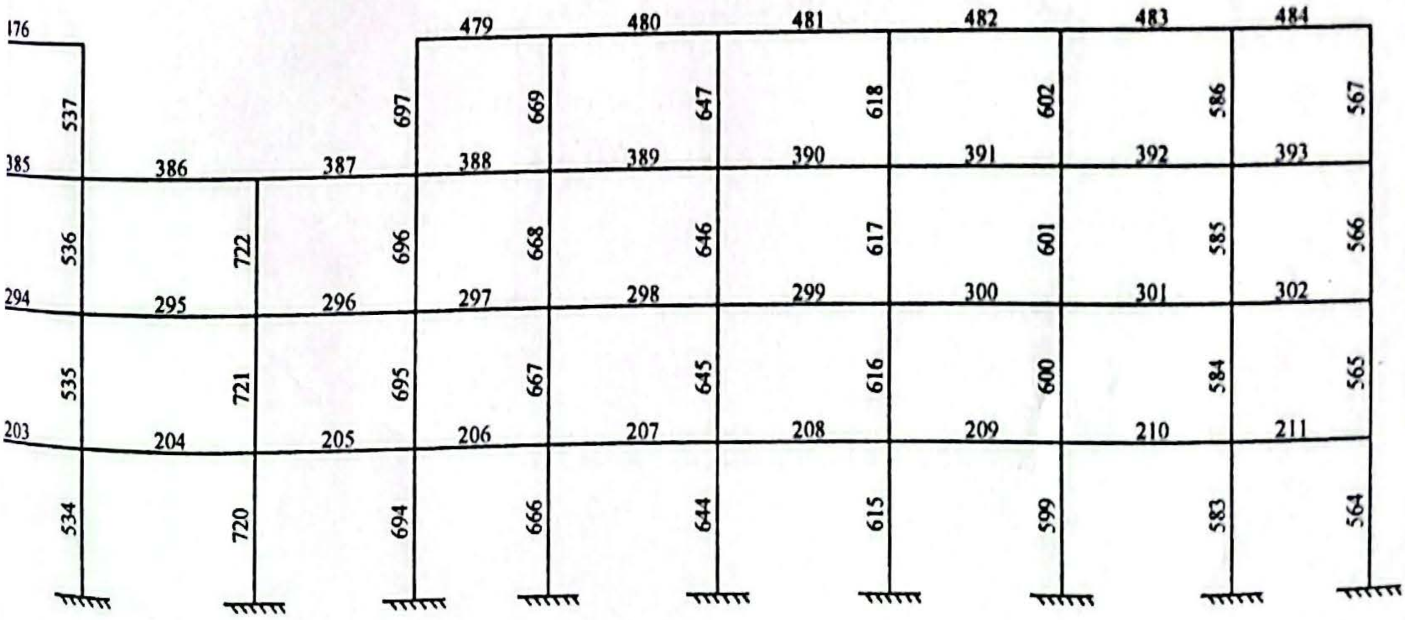
FRAME A-A



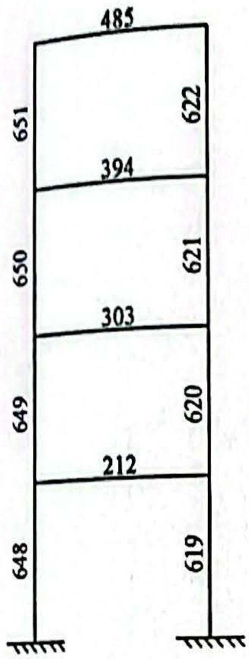
FRAME B-B



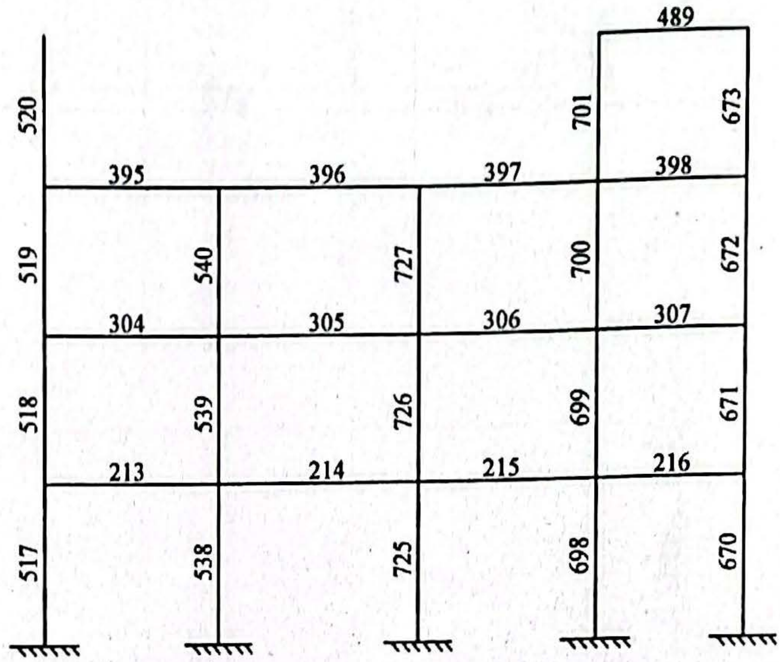
FRAME C-C



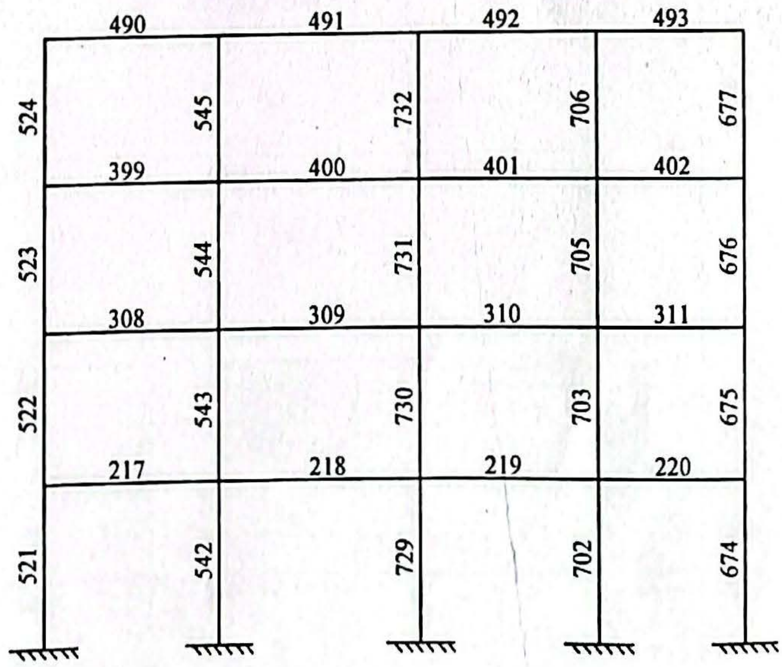
FRAME D-D



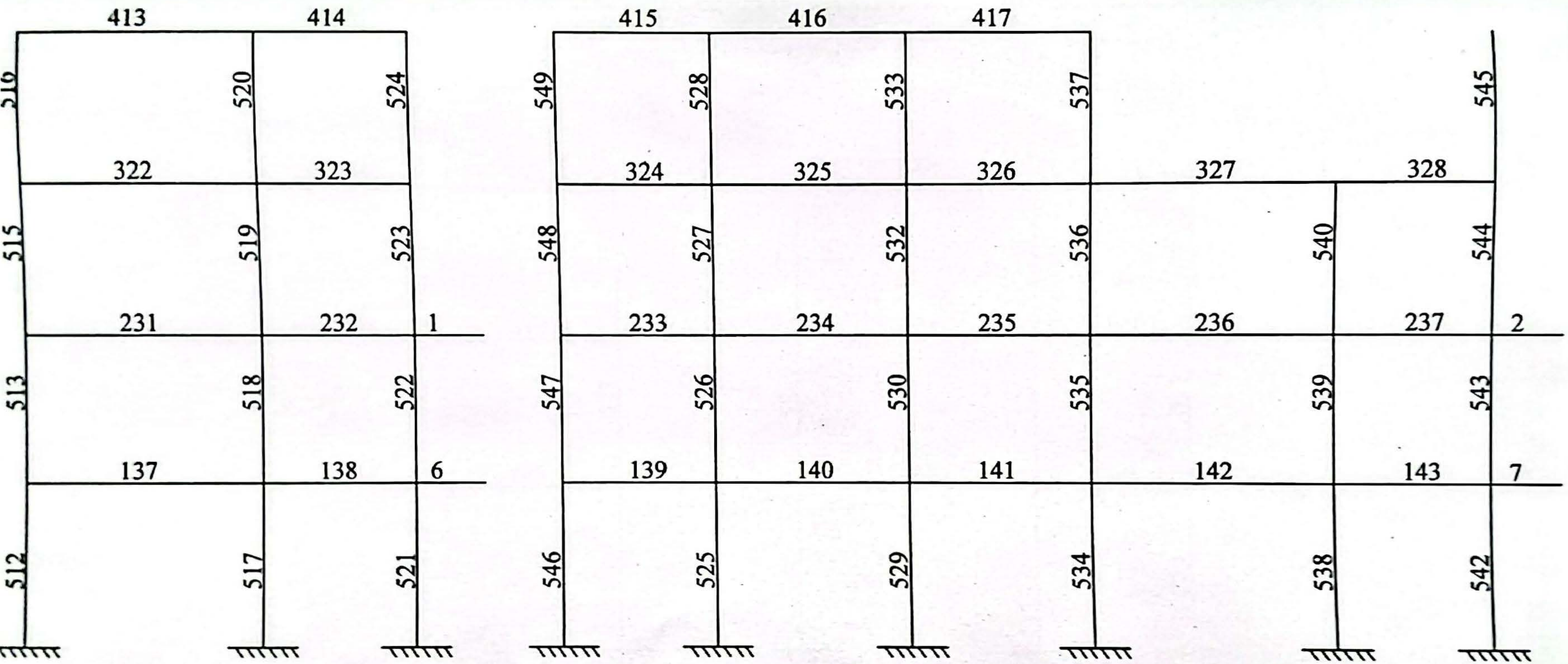
FRAME E-E



FRAME F-F

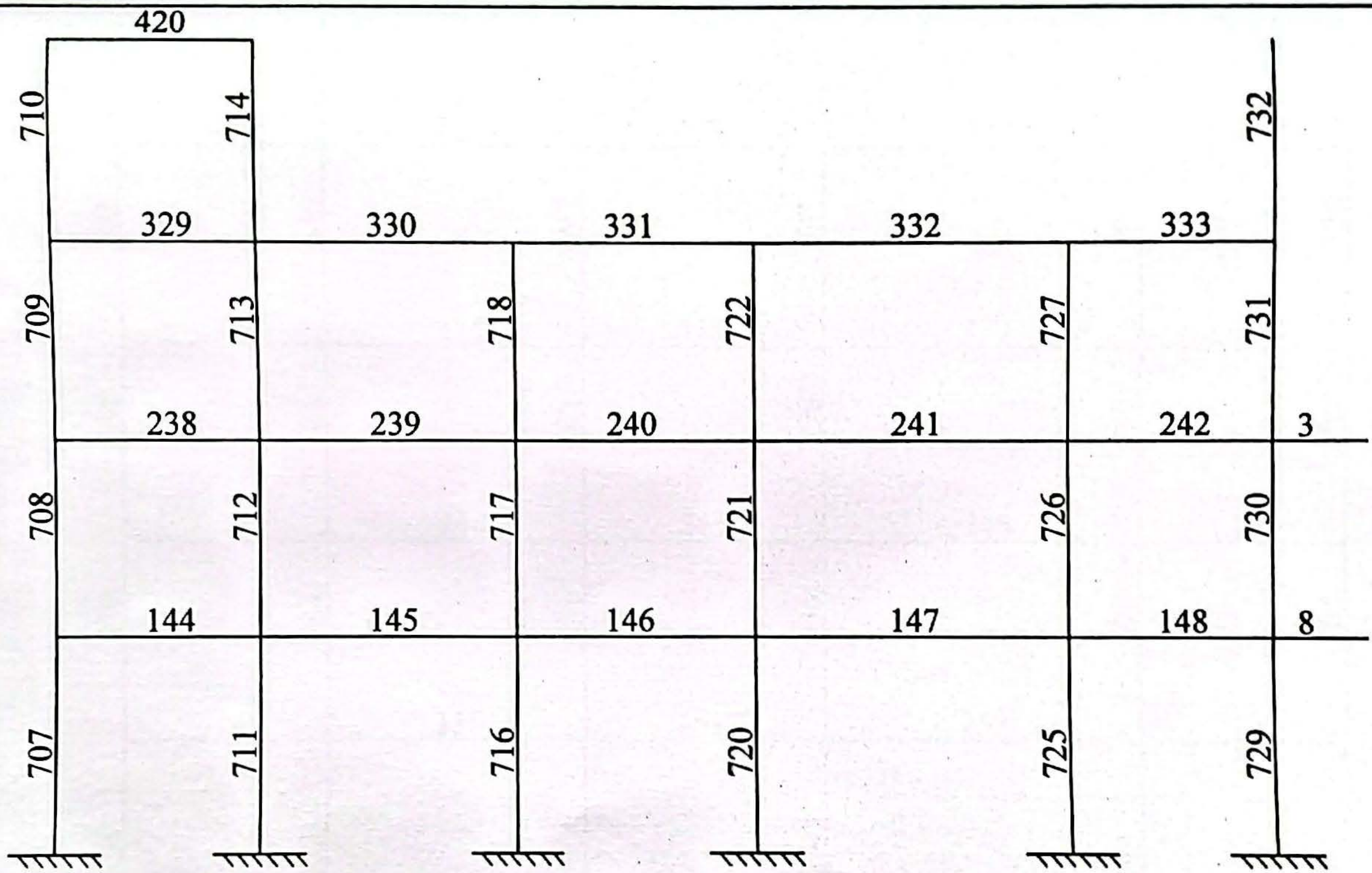


FRAME G-G



FRAME 1-1

FRAME 2-2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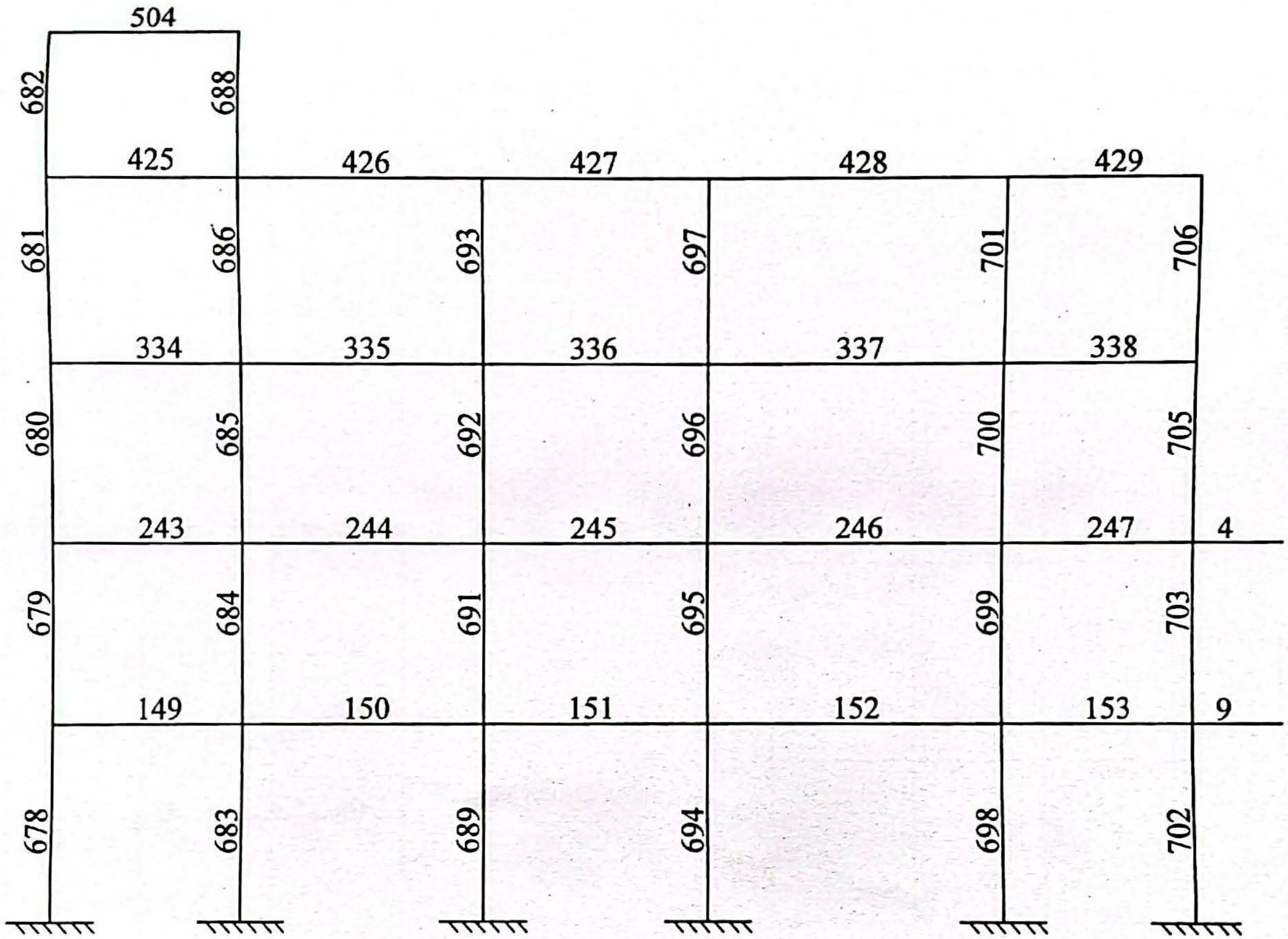
FRAME 3-3



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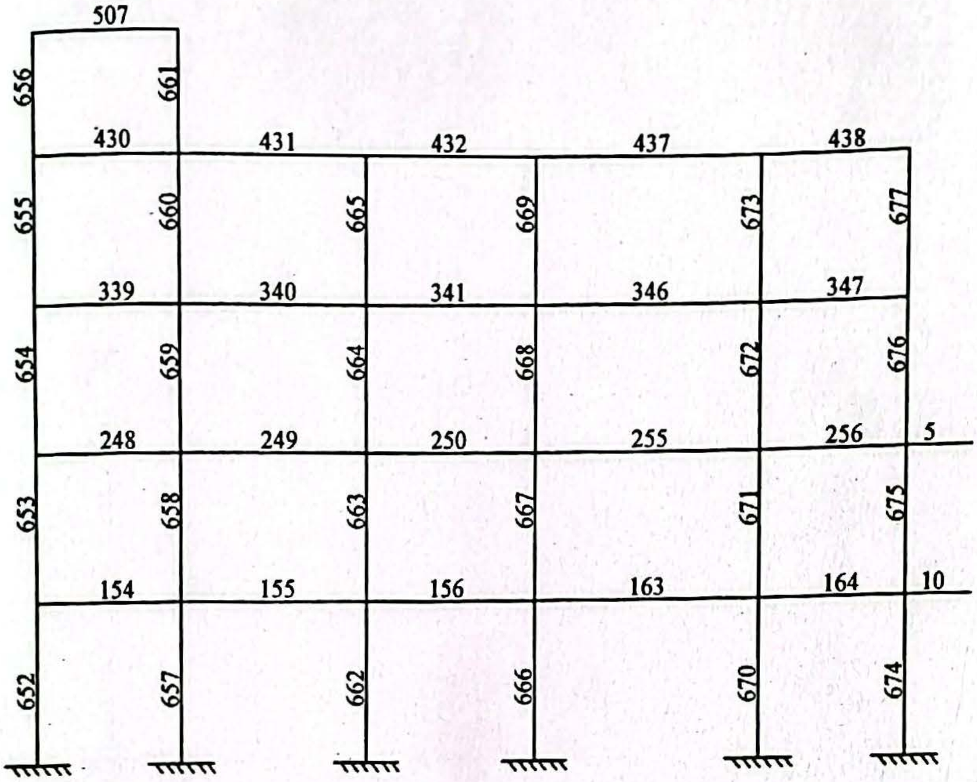
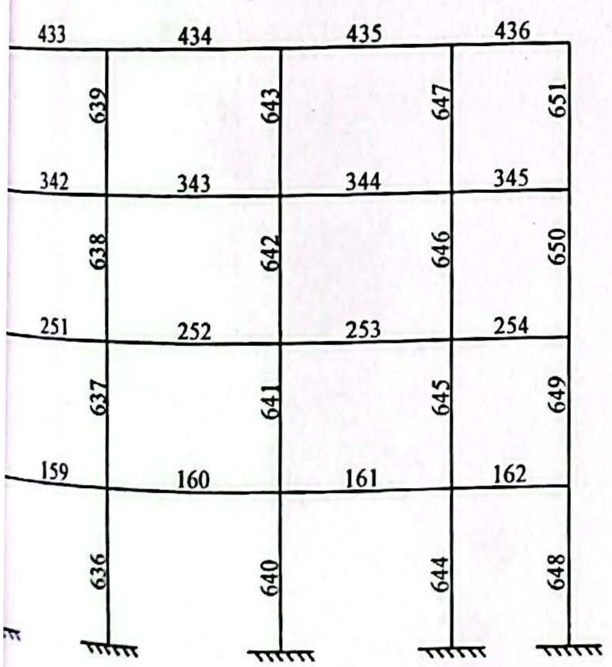
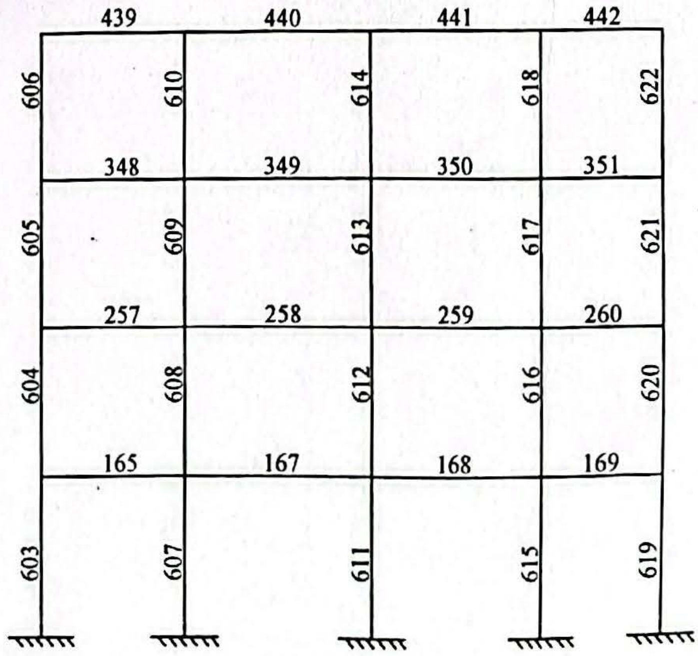
FRAME 4-4



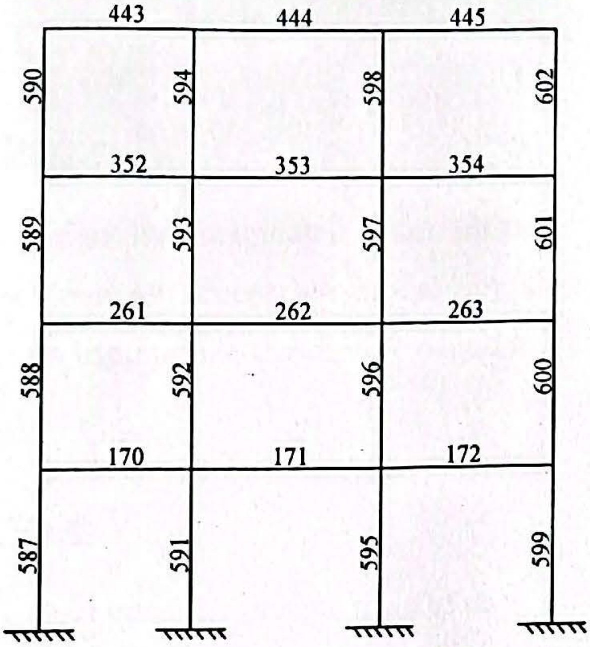
FRAME 5-5



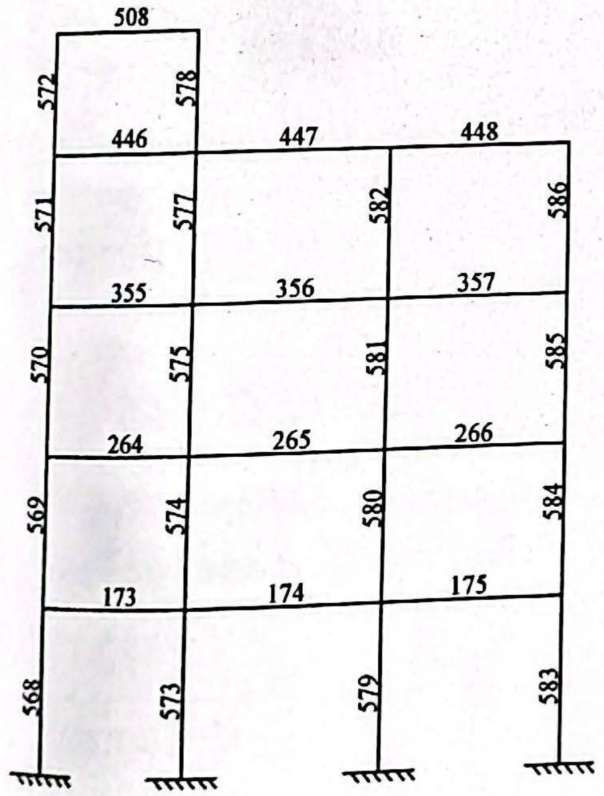


FRAME 6-6

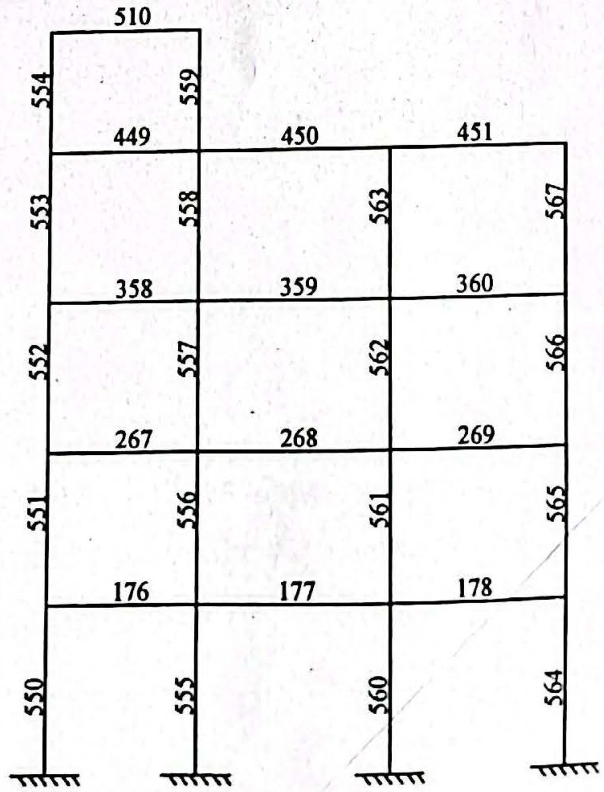
FRAME 7-7



FRAME 8-8



FRAME 9-9



FRAME 10-10

CHAPTER-4

DESIGN OF STRUCTURAL ELEMENTS

Limit State of Design

Limit state design has originated from ultimate or plastic design. The basic concept is to achieve an acceptable probability that a structure will not become unserviceable in its lifetime for the use for which it is intended, i.e. it will not reach a limit state.

Collapse limit State:

To satisfy this limit state, the strength must be adequate to carry the loads. Account must also be taken of stability. Violation of collapse limit state implies failure in the sense that a clearly defined limit state of structural usefulness has been exceeded. However, it does not mean a complete collapse. This limit state may correspond to:

Flexure, Compression, Shear, and Torsion

Serviceability limit State:

This state corresponds to development of excessive deformation and is used for checking members in which magnitude of deformations may limit the use of structure or its components. This limit state may corresponds to:

Deflection, Cracking and Vibration

4.1 DESIGN OF SLAB

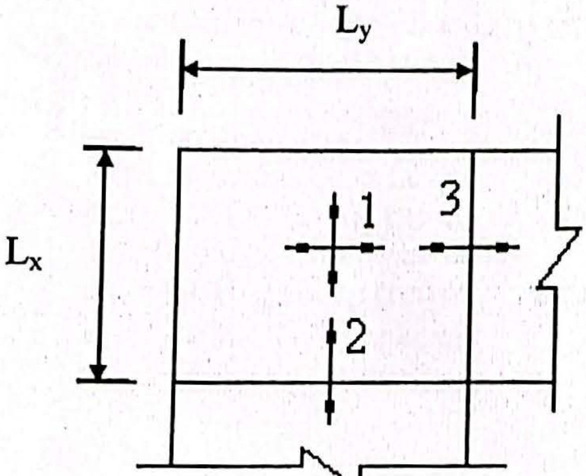
Slabs are plate elements forming floors and roofs of buildings and carrying distributed loads primarily by flexure. They may be supported on the walls, or on beams, or directly on columns.

One way slab:

These slabs have the length more than twice the breadth. It can be simply supported or continuous spanning in one direction only.

Two way slab:

When slabs are supported on four sides two spanning action occurs. These slabs may be simply supported or continuous on any or all sides. The deflection and bending moment in those slabs and considerably reduced as compared to one-way slab. Thus, a thinner slab can carry the same load when supported on all the four edges. It may be considered to consist of a series of interconnected beams with unit width and will transfer the load to the respective supports. It should also satisfy both serviceability and strength requirements.



1. Positive moments at midspan in the long and short spans
2. Negative moment in the short span
3. Negative moment in the long span
4. Zero moment at discontinuous edge (Coefficient = 0)

Fig. Direction of positive and negative moment coefficients

For S1 panel (two adjacent edges discontinuous) :

Ground floor (Room no 4)

Sample Calculation

Reference	Step	Calculation	Output
	1	**Type of slab:-** l_y(long span) = 4.5720 m l_x(short span) = 3.3528 m $\frac{l_y}{l_x}$ =1.364 < 2 Hence, two way slab.	

From preliminary design	2	**Thickness of slab:-** Overall depth (D) = 135 mm Use dia.(Ø) = 8 mm Cover = 20 mm ∴Effective Depth (d) = 135 - 8/2 - 20 = 111 mm	D = 135 mm d = 111 mm
IS 456 table 18	3	**Design load:-** Dead load = 25 x 0.13 = 3.375 KN/m^2 Live load = 2.50 KN/m^2 Floor finish load = 0.025 x 21 + 0.015 x 27 = <u>0.93 KN/m^2</u> ∴ Total design load = 6.805 KN/m^2 ∴Factored load(W_u) =1.5 x 6.805 =10.2075KN/m^2 Considering 1m width of slab, W_u = 10.2075 KN/m^2 x 1m = 10.2075 KN/m	W_u = 10.2075 KN/m
IS 456 table 26	4	**Bending Moment:-** Bending moment coefficients; α_{x-} = 0.0688 (short span continuous edge0 α_{x+} = 0.0515 (short span mid) α_{y-} = 0.047 (Long span continuous edge) α_{y+} = 0.035 (Long span mid) we have, $M_x = \alpha_x W_u l_x^2$ $M_y = \alpha_y W_u l_x^2$ Now, $M_{x-} = \alpha_{x-} W_u l_x^2$ = 0.068 x 10.2075 x 3.3528^2 = 7.890 KN-m $M_{x+} = \alpha_{x+} W_u l_x^2$ = 0.0515 x 10.2075 x 3.3528^2 = 5.909 KN-m $M_{y-} = \alpha_{y-} W_u l_x^2$ = 0.047 x 10.2075 x 3.3528^2 = 5.393 KN-m $M_{y+} = \alpha_{y+} W_u l_x^2$ = 0.035 x 10.2075 x 3.3528^2 = 4.016 KN-m	M_{x-} = 7.890 KN-m M_{x+} = 5.909 KN-m M_{y-} = 5.393 KN-m
IS 456-2000 Annex G	5	**Check for effective depth for max. bending moment;-** For f_y = 415 & fck = 20 $M_{max} = 0.138 f_{ck} bd^2$ $7.890*10^6 = 0.138*20*1000*d^2$ d= 53.467mm < 111 mm Hence, safe	M_{y+} = 4.016 KN-m
IS 456-2000 Annex G	6	**Area of steel Reinforcement:-** (a) For short span (x-direction) (i) Continuous edge $M_{u,x-} = 0.87\, f_y A_{st} d_x [\ 1 - \frac{f_y A_{st}}{f_{ck}\, b.d_x}\ ]$ $7.890 * 10^{-6} = 0.87*415*A_{st}*111*[\ 1 - \frac{415 * A_{st}}{20*1000*111}\ ]$	d is enough

		By solving ,we get $A_{st,x-} = 204.706\ mm^2$ <u>Check for spacing:</u> Spacing (S) = 1000*Area of one bar/$A_{st,x-}$ $= 1000*\frac{\pi*8^2/4}{204.706}$ $= 245.549$ mm Adopt spacing(s) = 240mm < (3*111=333mm or 300 mm whichever is smaller) ∴ Area provided, $A_{st,x-} = 1000*\frac{\pi*8^2/4}{240}$ $= 209.439\ mm^2 > 204.706\ mm^2$ <u>Check for minimum steel area percentage:</u> $p\% = A_{st}*100/b.D = \frac{209.439}{1000*135}*100$ $= 0.155\% > 0.12\%$ Hence O.K. ∴Use 8 mm Ø bar@240 mm c/c. (a) For short span (i) For mid span of slab: $M_{u,x+} = 0.87\ f_y A_{st} d_x\ [\ 1-\frac{f_y A_{st}}{f_{ck}\ b.d_x}\ ]$ $5.909E6 = 0.87*415*A_{st}*111[\ 1-\frac{415*A_{st}}{20*1000*111}\ ]$ By solving,we get $A_{st,x+} = 151.747\ mm^2$ <u>Check for spacing:</u> Spacing(s)= $1000*\frac{\pi*8^2/4}{151.747} = 331.245$ mm Adopt spacing(s) = 270 mm < 300 mm ∴Area provided,$A_{st,x+} = 1000*\frac{\pi*8^2/4}{270}$ $= 186.168\ mm^2 > 151.747\ mm^2$ <u>Check for minimum steel area percentage</u> $p\% = \frac{186.168}{1000*135}*100 = 0.137\% > 0.12\%$ Hence, O.K. ∴Use 8 mm ϕ bar@ 270 mm c/c. (a) For long span (y-direction):- (i) For continuous edge: Use 8 mm Ø bar for longer direction	$A_{st,x-}$ $=204.706mm^2$ $A_{st,x-}$ provided $=209.439mm^2$
IS 456-2000 Annex G			

IS 456-2000
Annex G

$\therefore d_y = 111 - 8/2 - 8/2 = 103$ mm

$$M_{u,y-} = 0.87 f_y A_{st} d_x \left[1 - \frac{f_y A_{st}}{f_{ck} b.d_y} \right]$$

$$5.393E6 = 0.87*415*A_{st}*103\left[1 - \frac{415*A_{st}}{20*1000*111} \right]$$

By solving ,we get

$A_{st,y-} = 149.523$ mm^2

T8 at 270mm, 186.168mm^2

Check for spacing:

$$\text{Spacing(s)} = 1000*\frac{\pi*8^2/4}{149.523} = 336.172 \text{ mm}$$

Adopt spacing(s) = 270 mm < 300 mm

$$\therefore \text{Area provided, } A_{st,y-} = 1000*\frac{\pi*8^2/4}{270}$$

$= 186.168$ mm$^2 > 149.523$ mm^2

Check for minimum steel area percentage

$$p\% = \frac{186.168}{1000*135}*100 = 0.137\% > 0.12\%$$

Hence, O.K.

$\therefore$ Use 8 mm Ø bar@270 mmc/c.

IS 456-2000
Annex G

(b) For long span:-

(i) For mid span of slab:

$$M_{u,y+} = 0.87 f_y A_{st} d_y \left[1 - \frac{f_y A_{st}}{f_{ck} b.d_y} \right]$$

$$4.016E6 = 0.87*415*A_{st}*103\left[1 - \frac{415*A_{st}}{20*1000*103} \right]$$

By solving ,we get

$A_{st,y+} = 110.448$ mm^2

Check for spacing:

$$\text{Spacing (s)} = 1000*\frac{\pi*8^2/4}{110.448} = 455.105 \text{ mm}$$

Adopt spacing(s) = 270 mm < 300 mm

$$\therefore \text{Area provided, } A_{st,y+} = 1000*\frac{\pi*8^2/4}{270}$$

$= 186.168$ mm$^2 > 110.448$ mm^2

Check for minimum steel area percentage

$$p\% = \frac{186.168}{1000*135}*100\% = 0.137\% > 0.12\%$$

Hence, O.K.

$\therefore$ Use 8 mm Ø bar@270 mmc/c.

7 Check for shear force at short edge:-

Maximum shear force:

$V = 1/2 * W_u.l_x$

$= 1/2 * 10.2075 * 3.3528$

$= 17.112$ KN

Nominal shear stress:

IS 456-2000 Clause 40.1

$$\tau_v = \frac{V}{bd} = \frac{17.112*1000}{1000*111} = 0.154 \text{ N/mm}^2$$

$$\text{Percent tensile steel} = 100\frac{A_s}{bd} = \frac{100*209.439}{1000*111} = 0.188\%$$

IS 456-2000 Table 19

Shear strength of M20 concrete for 0.188% steel,

$$\tau_c = 0.28 + \frac{0.36-0.28}{0.25-0.15}*(0.188-0.15)$$

$\tau_c = 0.3104 \text{N/mm}^2$

IS 456-2000 Clause 40.2.1.1.

Shear strength in slab,

$$\tau_c' = k\tau_c$$

for D = 135mm, k = 1.3

$\therefore \tau_c' = 1.3*0.3104$

$= 0.4035 \text{ N/mm}^2$

$\tau_v = 0.154 \text{ N/mm}^2 < \tau_c' = 0.4035 \text{ N/mm}^2$

Hence, safe

$\therefore$ No shear reinforcement is required.

8 Check for shear force at long edge:-

Maximum shear force:

$V = 1/2 * W_u.l_x$

$= 1/2 * 10.2075 * 3.3528$

$= 17.112$KN

Nominal shear stress:

IS 456-2000 Clause 40.1

$\tau_v = V/bd = 17.112*1000/(1000*111) = 0.154 \text{N/mm}^2$

percent tensile steel $= 100A_s/bd$

$= 100*209.439/1000*111$

$= 0.188\%$

Shear strength of M20 concrete for 0.188% steel,

$\tau_c = 0.28+(0.36-0.28)*(0.188-0.15)/(0.25-0.15)$

$\tau_c = 0.3104 \text{N/mm}^2$

IS 456-2000 Clause 40.2.1.1

Shear strength in slab,

$\tau_c' = k\tau_c$

for D=135mm, k=1.3

$\therefore \tau_c' = 1.3*0.3104$

$= 0.4035 \text{N/mm}^2$

$\tau_v = 0.154 \text{ N/mm}^2 < \tau_c' = 0.4035 \text{ N/mm}^2$

Hence safe

$\therefore$ No shear reinforcement is required.

9 Check for Development length: at short edge:-

M_1 at support:

Moment of resistance offered by 8mm dia bars @240mmc/c

IS 456-2000 Annex G		$M_{1,x} = 0.87 f_y A_{st} d_y [1-\frac{f_y A_{st}}{f_{ck} b.d_y}]$ $= 0.87*415*209.439/2* [1-\frac{415*209.439/2}{20*1000*111}]*111$ $= 4114640.035$ N-mm $= 4.115*10^6$ N-mm Maximum shear force, $V = 17.112$ KN $= 17.112*10^3$ N Development length of bar,	
IS 456-2000 Clause 26.2.1		$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} = \frac{\phi*0.87*f_y}{4\tau_{bd}}$ For M20, $\tau_{bd} = 1.2*1.6=1.92$ $L_d = 0.87*415\, Ø/4*1.2*1.6 = 47Ø$ Anchorage value of bar bent at 90° including 60mm straight length, $L_o = 60+8*8 = 124$ mm	
IS 456-2000 Clause 26.2.3.3.c		$L_d \le 1.3*\frac{M_1}{V} + L_o$ $47Ø \le 1.3*\frac{4114640.035}{17.112*10^3} + 124$ $47Ø \le 1.3*240.45+124$ $47Ø \le 312.589+124$ $Ø = 9.289$ mm > provided $Ø = 8$ mm Hence, safe in development length. Dia of bar is 8mm which is less than 9.289mm ∴ **O.K.**	
	10	**Calculation for Torsion Reinforcement** $l_x = 3353$ mm Length of Torsion reinforcement in both directions $= 0.2 l_x$ $= 0.2*3353 = 670.6$ mm Maximum positive steel Area, $A_{st} = 151.896$ mm^2 **In** corner of two edges discontinuous, i.e. corner ID: A-2 Steel Area required $= 0.75*A_{sst} = 0.75*151.896 = 113.92$ mm^2 Dia. of Bar (ϕ) = 8 mm So, spacing required (s) $= 670.6*\frac{\pi*8^2/4}{113.92} = 295.89$ mm Spacing adopted = 280 mm c/c < 3d or 300 mm whichever is smaller **In** corner of one edge discontinuous, i.e. corner ID B-2 and A-3 Steel Area required $= 0.75*A_{st}/2 = 0.75*151.896/2$ $= 56.96$ mm^2 So, spacing required (s) $= 670.6*\frac{\pi*8^2/4}{56.96} = 591.77$ mm Spacing adopted = 280 mm c/c < 3d or 300 mm whichever is smaller	

4.2 DESIGN OF BEAM

Beam is a reinforced concrete flexural member. It carry load by bending action. The beam may be rectangle, L and T section consisting of singly and doubly reinforcement. Design of the beam element requires the determination of the cross sectional dimensions and reinforcement details. It should satisfy both serviceability and strength requirements.

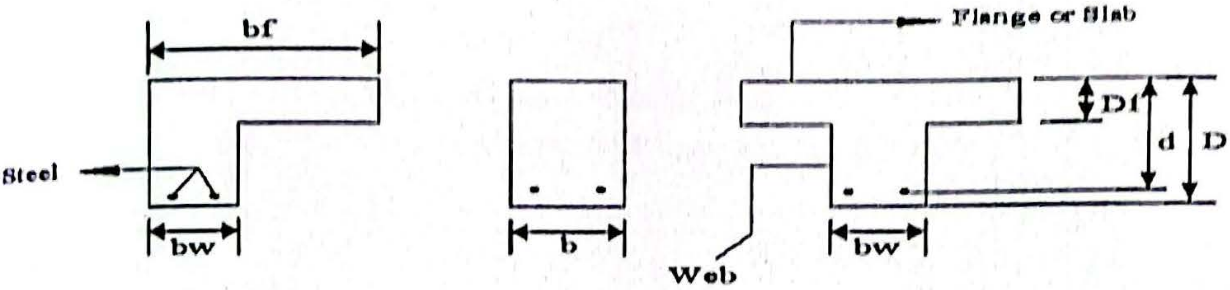


Fig. L-beam, rectangular beam and T- beam

Material characteristics

Concrete Grade = M20 (f_{ck} = 20 N/mm^2)

Steel Grade = Fe415 (f_y = 415 N/mm^2)

Beam Section

Width of Beam (b_w) = 230 mm

Overall depth (D) = 450 mm

Effective cover (d') = 45 mm

Effective depth (d) = 405 mm

Depth of flange (D_f) = 135 mm (Ground floor)

Sample Calculation

Beam No. 143

Reference	Step	Calculation	Remarks
		Design of beam at support (Rectangular beam)	
		b_w = 230 mm	
		d = 405 mm	
IS 456-2000 Annex G Clause 38.1	1.	**Limiting moment capacity of the Rectangular Beam**	
		$M_{u,lim} = 0.138 f_{ck} b d^2$ (for Fe415)	
		$= 0.138*20*230*405^2$	
		= 104123070 N-mm	
		= 104.123 KN-m	
		$M_{u,given}$ = 245.490 KN-m	
		$M_{u,lim} < M_{u,given}$	
		Hence, doubly reinforced beam	

IS 456-2000 Pg. 70 (Clause 38.1)	**2.**	**Balanced Depth of Neutral Axis** $(X_{u,lim}) = 0.48d = 0.48*405 = 193.995mm$	
IS 456-2000 Annex G Clause 38.1	**3.**	**Calculation of tension steel reinforcement** $A_{st1} = \frac{M_{u,lim}}{0.87 * f_y(d - 0.416x_{u,lim})}$ $= \frac{104123070}{0.87 * 415(405 - 0.416 * 193.995)}$ $= 889.273\ mm^2$ $M_{u,2} = M_{u,given} - M_{u,lim}$ $= 245.490 - 104.123 = 141.367$ KN-m	
SP 16 Clause 2.3.2		$A_{st2} = \frac{M_{u,2}}{0.87 * f_y(d - d')} = \frac{141.367 * 10^6}{0.87 * 415(405 - 45)}$ $= 1087.623\ mm^2$ Total tension reinforcement $A_{st} = A_{st1} + A_{st2}$ $= 889.273 + 1087.623$ $= 1976.896\ mm^2$	
SP 16 Clause 2.3.2	**4.**	**Calculation of compression steel reinforcement** $A_{sc} = \frac{0.87 * f_y * A_{st2}}{(f_{sc} - f_{cc})}$ Where, from table For d/d' = 45/405 = 0.111	
SP 16 Table F		$f_{sc} = 350.55$ And $f_{cc} = 0.446f_{ck} = 0.446*20 = 8.92$ $\therefore A_{sc} = \frac{0.87 * 415_y * 1087.623}{(350.55 - 8.92)} = 1149.449\ mm^2$	
IS 456-2000 Clause 26.5.1.1.a	**5.**	**Check for Minimum Reinforcement** $A_{st,min} = (0.85*b_w*d)/f_y$ $= (0.85*230*405)/415$ $= 190.787\ mm^2 < A_{st}\ (1976.896\ mm^2)$ **O.K.**	
IS 456-2000 Clause 26.5.1.1.b	**6.**	**Check for Maximum Reinforcement** $A_{st,max} = 0.04*b_w*D = 0.04*230*450$ $= 4140\ mm^2 >$ Ast $(= 1976.896\ mm^2)$ **O.K.** Now, Provide tension steel = 2 Nos. 12 mm ϕ bars and 9 Nos. 16 mm ϕ bars Hence, actual steel area provided = $2035.752\ mm^2$	

		Design of beam at mid section (T-beam design)	
IS 456-2000 Clause 22.2.b	**7.**	**Calculation of effective span (l_{eff})** We have, c/c span of beam, L = 3353 mm From code, for continuous beam, Clear span = 3353 – 450 = 2903 mm Since, Support width = 450 mm > [1/12*clear span =1/12*2903 = 242.91 mm or 600 mm whichever is less] Then, Effective span (l_{eff}) = clear span = 2903 mm Hence, Distance between zero moments, $l_o = 0.7*l_{eff}$ = 0.7*2903 = 2032.1 mm	
IS 456-2000 Clause 23.1.2	**8.**	**Calculation of flange width (b_f)** Depth of flange (D_f) = Overall depth of slab = 135 mm $\therefore b_f = l_o/6 + b_w + 6D_f$ = 2032.1/6 + 230 + 6*135 = 1378.683 mm	
IS 456-2000 Annex G.2.2.1	**9.**	**Calculation of Limiting Moment** D_f/d = 135/405 = 0.333 > 0.2 $M_{u,lim} = 0.36f_{ck}*x_{u,l}*b_w*(d-0.416x_{u,l})+0.446f_{ck}(b_f-b_w)y_f*(d-y_f/2)$ Where, yf = (0.15xu,l + 0.65Df) ≤ D_f = (0.15*193.995 + 0.65*135) = 116.849 mm ≤ D_f (=135 mm) Mu,lim = 0.36*20*193.995*230(405 – 0.416*193.995) + 0.446*20*(1378.683-230)*116.849* (405-116.849/2) = 519125070.7 N-mm = 519.125 KN-m Since, $M_{u,given}$ = 17.404 KN-m < $M_{u,lim}$ = 519.125 KN-m Hence, Singly reinforced section	
IS 456-2000 Annex G	**10.**	**Calculation of Actual Depth of Neutral Axis** $M_u = 0.36f_{ck}*x_u*b_f(d-0.416*x_u)$ 17.404*106 = 0.36*20*x_u*1378.6833*405 - 0.416*x_u) 1753.283 = 405x_u - 0.416x_u^2 0.416x_u^2 – 405x_u + 1753.283 = 0 $\therefore x_u$ = 4.348 mm < D_f (= 135 mm) Hence, Neutral Axis lies in the flange	
IS 456-2000	**11.**	**Calculation of Tension Steel Reinforcement** $M_u = 0.87f_yA_{st}(d – 0.416x_u)$ 17.404*106 = 0.87*415A_{st}(405 – 0.416*4.348) $\therefore$ Ast = 119.56 mm^2	

Annex G IS 456-2000 Clause 26.5.1.1.a	**12.**	**Check for Minimum Reinforcement** $A_{st,,min} = (0.85*b_w*d)/f_y = (0.85*230*405)/415$ $= 190.789\ mm^2 > A_{st}\ (= 119.56mm^2)$ Hence, Not safe So, Adopt Minimum steel area, $A_{st} = 190.789\ mm^2$	
   IS 456-2000 Clause 26.5.1.1.a	**13.**	**Check for Maximum Reinforcement** $A_{st,max} = 0.04*b_w*D = 0.04*230*450$ $= 4140\ mm^2 > A_{st}\ (= 190.789\ mm^2)$ **So, O.K.** Now, Provide tension steel = 2 Nos. 12 mm ϕ bars ∴ Actual steel area provided = 226.1947 mm^2	
IS 456-2000 Clause 40.1 IS 456-2000 Table 19 IS 456-2000 Clause 40.4.c	**14.**	**Design of Shear Reinforcement** At support Shear force, $V_u = 157.479$ KN Nominal shear stress, $\tau_v = V_u/bd$ $= 157.479*103/(230*405)$ $= 1.6906\ N/mm^2$ Ast provided, 2035.732 mm^2 Percentage of steel = $100A_s/bd$ $= (100*2035.752)/(230*405)$ $= 2.185\ \%$ For M20 concrete and % of steel, $\tau_c = 0.8066\ N/mm^2$ For M20 concrete, $\tau_{c,max} = 2.8$ N/mm2 Since, $\tau_{c,max} = 2.8N/mm^2 > \tau_v\ (= 1.6906\ N/mm^2) > \tau\ (=0.8066\ N/mm^2)$ **So,** Design of Shear reinforcement is required Providing 8 mm dia. Bars as 2-legged vertical stirrup Hence, Area of stirrups, $A_{sv} = 2*\pi*8^2/4 = 100.53\ mm^2$ Shear strength, $V_{us} = V_u - \tau_c*b_w*d$ $= (157.479*1000 - 0.8066*230*405)$ $= 82344.21$ N And, Spacing of stirrups, $S_v = (0.87*f_y*A_{sv}*d)/V_{us}$ $= (0.87*415*100.53*405)/82344.21$ $= 178.52\ mm \approx 178$ mm c/c	
IS 456-2000 Clause 40.4.a (pg.73) IS 456-2000 Clause 26.5.1.5	**15.**	**Check for Maximum spacing** The maximum spacing of shear reinforcement along the axis of member shall not exceed following a) 0.75*d = 0.75*405 = 307.75 mm b) 300 mm (Whichever smaller) Since, S_v = 178 mm c/c < (300 mm) **O.K.**	

	16.	**Check for Development Length**	
		$A_{st} = 2035.752\ mm^2$	
		$M_1 = 0.87*f_y*A_{st}\left(d - \frac{f_y * A_{st}}{b_w * f_{ck}}\right)$	
		$= 0.87*415*2035.752\left(405 - \frac{415*2035.752}{230*20}\right)$	
IS 456-2000 Annex G		$= 162.686*10^6$	
		$= 162.686$ KN-m	
		$V_u = 157.479$ KN	
		Condition to be satisfied:	
		$L_d \leq 1.3M_1/V + L_o$	
		$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} = \frac{0.87*f_y\phi}{4\tau_{bd}} = \frac{0.87*415*16}{4*1.2*1.6} = 752.1875$ mm	
IS 456-2000 Clause 26.2.1		$1.3M_1/V_u + L_o = (1.3*162.686*106)/(157.479*103) + d$ (d=405mm)	
		$= 1342.98 + 405 = 1747.98$mm	
IS 456-2000 Clause 26.2.3.3.c		So, L_d (= 752.18 mm) < $(1.3M_1/V_u + L_o)$(=1747.38mm) **So, O.K.**	
	17.	**Check for Deflection**	
		Condition to be satisfied	
		$l_{eff}/d \leq \alpha\beta\gamma\delta\lambda$	
		$l_{eff}/d = 2903/405 = 7.168$	
		$\alpha = 26$ for continuous beam	
IS 456-2000 Clause 23.2 (fig. 4)		$\beta = 1$ for span less than 10 m	
		$\gamma = 0.55 + \frac{477 - f_s}{120\left(0.9 + \frac{M}{b_w d^2}\right)} \leq 2$	
BS 8110-1985 Clause 3.4.5.6		where, $f_s = 0.58*f_y*\frac{A_s(required)}{A_s(provided)}*1/\beta_b$	
		$= 0.85*415*{}^{119.5572}/_{226.1947}*1/1$	
		$= 127.224$	
(A.K. Jain Pg. 187)		So, $\gamma = 0.55 + \frac{477 - 127.224_s}{120\left(0.9 + \frac{17.404*10^6}{230*405^2}\right)} = 2.69$	
		Hence, Adopt Max. $\gamma = 2$	
		$\delta = 1$	
		$\lambda = 0.8$	
		$\therefore \alpha\beta\gamma\delta\lambda = 26*1*2*1*0.8 = 41.6$	
		Since, $l_{eff}/d = 7.168 < 41.6$ (= $\alpha\beta\gamma\delta\lambda$) **So, O.K.**	

4.3 DESIGN OF COMPRESSION MEMBER

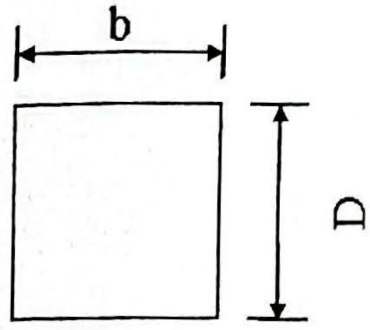


Fig. Square Column

Column may be defined as a structural element used primarily to support axial compressive loads and with a height of at least three times its dimension. A compression member subject to pure axial load rarely occurs in practice. All columns are subjected to same moment which may be due to accidental eccentricity or due to end restraint imposed by monolithically placed beams or slabs. The strength of columns depends on the strength of materials, shape and size of the cross-section, length and the degree of positional and directional restraints at its ends.

SUBJECTED TO BIAXIAL BENDING:

Exact design of members subject to axial load and biaxial bending is extremely laborious. Therefore, the IS Code 456: 2000 permits the design of such members by the following equation:

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \leq 1.0$$

Where,

M_{ux}, M_{uy} are the moments about x and y axes respectively due to design loads M_{ux1}, M_{uy1} are the maximum uniaxial moment capacities with an axial load P_u, bending about x and y axes respectively and α_n is an exponent whose value depends on P_u/P_{uz} (see table below) where,

$$P_{uz} = 0.45*f_{ck}*Ac + 0.75*f_y*A_{st}$$

Where,

A_c = Gross X-section area of column

A_{st} = Area of reinforcement bars.

f_{ck} = Characteristics strength of Concrete.

f_y = Characteristics strength of reinforcement bar (rebar).

TABLE

P_u/P_{uz}	α_n
≤ 0.2	1.0
≥ 0.8	2.0

For intermediate values, linear interpolation may be done. Chart 63 can be used for evaluating P_{uz}. For different values of Pu/P_{uz} the appropriate values of α_n has been taken and curves for the equation

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = 1.0 \text{ has been plotted in Chart 64}$$

Material Characteristics:

Concrete Grade: M20

Steel Grade Fe: 415

Section Characteristics:

Size of column = 450*450 mm^2

Effective Cover (d') = 40 mm

Effective Depth (d) = 450 -40 = 410 mm

$\frac{d'}{D} = 0.089 \approx 0.1$ for all Column

<u>Sample Calculation</u>

Reference	Step	Calculation	Remarks
	1.	**Design Parameters:** Factored load, P_u = 245.095 KN Factored moment , M_{ux} = 414.947 KN-m M_{uy} = 30.731 KN-m Reinforcement is distributed equally on four sides.	All the Design Parameters are taken from the output of Structural Analysis Program.
IS 456:2000 Cl. 25. 1.2	2.	**Slenderness Ratio Consideration:** $\frac{L_e}{D} = \frac{3156}{450} = 7.01 \leq 12$ Hence, the Column is Short Column	
IS 456:2000 Cl. 25. 4	3.	**Eccentricity Consideration:** Minimum Eccentricity: $e_{min} = \frac{l}{500} + \frac{b}{30}$ or 2.0 cm whichever is less	

		Where, l is unsupported length of column b is lateral dimension of column. $e_{min} = \frac{3156}{500} + \frac{450}{30} = 21.31mm$ or 2.0cm Thus, $e_{min} = 2.0$ cm Eccentricity due to design moment, e $= M_u / P_u$ $= \frac{30.731 * 10^6}{245.095 * 10^3}$ =125.38mm = 12.53cm	
IS 456:2000 Cl. 39.3		Eccentricity due to design moment is greater than the minimum eccentricity and 0.05 times the lateral dimension. Hence the column is design for given axial load and biaxial moments.	
	4.	**Calculation of % Reinforcement:** Assume percentage reinforcement (p_t) = 2% Then, $\frac{p_t}{f_{ck}} = 0.1$	
IS 456:1978 (SP - 16) CHART 44		$\frac{P_u}{f_{ck} * b * D} = \frac{245.095 * 10^3}{20 * 450 * 450} = 0.061$ $\frac{M_u}{f_{ck} * b * D} = 0.145$ $M_{ux1} - M_{uy1} = 264.262$ KN-m	
	5.	**Calculations:** $P_{uz} = 0.45 * f_{ck} * A_c + 0.75 * f_y * A_{st}$ $= 0.45*20*450*450+0.75*415*(2/100*450^2)$ = 3083.063 KN $\frac{P_u}{P_{uz}} = \frac{245.095}{3083.063} = 0.079$ For this α_n is interpolated from Table - 1	
IS 456: 2000 Cl.39.6		$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \leq 1.0$ $\left(\frac{141.947}{264.262}\right)^{0.798} + \left(\frac{30.731}{264.262}\right)^{0.798} = 0.788 < 1.0$ **O.K.** Hence 2% reinforcement is provided for the given column. A_{st} = 2% of bD $= 2/100*450^2$ $= 4050mm^2$	

		Provide 4-25mmΦ +4-28mmΦ bars Provided A_{st} = 4456.504 mm^2	
	6.	**Check for % steel:** $P_t = \frac{4426.504}{450 * 450} * 100$ = 2.2% 0.8 %< 2.2 %< 4% **O.K.**	
IS 456: 2000 Cl. 26.5.3.2C	**7.**	**Design for Transverse Reinforcement:** Dia. of Transverse reinforcement, Φ_t < 6mm < 1/4 times larger longitudinal bar = 28/4 = 7m Provided dia. of transverse reinforcement = 8mm Φ bars.	
	8.	**Spacing of Transverse reinforcement:** Spacing is minimum of: S_v> minimum dimension of column i.e.450mm > 16 times smaller dia. of longitudinal bar i.e. 16*25 = 400mm >300mm whichever is less. Provided 8mmΦ bars @ 300mm c/c. **O.K.**	
IS 456: 2000 Table 19 BS 8110	**9.**	**Shear Check in column** M_{1x} (Moment at top of the column) = 3.36 KN-m M_{2x} (Moment at bottom of the column) =7.41 KN-m P_u = 1356 KN Column height h= 3.61 m ⇒ $V = (M_{1x} + M_{2x})/h = (3.36 + 7.41)/3.61 = 2.98$ KN Shear stress V or $\tau_v = V/bd = 2.98*10^3/450^2$ $= 0.014$ N/mm^2 ⇒ Find τ_c' value: b = 450 mm, d = 450 mm P_t = 2% (assumed) τ_c for f_{ck} (= 20 N/mm^2) = 0.79 N/mm^2 ⇒ Increased shear due to compression $\tau_c' = \tau_c + 0.75(P/A_c)(Vd/M) \leq 0.8\sqrt{f_{ck}}$ or 5 N/mm^2 So, $\tau_c' = 0.79+0.75(1356*10^3/450^2)(2.98*10^3*450/7.41*10^6)$ $= 1.699 \approx 1.7$ N/mm^2 ⇒ **Check against maximum shear** $\tau_{c,max} = 0.8\sqrt{f_{ck}} = 0.8\sqrt{20} = 3.58$ N/mm^2 < 5 N/mm^2 $\tau_v < \tau_c' < \tau_{c,max}$ Hence, section is safe in shear, Only nominal shear steel is required.	

4.4 DESIGN OF STAIRCASE

Staircase is an inclined structural system to provide pedestrian access to different levels within a building. The geometrical forms of staircase may be quiet different depending on the individual circumstances involved. The stairs and landing slab can angled in different form to get different types of staircase. The shape and structural arrangement of the staircase would generally depend on two main factors.

a) Type of construction of the structure around the staircase

b) Availability of space

The value of riser and tread to be adopted depends upon the type and use of the building.

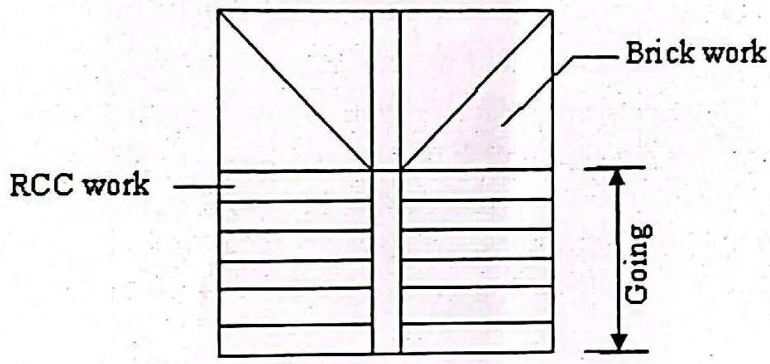


Fig. Open well staircase

Sample Calculation for Open well staircase

Reference	Step	Calculation	Remarks
	1	**For inclined portion** $L_e = 3.353$m 1) Superimposed load = 5 KN/m2 Dead load 2) Finishes = 27*0.015 + 21*0.025 = 0.93 KN/m2 Fy = 415, fck = 20 Floor ht. = 3.302 m No. of riser = 18 ∴ Riser (R) = Floor height/No. of riser = 3.302/18 = 183 mm Tread (T) = 12" = 304.8 mm Slab thickness (Waist slab) = span/20 = 3.353/20 = 167.65 mm So, Adopt (D) = 175 mm $X = \sqrt{((304.8)^2 + (183)^2)} = 356$ mm	

3) Self wt. of slab = D*0.356*25 = 1.56 KN/m = 1.56/0.3048 = 5.12 KN/m^2

4) Self wt. of step = 1/2 *0.183*0.3048*25 = 0.7 KN/m = 0.7/3.048 = 2.3 KN/m^2

Total load = 5 + (0.93 ≈ 1) + 5.12 +2.3 = 13.42 KN/m^2

Taking 1 m width of flight

Total wt. (W) = 1*13.42 KN/m

W_U = 1.5*13.42 = **20.13 KN/m**

2 **For landing zone**

Live load = 5 KN/m^2 (Assuming 1m of width of slab)

Finishes = 1 KN/m^2

Self wt. of brick step = 1/2*1.3716*1.524*(1.83 – 0.025 – 0.015)*19 = 2.84 KN

∴ UDL = 2.84/1.524 = 1.86 KN/m

Slab wt. = 5.12 KN/m^2 = 5.12 KN/m

Hence, Total wt. (W) = 12.98 KN/m

W_U = 1.5*12.98 = 19.47 KN/m

$R_A = R_B$ = Wl/2 = 33.75 KN

So, Maximum Moment = Wl^2/8 = (20.13*3.353)^2/8

∴ M_u = 28.29 KN-m

Check for depth

IS 456-2000 Annex G

$M_u = 0.138 f_{ck} b d^2$

$$\text{So, } d = \sqrt{\frac{28.29*10^6}{0.138*20*1000}}$$

IS 456-2000 Annex G

= 101.24 mm < 175 mm **O.K.**

Adopt d = 120 mm and D = 150 mm

$$\therefore M_u = 0.87*f_y*A_{st}\left(d - \frac{f_y A_{st}}{f_{ck} b}\right)$$

$$28.29*10^6 = 0.87*415*A_{st}\left(120 - \frac{415*A_{st}}{20*1000}\right)$$

Hence, Ast = 750.3 mm^2

Providing the 12 mm dia. Bar

$$\text{So, No. of bars} = \frac{750.3}{\pi*12^2/4} = 6.63 \approx 7 \text{ No.}$$

Provide 12 mm φ bars @ 140 mm c/c

Check for shear

IS 456-2000 Clause 40.1

τ_v = Nominal shear stress

= V_u/bd

= (33.75*1000)/(1000*120) = 0.2812 N/mm^2

IS 456-2000 Table 19

Steel Percentage = $100A_{st}/bd = \dfrac{100*7*\pi*12^2}{1000*120*4} = 0.66\%$

For M20 concrete

Design shear strength (τ_c) = 0.5312 N/mm^2

IS 456-2000 Clause 40.2.1.1

$\tau_c' = k*\tau_c = 1.3*0.5312 = 0.69$ N/mm^2

$\tau_v < \tau_c'$ **O.K.** (No. shear reinforcement required)

<u>Check for Development length</u>

IS 456-2000 Clause 26.2.1

$$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} = \frac{0.87*f_y\phi}{4\tau_{bd}} = \frac{0.87*415*\phi}{4*1.2*1.6} = 47\phi$$

$L_d = 47*12 = 564$ mm

IS 456-2000 Clause 26.2.3.3.c

$L_d = 1.3M_1/V + L_o$ (let L_o anchorage length = 0)

$= (1.3*28.29*10^6)/(33.75*10^3) + 0$

$= 1089.69$ mm

Since, $47\phi < 1.3M_1/V$ **O.K.**

3 **Cantilever portion**

Total wt. of slab = 0.15*25*1.524*3.048 = 17.45 KN

Total wt. of brick step = 2*1/2*1.524*1.3716*.183*19

= 7.268 KN

Total wt. of the RCC step = ½*0.3048*0.183*1.524*25

= 1.06KN

Hence, Total wt. = 25.778 KN

And Ultimate load = 25.778*1.5 = 38.667 KN

UDL (W_u) = 38.667/(1.524*3.048) = 8.324 KN/m

$M_u = W_ul^2/2 = (8.324*1.524^2)/2 = 9.66$ KN-m

<u>Check for depth</u>

IS 456-2000 Annex G

$M_u = 0.138f_{ck}bd^2$

$$\text{So, } d = \sqrt{\frac{9.66*10^6}{0.138*20*1000}}$$

= 59.16 mm < 175 mm **O.K.**

Adopt d = 120 mm and D = 150 mm

<u>To find area of steel</u>

IS 456-2000 Annex G

$$\therefore M_u = 0.87*f_y*A_{st}\left(d - \frac{f_yA_{st}}{f_{ck}b}\right)$$

$$9.66*10^6 = 0.87*415*A_{st}\left(120 - \frac{415*A_{st}}{20*1000}\right)$$

Hence, Ast = 232.3 mm^2

Providing the 12 mm dia. Bar

		So, No. of bars = $\frac{232.3}{\pi * 12^2 / 4}$ = 2.053 ≈ 3 No.	
		Spacing = 1000/3 = 333.33 mm	
		Adopt spacing = 300 mm	
		Provide 12 mm ϕ bars @ 300 mm c/c	
		Check for shear	
		τ_v = Nominal shear stress	
IS 456-2000 Clause 40.1		= V_u/bd ($V_u = W_u l$ = 8.324*1.524 = 12.685 KN)	
		= (12.685*1000)/(1000*120) = 0.105 N/mm^2	
		Steel Percentage = $100A_{st}/bd$ = $\frac{100 * 3 * \pi * 12^2}{1000 * 120 * 4}$ = 0.282%	
		For M20 concrete	
IS 456-2000 Table 19		Design shear strength (τ_c) = 0.375 N/mm^2	
IS 456-2000 Clause 40.2.1.1		$\tau_c' = k * \tau_c$ = 1.3*0.375 = 0.4875 N/mm^2	
		$\tau_v < \tau_c'$ **O.K.** (No. shear reinforcement required)	
		Check for Development length	
IS 456-2000 Clause 26.2.1		$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} = \frac{0.87 * f_y \phi}{4\tau_{bd}} = \frac{0.87 * 415 * \phi}{4 * 1.2 * 1.6} = 47\phi$	
		L_d = 47*12 = 564 mm	
IS 456-2000 Clause 26.2.3.3.c		$L_d = 1.3M_1/V + L_o$ (let L_o anchorage length = 0)	
		= (1.3*9.66*10^6)/(12.685*10^3) + 0	
		= 990.63 mm	
		Since, 47ϕ < 1.3M$_1$/V **O.K.**	

4.5 DESIGN OF MAT FOUNDATION

Foundations are structural elements that transfers loads from the buildings or individual columns to the earth. If these loads are to be properly transmitted, foundation must be designed to prevent excessive settlement or rotation to minimize differential settlement and to provide adequate safety against sliding and overturning. If the loads transmitted by the columns in the structure are so heavy or the allowable soil pressure so small that individual footings would cover more than 50% of the whole area it may be better to provide continuous footing under all columns and walls. Such a footing is called a raft or mat foundation. The raft is designed by analyzing it as a series of continuous strips centered on the appropriate column rows in both directions.

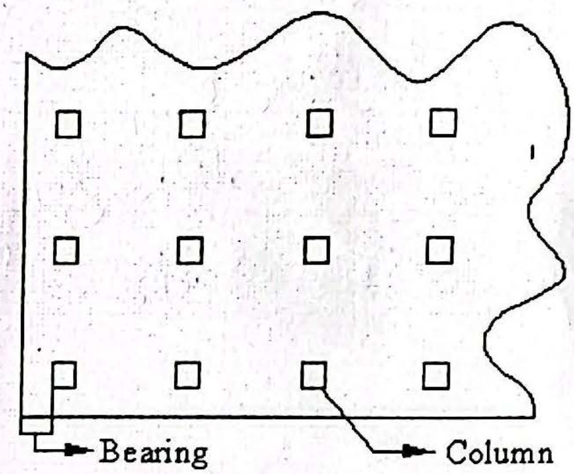


Fig. Mat Foundation (Plan)

Total vertical column load = 42846.30KN

Area of mat = 170.032+3.349+23.414+27.574+38

= 614.321 m^2

Centroid of the plan ($\bar{x}$, $\bar{y}$) is,

$$\bar{x} = \sum_{i=1}^{n} \frac{A_i * x_i}{A_i} = 17.729\text{m}$$

$$\bar{y} = \sum_{i=1}^{n} \frac{A_i * y_i}{A_i} = 9.436\text{m}$$

Moment of inertia about both centroidal x and y axes using parallel axis theorem.

$$I_{xx} = \sum_{i=1}^{n} \left[(I_{xxc})_i + A_i * y_i^2 \right] = 20427.096 \text{ m}^4$$

$$I_{yy} = \sum_{i=1}^{n} \left[(I_{yyc})_i + A_i * x_i^2 \right] = 62866.098 \text{ m}^4$$

Centroid due to load ($\bar{x}_1$, $\bar{y}_1$) is,

$\bar{x}_1$ = 18.4765m

$\bar{y}_1$ = 9.124m

Now,

e_x = 18.4765-17.729 = 0.7475m

e_y = 9.124-9.435 = -0.311m

$M_x = P*e_y$ = 42846.3*(-0.311) = -13325.199KN-m

$M_y = P*e_x$ = 42846.3*0.7475 = 32027.609 KN-m

$$\frac{P}{A} = \frac{42846.30}{614.321} = 69.746 \text{KN/m}^2$$

Soil Pressure at different points are given by,

$$\sigma = \frac{P}{A} + \frac{M_y}{I_y} * x + \frac{M_x}{I_x} * y$$

For corner A-2,

$\sigma = 69.746 - 7.013 + 6.155 = 68.888$ KN/m^2

Other data are shown in tabular form below,

Position	x(m)	y(m)	σ (KN/M^2)
A-2	-13.766	-9.435	68.888
A-3	-8.519	-9.435	71.561
A-4	-4.556	-9.435	73.580
A-5	-1.203	-9.435	75.288
A-6	3.065	-9.435	77.462
A-7	7.333	-9.435	79.637
A-8	11.601	-9.435	81.811
A-9	15.869	-9.435	83.985
A-10	19.897	-9.435	86.037
B-2	-13.766	-5.407	66.260
B-10	19.897	-5.407	83.410
C-2	-13.766	-1.139	63.476
C-10	19.897	-1.139	80.626
D-1	-17.729	2.149	59.312
D-2	-13.766	2.149	61.331
D-7	7.333	2.149	72.08
D-8	11.601	2.149	74.254
D-9	15.869	2.149	76.429
D-10	19.869	2.149	78.481
E-6	2.84	6.246	67.11
E-7	8.008	6.246	69.754
F-1	-17.729	8.006	55.491
F-5	-0.528	8.006	64.254
G-1	-17.729	12.034	52.864
G-2	-13.766	12.034	54.883
G-3	-8.519	12.034	57.556
G-4	-4.556	12.034	59.815
G-4	-0.528	12.034	61.627

The depth of the mat foundation shall be governed by two way shear also known as punching shear. In case location of critical shear is not obvious it may be necessary to check in all possible direction.

Shear strength of concrete,

$$\tau_c = \tau_c = 0.25\sqrt{f_{ck}}$$

$$= 0.25\sqrt{20}$$

$$= 1.118 \text{ N/mm}^2$$

1) For column A-2:

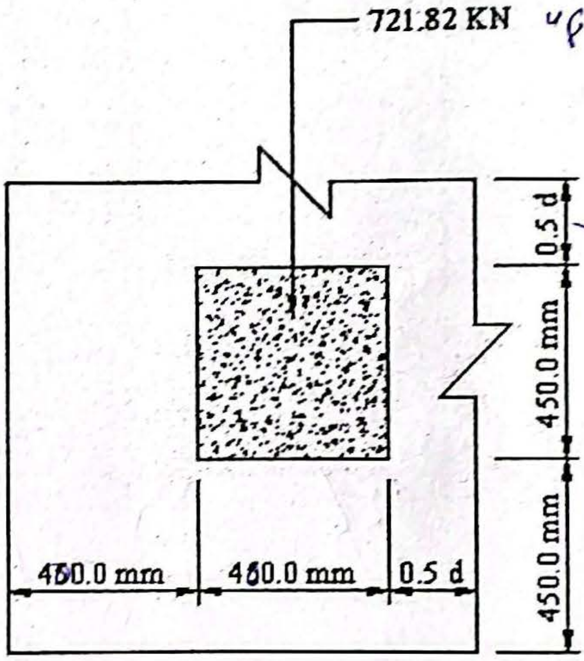


Fig. A-2

Perimeter $b_o = 2*(450+450+0.5d)$

$= (1800+d)$

$$\tau_c = \frac{V_u}{bd}$$

Where,

V_u = Max. Ultimate force at corner edge column.

$$1.118 = \frac{1.5 * 721.82 * 1000}{(1800+d)*d}$$

Thus, d = 412.406mm

2) For column C-5:

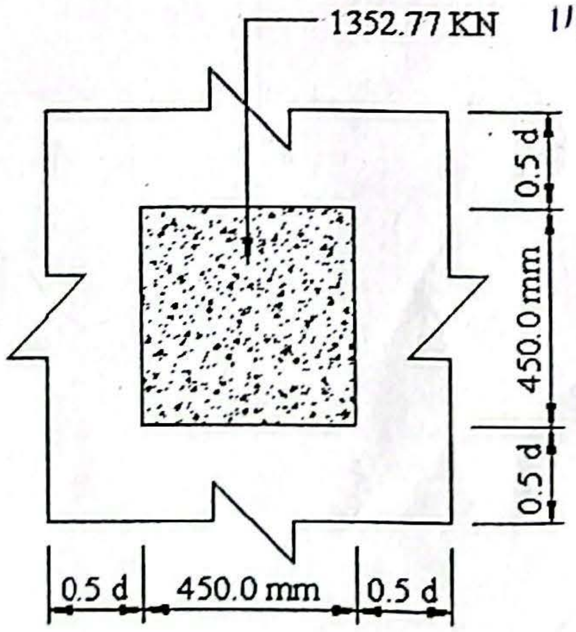


Fig. C-5

Perimeter, $b_o = 4*(450+d)$

$= (1800+d)$

$$\tau_c = \frac{V_u}{bd}$$

$$1.118 = \frac{1.5 * 1352.77 * 1000}{(1800+d)*d}$$

Thus, d = 485.191 mm

3) For column D-2:

Perimeter, b_o = (0.5d+450+0.5d
)*2+(0.5d+450+450)*2
= (2700+3d)

$$\tau_c = \frac{V_u}{bd}$$

$$1.118 = \frac{1.5 * 1210.22 * 1000}{(2700 + 3d) * d}$$

Thus, d = 412.406mm

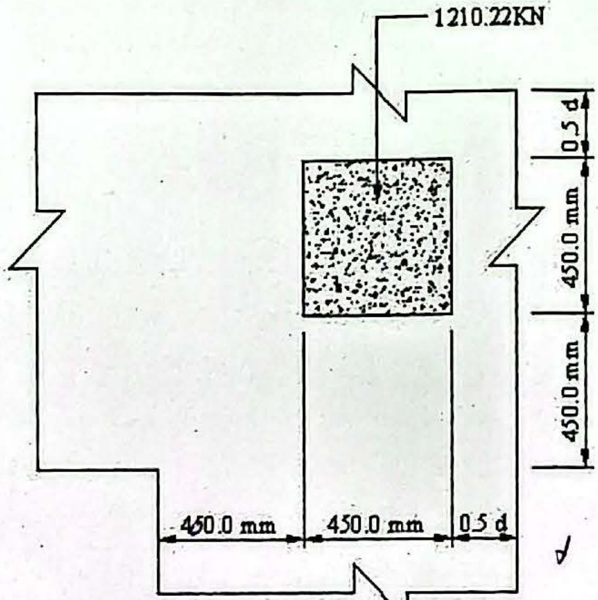


Fig. D-2

4) For column B-2 :

Perimeter, b_o = 2*(450+450+0.5d)+(0.5d+0.5d+450)
= (2250+2d)

$$\tau_c = \frac{V_u}{bd}$$

$$1.118 = \frac{1.5 * 1007.50 * 1000}{(2250 + 2d) * d}$$

Thus, d = 433.632mm

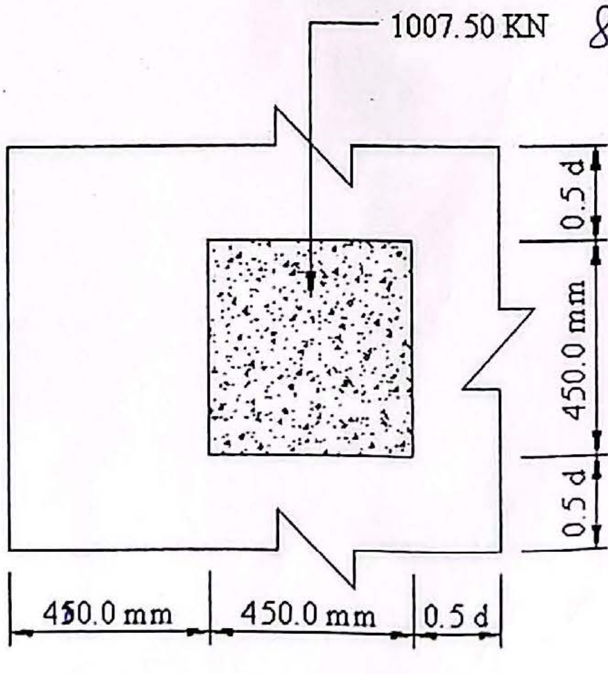


Fig. B-2

5) For column F-5 :

Perimeter b_o= (d+450)+(900+0.5d)+(0.5d+450+761.85+0.8465d)

= (2561.85+2.8465d)

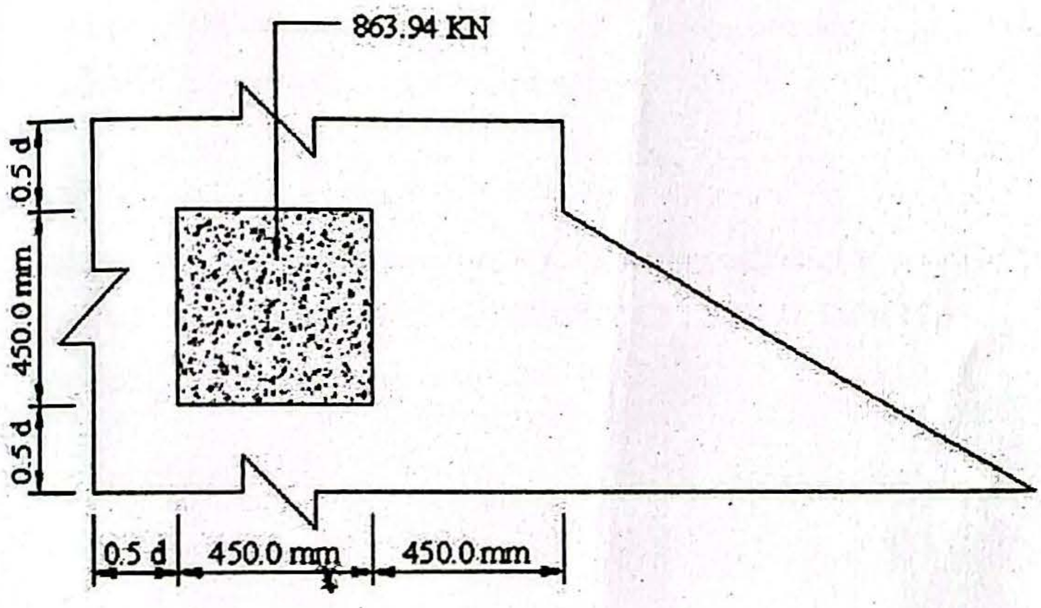


Fig. F-5

$$\tau_c = \frac{V_u}{bd}$$

$$1.118 = \frac{1.5 * 863.99 * 1000}{(2561.85 + 2.8465d) * d}$$

Thus, d = 330.856mm

Thus,

Adopt effective depth = 500mm

Adopt overall depth = 500+2*75 = 650mm

(Taking eff. cover = 75mm)

Calculation of required area of Rebar:

We have,

$$BM = 0.87 * f_y * A_{st} * \left[d - \frac{f_y * A_{st}}{f_{ck} * b} \right]$$

Here,

$d = 500mm$

$b = 1000mm$

$f_y = 415$ Mpa

$f_{ck} = 20$ Mpa

The loading for BM calculation is assumed as UDL having intensity W , which is the max. of intensities at the edges of the corresponding strip.

Along X - direction:

The bending moment (BM) was obtained using the coefficient 1/10 and length (l) as center to center of column distance. (IS: 456:2000 CI.22.5.1121 Table 12)

<u>For strip A-A:</u>

$W = 86.037$ KN/m^2

$$BM = \frac{Wl^2}{10} = \frac{86.037 * 4.572^2}{10} = 179.845 \text{ KN-m per m width}$$

$A_{st} = 1041.225$ mm^2

<u>For strip B-B:</u>

$W = 83.410$ KN/m^2

$$BM = \frac{Wl^2}{10} = \frac{83.410 * 4.572^2}{10} = 174.353 \text{ KN-m per m width}$$

$A_{st} = 1007.975$ mm^2

<u>For strip C-C :</u>

$W = 80.626$ KN/m^2

$$BM = \frac{Wl^2}{10} = \frac{80.636 * 4.572^2}{10} = 168.534 \text{ KN-m per m width}$$

$A_{st} = 972.8584$ mm^2

<u>For strip D-D:</u>

$W = 78.481$ KN/m^2

$$BM = \frac{Wl^2}{10} = \frac{78.481 * 4.572^2}{10} = 164.050 \text{ KN-m per m width}$$

$A_{st} = 945.867$ mm^2

For strip E-E:

W = 69.754 KN/m^2

$$BM = \frac{Wl^2}{10} = \frac{69.754 * 4.572^2}{10} = 145.808 \text{ KN-m per m width}$$

A_{st} = 836.745 mm^2

For strip D-D:

W = 78.481 KN/m^2

$$BM = \frac{Wl^2}{10} = \frac{78.481 * 4.572^2}{10} = 164.050 \text{ KN-m per m width}$$

A_{st} = 945.867 mm^2

For strip E-E:

W = 69.754 KN/m^2

$$BM = \frac{Wl^2}{10} = \frac{69.754 * 4.572^2}{10} = 145.808 \text{ KN-m per m width}$$

A_{st} = 836.745 mm^2

For strip F-F:

W = 64.254 KN/m^2

$$BM = \frac{Wl^2}{10} = \frac{64.254 * 4.572^2}{10} = 134.311 \text{ KN-m per m width}$$

A_{st} = 768.513 mm^2

For strip G-G:

W = 61.627 KN/m^2

$$BM = \frac{Wl^2}{10} = \frac{61.627 * 4.572^2}{10} = 128.820 \text{ KN-m per m width}$$

A_{st} = 736.070 mm^2

Along Y - direction:

The bending moment (BM) was obtained by using coefficient 1/12 and length (l) as center to center of column distance (IS: 456: 2000 Cl.22.5.1121 Table 12)

<u>For strip 1-1 :</u>

$W = 59.312\ KN/m^2$

$$BM = \frac{59.312 * 5.182^2}{12} = 132.726 \text{ KN-m per m width}$$

$A_{st} = 759.138\ mm^2$

<u>For strip 2-2 :</u>

$W = 68.888\ KN/m^2$

$$BM = \frac{68.888 * 5.182^2}{12} = 154.155 \text{ KN-m per m width}$$

$A_{st} = 886.543\ mm^2$

<u>For strip 3-3 :</u>

$W = 71.561\ KN/m^2$

$$BM = \frac{71.561 * 5.182^2}{12} = 160.136 \text{ KN-m per m width}$$

$A_{st} = 922.363\ mm^2$

<u>For strip 4-4 :</u>

$W = 73.580\ KN/m^2$

$$BM = \frac{73.580 * 5.182^2}{12} = 164.654 \text{ KN-m per m width}$$

$A_{st} = 949.498\ mm^2$

<u>For strip 5-5 :</u>

$W = 75.288\ KN/m^2$

$$BM = \frac{75.288 * 5.182^2}{12} = 168.476 \text{ KN-m per m width}$$

$A_{st} = 972.505\ mm^2$

<u>For strip 6-6 :</u>

$W = 77.462\ KN/m^2$

$$BM = \frac{77.462 * 4.268^2}{12} = 1117.586 \text{ KN-m per m width}$$

$A_{st} = 669.984\ mm^2$

For strip 7-7 :

$W = 79.637\ KN/m^2$

$$BM = \frac{59.312 * 5.182^2}{12} = 154.155 \text{ KN-m per m width}$$

$A_{st} = 689.369\ mm^2$

For strip 8-8 :

$W = 81.811\ KN/m^2$

$$BM = \frac{81.811 * 4.268^2}{12} = 124.188 \text{ KN-m per m width}$$

$A_{st} = 708.775\ mm^2$

For strip 9-9 :

$W = 83.985\ KN/m^2$

$$BM = \frac{83.985 * 4.268^2}{12} = 127.488 \text{ KN-m per m width}$$

$A_{st} = 728.214\ mm^2$

For strip 10-10 :

$W = 86.037\ KN/m^2$

$$BM = \frac{86.037 * 4.268^2}{12} = 130.603 \text{ KN-m per m width}$$

$A_{st} = 746.594\ mm^2$

Summary

Required area of Rebar, A_{st} (in mm^2)

Along X - direction:

Strip	Strip	Strip	Strip	Strip	Strip	Strip
A-A	B-B	C-C	D-D	E-E	F-F	G-G
1041.225	1007.975	972.8545	945.867	836.745	768.513	736.070

Along Y - direction:

Strip	Strip	Strip	Strip	Strip
1-1	2-2	3-3	4-4	5-5
759.138	886.543	922.363	949.498	972.505

Strip	Strip	Strip	Strip	Strip
6-6	7-7	8-8	9-9	10-10
669.984	689.369	708.775	728.214	746.594

Calculation of Spacing:

Along x - direction:

Strip	Dia. provided (mm)	Spacing (mm)
A-A	16	199.58
B-B	16	207.81
C-C	16	217.26
D-D	16	226.14
E-E	16	263.78
F-F	16	295.506
G-G	16	313.42

Spacing of Grid A-A to D-D = 16 mm dia. @ 200mm c/c

Spacing of Grid E-E to G-G = 16 mm dia. @ 260 mm c/c

Along Y - direction:

Strip	Dia. Provided (mm)	Spacing (mm)
1-1	16	300.47
2-2	16	244.62
3-3	16	232.47
4-4	16	224.04
5-5	16	217.36
6-6	16	357.59
7-7	16	343.40
8-8	16	330.27
9-9	16	318.09
10-10	16	307.38

Spacing of Grid 1-1to 5-5 = 16 mm dia. @ 210mm c/c

Spacing of Grid 6-6 to 10-10 = 16 mm dia. @ 300 mm c/c

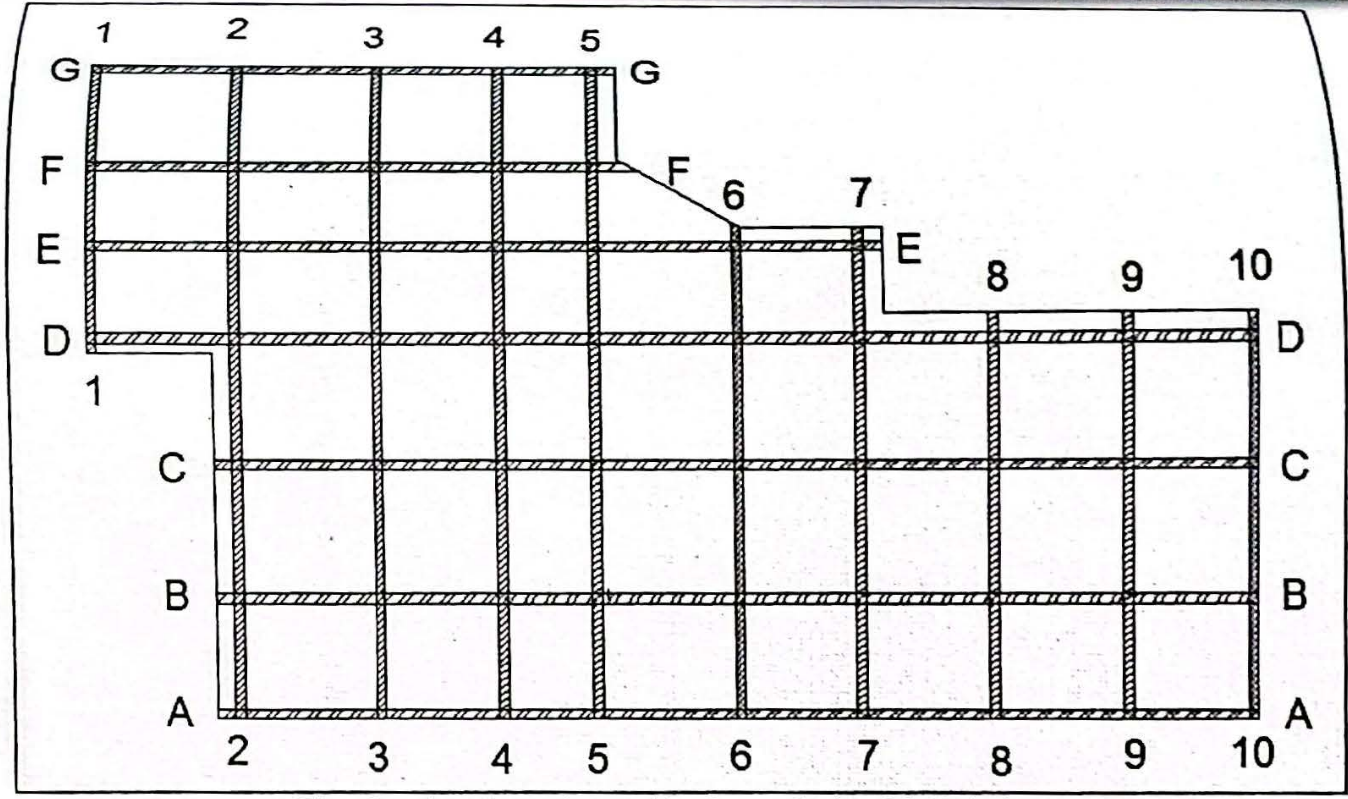


Fig. showing the strip used in the analysis of foundation

- Strips parallel to the Horizontal direction in the above figure are named in the alphabetical order.
- Strips parallel to the Vertical direction in the above figure are named in the numerical order.
- For the analysis of all intermediate strip, 1m strip is taken with its central line corresponding to the centre line of columns.
- For the analysis of the entire edge strip, 1m strip is taken inward from edge of the mat.

4.6 DESIGN OF BASEMENT WALL

Basement wall or retaining wall is a reinforced cement concrete structure generally built beneath the ground level. The main purpose of designing this structural member is to retain the lateral active pressures of earth, seepage as well as surcharge load due to vehicles. It is designed for both moment and shear criteria.

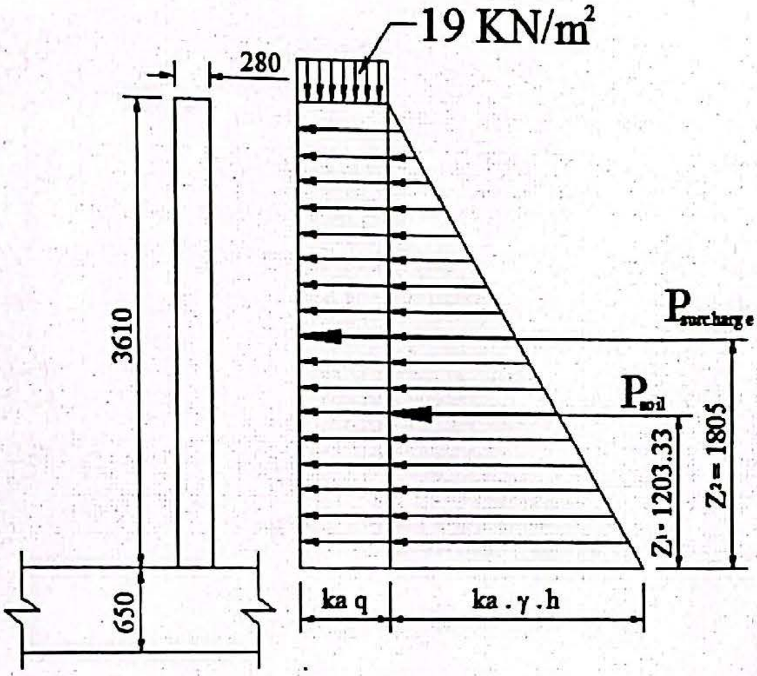


Fig. showing lateral active pressure distribution on basement wall

Design constants:

Cantilever retaining wall

f_y = 415 N/mm ^{2}

f_{ck} = 20 N/mm ^{2}

Specific wt. of soil (γ_s) = 19 KN/m^3

Angle of internal friction of soil (ϕ_s) = 30°

Height of wall (h) = 3.61 m

Specific wt. of water (γ_w) = 10 KN/m^3

Surcharge load (q) = 19KN/m^2 (Considering heavy vehicle like loaded truck)

Then, Coefficient of active earth pressure (k_a) is,

$$K_a = \left(\frac{1 - Sin\phi_s}{1 + Sin\phi_s}\right) = 1/3 = 0.333$$

Sample Calculation

Reference	Step	Calculation	Remarks
	1	**Calculation of Earth pressure** Considering pressure per meter width of wall, Total active earth pressure due to soil, $(Pa)_{soil} = (1/2*k_a\gamma_s h*h)$ $=(1/2*1/3*18*3.61^2)$ = 39.096 KN Lever arm $(Z)_{soil} = 1/3*h = 1/3*3.61 = 1.203$ m Total active earth pressure due to surcharge, $(Pa)_{sur} = (k_a q*h) = (1/3*19*3.61) = 22.863$ KN Lever arm $(Z)_{surcharge} = h/2 = (3.61/2) = 1.805$ m	
	2	**Moment about the base of wall** i) Due to soil pressure, (portion II in fig.) Moment due to soil pressure $(M)_{soil} = (Pa)_{soil}*(Z)_{soil}$ = 39.096*1.203 KN-m = <u>47.046 KN-m</u> ii) Due to surcharge load (portion I in fig.) Moment due to surcharge $(M)_{surcharge} = (Pa)_{surcharge}*(Z)_{soil}$ = 22.863*1.805 KN-m = <u>41.268 KN-m</u> ∴ Total moment at base (M) = 88.314 KN-m Ultimate Moment (M_{11}) = 1.5*88.314 **= 132.471 KN-m**	
IS 456-2000 Clause 32.2.3	3	**Thickness of wall** Assuming height/thickness ratio = 20 Height of basement wall (h) = 3.61m = 3610 mm Then, thickness of wall (d) = (h/20) = (3610/20) = 180.5 mm Total thickness (D) = d + cover + $\phi/2$ = (180.5 +50 +16/2) mm = 238.5 mm Adopt D = 250 mm Then, Thickness (d) = 250– 50 = 200 mm (effective)	
IS 456-2000 Annex G	4	**Calculation of Effective depth (d)** $d = \sqrt{\frac{M_u}{0.138bf_{ck}}} = \sqrt{\frac{132.47*10^6}{0.138*1000*20}} = 219.08$ mm So, Adopt **d = 230 mm** Overall depth **(D)** = d + 50 (= effective cover) = **280 mm**	
IS 456-2000 Annex G	5	**Calculation of Limiting Moment ($M_{u,lim}$)** $M_{u,\,lim} = 0.138 f_{ck} bd^2$ = 0.138*20*1000*230*230 = 146 KN-m Since, Mu,lim > Mu, design is singly reinforced	

6 Calculation of main reinforcement (Vertical)

Area of steel required per meter is given by

$$Mu = 0.87*fy*Ast*(d-\frac{f_y A_{st}}{f_{ck} b})$$

$$\text{i.e } 132.47*10*6 = 0.87*415*Ast\left(230 - \frac{415*A_{st}}{20*1000}\right)$$

IS 456-2000 Annex G

Hence, Ast = 1931.96 mm2

Providing 16 mm dia. Bars.

$$\text{No. of bars (N)} = \left(\frac{1931.96}{\pi*16^2/4}\right) = 9.6 \approx 10 \text{ No.}$$

Spacing required (S) = (Breadth – 2*cover – dia)/(N-1)

= (1000 – 2*50– 16)/(10-1) mm

= 100.8 mm ≈ 100 mm

Provide 16 mm ϕ bars @ 100 mm c/c

Provided area of steel = 2010.62 mm^2

7 Check for minimum reinforcement

IS 456-2000 Cl. 32.5

$A_{st,min}$ = 0.12 % of bD

= 0.12/100*1000*280

= 336 mm^2

$A_{st,min} < A_{st,\ provided}$ **O.K.**

8 Check for Max. dia.

Max. Dia. = 1/8*D = 1/8*280 = 35 mm > 16 mm **O.K.**

IS 456-2000 32.5.2.a

9 Check for maximum spacing

IS 456-2000 Cl.32.5.2.a

The maximum spacing of main reinforcement shall not exceed the following:

a. 3d = 3*230 = 690 mm

b. 450 mm

Provided spacing (100 mm) < Max. spacing (a & b)

O.K.

10 Area of distribution steel (Horizontal bar)

IS 456-2000 Cl.32.5 (1)

Providing minimum steel = 0.20% of hD

= (0.2/100)*3610*280 mm^2

= 2021.6 mm^2

As temperature changes occur at front face of retaining wall, 2/3rd of horizontal reinforcement is provided on front face and 1/3rd of horizontal reinforcement is provided on inner face

Front face horizontal reinforcement = 2/3*2021.6

= 1347.73 mm^2

Inner face horizontal reinforcement = 1/3*2021.6

= 673.86 mm^2

Providing 8 mm dia. Bars.

i) In front face, No. of bars (N) = $\left(\dfrac{1347.73}{\pi * 8^2/4}\right)$

$= 26.81 \approx 27$ No.

Spacing required (S) = (Height – 2*cover – dia.)/(N – 1)

= (3610 – 2*75 – 8)/(27 – 1)

= 130 mm

Provide 8 mm ϕ bars @ 130 mm c/c

ii) In inner face, No. of bars (N) = $\left(\dfrac{673.86}{\pi * 8^2/4}\right)$

$= 13.4 \approx 14$ No.

Spacing required (S) = (Height – 2*cover – dia.)/(N – 1)

= (3610 – 2*75 – 8)/(14 – 1)

= 265.53 mm

Provide 8 mm ϕ bars @ 260 mm c/c

11. Check for maximum spacing

IS 456-2000 Clause 32.5.(b &d)

The maximum spacing of main reinforcement shall not exceed the following:

a. 3d = 3*230 = 690 mm

b. 450 mm

Provided spacing (i & ii) < Max. spacing (a & b)

O.K.

12 Inner face vertical reinforcement

To support these horizontal bars on the inner face, vertical bars are used

IS 456 – 2000 Clause 32.5.a (1)

The maximum spacing of main reinforcement shall not exceed the following:

a. 3d = 3*230 = 690 mm

b. 450 mm

Provide 10 mm ϕ bars @ 450 mm c/c

13. Check for shear

Maximum shear occurs at support (fixed) i.e. at the base of wall

Then, V_{max} = (6.333*3.61 +½*21.6*3.61) = 61.850 KN

IS 456-2000 Clause 40.1

Nominal shear stress (τ_v) = V_u/bd

= (61.85*10^3)/(1000*230)

= 0.269 N/mm^2

As provided, area of steel = 2010.62 mm^2

So, 100A_s/bd = (2010.62*100*)/(1000*230)

= 0.874%

IS 456-2000 Table 19		Then, for M20 concrete (from Table 19) Design shear strength (τ_c) = 0.589 N/mm^2 (Interpolating the value in table with respect to M20 and value of 100A$_s$/bd) Here, $\tau_c > \tau_v$, So, design is safe in shear and shear reinforcement is not required. **Again,**	
	14.	**Curtailment of Vertical Reinforcement (Outer face)** No bars can be curtailed in less than L_d distance from the bottom Development length of bar,	
IS 456-2000 Clause 26.2.1		$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} = \frac{\phi * 0.87 * f_y}{4\tau_{bd}}$ For M20, $\tau_{bd} = 1.2*1.6 = 1.92$ $L_d = (0.87*415\ \emptyset)/(4*1.2*1.6)$ $= 47\emptyset = 47*16 = 752$ mm **Let** us curtail bars at 1/3rd distance from bottom i.e. 3.61/3 = 1.203 m $Z_3 = 3.61 - 1.203 = 2.407$ m $P'_{soil} = ½*k_a\gamma_s*Z_3^2 = ½*1/3*18*2.407^2*1$ = 17.38 KN per m Lever arm (Z_1') = 2.407/3 = 0.802 m $P'_{surcharge} = k_a*q*Z_3 = 1/3*19*2.407 = 15.24$ KN Lever arm (Z_2') = 2.407/2 = 1.203 m Taking moment at section i.e. 1/3rd height from base Moment = 17.38*0.802 + 15.24*1.203 = 32.27 KN-m Ultimate moment (M_{22}) = 1.5*32.27 = 48.40 KN-m Since, $M_{22} < 50\%$ of M_{11}, so, spacing of vertical bars are doubled from the height of 1/3rd of base of wall. Hence Adopt **16mm dia. bars @ 200 mm c/c**	