DESIGN AND ANALYSIS OF EARTHQUAKE RESISTANCE COMMERCIAL BUILDING

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Abstract: **Earthquake is a very serious problem since they harm human life and tall structures, building and all. So, proper construction is required with proper implementation of IS code. In this project, it includes the seismic analysis of structure for static and dynamic analysis in ordinary resisting frame and special moment resisting frame. The project will be carried out manually and will be validated by SAP 2000 software. The present paper deals with analysis and design of structures which resists earthquake and reduce some disastrous incidents.**

I. INTRODUCTION

This project work is for the **Structural Analysis and Design of twelve storied Earthquake Resistant Building**. As we have experienced the disastrous earthquake repeatedly in Nepal, this work has given opportunity to develop the integrated design concept for making the building earthquake resilient to the students. Structural analysis is the backbone of civil engineering. During recent years, there has been a growing emphasis on using computer aided software and tools to analyze the structures. The importance of this project is to be familiar to the increased use of the professional computer software in analysis and design. These developments are most welcome, as they relieve the engineer of the often lengthy calculations and procedures required to be followed while large or complicated structures are analyzed using classical methods. This project report covers the structural design works carried out which discuss, briefly, the design criteria, assumptions, analysis and design of different components of the building that will be able to resist the earthquake motions.

This project consists of structural analysis, design and structural dealing of a twelve storied building for earthquake resistance.

The general features of this proposed building are as follows:

1.	Name of project	Structural Analysis and Design of twelve stories Earthquake Resistant Building
2.	Utility of building	Commercial complex
3.	Location	Kathmandu, Nepal
4.	Structure system	RCC framed structure
5.	Plinth area	807.79 sq. m.
6.	No. of stories	2B + G +10 stories
7.	Floor to floor height	3.5 m
8.	Types of slab	two way (125 mm thickness)
9.	Types of beam	rectangular; 350 mm × 650 mm (Secondary beam 300 mm × 500 mm)
10.	Types of column	square; 650 mm ×650 mm
11.	Types of foundation	Matt foundation
12.	No. of staircase	3 (one dog legged, one steel staircase and one open well)
13.	No. of lifts	3
14.	Method of design	SAP 2000 V16
15.	Design concept	limit state design
16.	Concrete grade	M25
17.	Reinforcement	Fe415
18.	Dead load	as per IS 1893(part 1): 2002
19.	Live load	as per IS 1893(part 2): 2002
20.	Seismic load	as per IS 1893(part 1): 2002
21.	Soil type	soft soil
22.	Topography	hilly terrain

II. LITERATURE REVIEW

- Ashraf M. EI-Shahhat et.al (3) investigated the safety of multistory buildings during construction. The safety of the structure, in these early days of its life is greatly influenced by a large no of factors including the loads, the geometry, and the material properties of the building and the method of construction. The probability of failure of the building during its relatively short period of construction is greater than that of its service life.

- Arindam Sahu1, Rohit Bose2, Indrasish Mukherjee3 and Sahil Hossain Mondal have done sap analysis on multi stories buildings.
- V.Varalaxmi: The design & analysis of multistored G+4 building at Kukatpally, Hyderabad, India. The study includes design & analysis of columns, beams, footings & slabs by using well known civil engineering software named as SAP2000.
- Journal published on www.irjet.net on "TO STUDY THE EARTHQUAKE RESISTANT DESIGN OF STRUCTURE" Ishita Arora, Er. Rajinder Singh
- www.ijbssnet.com : "Construction of Earthquake Resistant Buildings and Infrastructure Implementing Seismic Design and Building Code in Northern Pakistan 2005 Earthquake Affected Area." They discussed the building construction found and the reasons and causes for large scale destruction to the buildings and infrastructure. The buildings were built without implementing code. Then they discussed the building code of Pakistan (including seismic provisions), particularly for the earthquake affected area, and its implementation. They also discussed the building code and seismic design for construction in Japan and compared it with the practices in Pakistan. They described the seismic design and how to use seismic design in different kind of building structures to make the building structures more resistant to earthquakes. In this paper, they suggested some solutions for the construction of building structures in Pakistan to make the building structures more resistant to earthquake and to lessen the damage.
- A Report on the Workshop on Earthquake Resistant Construction in Civil Engineering Curriculum Newsletter of the Indian Society of Earthquake Technology, January 1998. C.V.R. Murty1, Ravi Sinha2, and Sudhir K. Jain3
- Earthquake Resistant Designs: Nimita A. Tijore1 Rushabh A. Shah2 1M. E., Construction management 2Asst. professor, Civil Engineering Department S.N.P.I.T. & R. C, Umrakh, Bardoli, India They talked about serious problem caused by earthquake and preventive measures by constructing earthquake resistance building. Involving different technique like base isolation , concept of frame structures and so on.
- International Journal of Engineering Trends and Technology (IJETT) – Volume 33 Number 9- March 2016 ISSN: 2231-5381: Study on Earthquake Resistant Building (Base Isolation) Prashika Tamang1 Bijay Kumar Gupta2, Bidisha Rai3, Karsang Chukey Bhutia4, Chungku Sherpa5 They explained the effects of earthquake and its preventive measures. They explained the adoption of base isolation using lead rubber bearing at foundation is done in project for the protection of buildings and lives from the fatal earthquake vibration. It also preserves the economic and social state of a country.
- Future trends in earthquake-resistant design of structures: Durgesh C. Rai Department of Earthquake Engineering, University of Roorkee, Roorkee 247 667, India. In this project he has explained about disaster done earthquake and preventive way like, construction with energy dissipation system, active control system, base isolation and etc.

III. METHODOLOGY

In order to achieve the objective of the project as mentioned above, the following procedures are adopted in Analysis and Design of Building for Earthquake Resistance.

3.1 Preliminary Design

The approximate sizes of the structural elements were determined through preliminary design so that after analysis the pre-assumed dimensions might not deviated considerably, thus making the final design for both safe and economical purpose. Approximate size of various elements has been determined as follows.

3.1.1 Slab

For slab, preliminary design is done according to deflection criteria as specified in IS: 456-2000 Clause 23.2.1.

$$\text{Span}/d_{eff} = \alpha\beta\delta\gamma\lambda$$

Where,

α = basic value of span to effective depth ratios for spans up to 10 m
=7 for cantilever
= 20 for simply supported
= 26 for continuous
β = a factor which accounts for correction in the values of α for spans greater than 10 m.
= 10 / span, where span is in meter.
δ = a factor which depends on the area of compression reinforcement.
γ = a modification factor which depends on the stress at service and amount of steel for tension reinforcement
λ = a factor for fanged beams which depends on the ratio of web width to the flange width.

3.1.2 Beam

The preliminary design of beam has been performed using thumb rule of 1" to 12"
i.e.

$$d_{eff} = \text{span}/12 \text{ to span}/15$$

Basic is adopted to consider the preliminary design of the beam section with ratio:

b/D= 1/2 or 2/3.

3.1.3 Column

Preliminary design of column is done considering column of grid as shown below. For the load acting in the column, live load is decreased according to IS 975: 1978(part 2) and then design is carried out using SP-16.

3.2 Load Calculations

Load calculation for superimposed load and dead load are done by the IS: 875-1987(part 1 and part 2) as reference. The exact value of unit weights of the material used in the building has been extracted from the code and was used in calculations. Thickness of materials was taken as per design requirement.

According to IS: 1893- 2002, Nepal lies on the fifth zone. Hence, the effect of earthquake is predominant than the wind load. So, the frame is analyzed for the earthquake as lateral load. For the determination of lateral load, it is assumed that the mass to be lumped at the floor level and lumped mass having the value corresponding to the mass of floor, part of the support system above and below the floor and reduced live load, base shear (V_B) is then calculated using the coefficient design method of IS: 1893-2002.

Identification of loads:

- Dead loads are calculated as per IS: 1893 (Part 1) – 2002
- Imposed loads according to IS: 1893 (Part 2) - 2002
- Seismic load according to IS: 1893 (Part 1): 2002 considering Kathmandu located at Zone V.

There are two methods to determine the earthquake force in a building:

1. Seismic coefficient method (static method)
2. Response spectrum method or modal analysis method of spectral acceleration method (dynamic method)

3.2.1 Response Spectrum Method:

Response spectrum of any building gives us a plot of peak or steady state response(Displacement, Velocity or Acceleration) of a series of oscillators of a varying natural frequency, that are forced into motion by the same base vibration or shock. The resulting plot can then be used to pick off the response of any linear system, given its natural frequency of oscillation. Response spectrum analysis requires that isolator units be modeled using amplitude- dependent values of effective stiffness.

3.2.2 Methods of analysis:

The building is modeled as a space frame. **SAP 2000 V16** is adopted as the basic tool for the execution of analysis. SAP 2000 V16 program is based on Finite Element Method. Due to possible actions in the building, the stresses, displacements and fundamental time periods are obtained using SAP 2000 V16 which are used for the design of the members.

IS 1893-2002 (part 1) is followed for the seismic analysis of the building. The fundamental time period of the structure is calculated as specified in code.

The following combinations are considered for the analysis.

Load Combination:

1)**1.5(DL+LL)** along x and y direction
2)**1.2(DL +LL ± EQ)** along x and y direction
3)**1.5(DL ±EQ)** along x and y direction
4)**0.9DL± 1.5EQ** along x and y direction

3.3 Design:

The following materials are adopted for the design of the elements:

- **Concrete Grade: M25**
  - M25 for Beam, Column, Mat foundation, Stair case and Slab.
- **Reinforcement Steel (Deformed Bars) – Fe415**
  - Fe 415 for longitudinal as well as for lateral ties

Limit state method is used for the design of RC elements.

3.4 Detailing:

The structure is designed with due consideration to provide ductile behavior and comply with the requirements given in **IS: 456-2000: IS: 1893-2002, IS 13920:1993.** Detailing was done by determining number, size, layout and location of reinforcement giving the element dimension and areas of steel required.

IV. RESULTS AND DISCUSSION

4.1 Preliminary Design

For analysis of the building, it requires the rough idea on the member sizes used in the building as beam, column and slab. According to which the contributed dead load of the member to the structure could be estimated.
The size of the members is dependent on the **Limit state of Serviceability on Deflection and Cracking**. For this, the **IS code 456-2000** is referred to make sure.

4.1.1 MAIN BEAM

In Y-direction,
Maximum longer span = 8000 mm
Maximum longer span/effective depth = 12-15
8000/13= d
Therefore d = 615.38 mm, adopt=620 mm
Take effective cover = 30 mm
Therefore, overall depth (D) = 650 mm
Take b = D/2 or 2*D/3
= 650/2 = 325 mm OR 2*650/3 = 433 mm
= 325 mm to 433 mm
Take width of the beam = 350 mm
Therefore, **Main beam = 350 mm x 650 mm**

4.1.2 SLAB

% of steel (0.1% to 0.4%) = 0.3% (assumed)
Using the deflection criteria for the continuous slab,

$$Span/d \leq \alpha\beta\delta\gamma\lambda$$

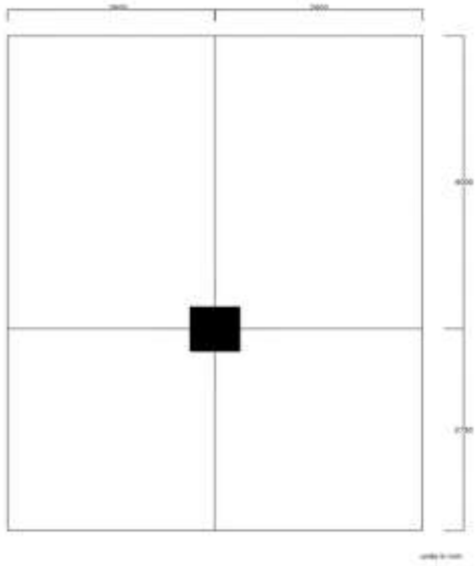
Where,
α= 26 δ= 1
β= 1 λ= 1
Now for the modification factor γ for tension reinforcement,
Let, f_y= 415 KN/m^2
f_s= 0.58 *f_y * A_{st} of X-section of steel required/Area of X-section of steel provided
= 0.58 * 415 * 1
= 240.7 N/mm^2
So, γ = 1.64 from fig. 4

Effective depth of slab (d) = $l_x/\alpha\beta\delta\gamma\lambda$
= 5000/ (26 * 1 * 1.64 * 1)
= 117.26 mm
Provide effective cover = 20 mm
Therefore, Total depth of slab (D) = 137 mm; **Adopt 125 mm as slab thickness.**
Since thickness of slab is more than 125 mm, we will adopt 125 mm as slab thickness. To compensate this, secondary beam is designed since the longest span of beam is 8000 mm.

4.1.3 SECONDARY BEAM

In Y direction, longest span of beam= 8000 mm
So, Span/ effective depth= 17
d= 8000/17
= 470.58 mm
Taking effective cover= 20 mm
Overall depth, D= 470.58+ 20 = 490.58 mm
Adopt overall depth, D= 500 mm
For width of beam (b),
b= 0.5D to 0.67D
= 250 mm to 333.33 mm
Adopt width of beam, b= 300 mm
Therefore, section of secondary beam along X- axis = 300 mm * 500 mm at spacing 8000/3= 2667 mm c/c.

4.1.4 COLUMN



Critical column

Area = (2.75 m + 4.0 m) * (2.5 m + 2.5 m)
=33.75 m^2

Dead Load Calculations:

1. Beam:

Load = (6.150 + 4.400) * 0.350* 0.650* 25
= 120.006 KN

2. Slab:

Load = 33.75 * 0.125* 25
= 105.468 KN

3. Assume (600 * 600) mm column

Self-weight of column = 0.6 * 0.6 *3 * 25
= 27 KN

Total dead load = 120.006+ 105.468+ 27
= 252.457 KN

Floor finish (thickness = 20 mm)

Dead load = 33.75 * 0.02* 20 = 13.5 KN

Assume thickness of plaster = 12 mm thick

Dead load = 33.75* 0.012 *20 = 8.1 KN

Total dead load for floor finish = 13.5+ 8.1 KN
= 21.6 KN

Total dead load= 252.457+ 21.6 = 274.057 KN

For 12 stories,

Total dead load = 12* 274.057
= **3288.684 KN**

Live load calculations

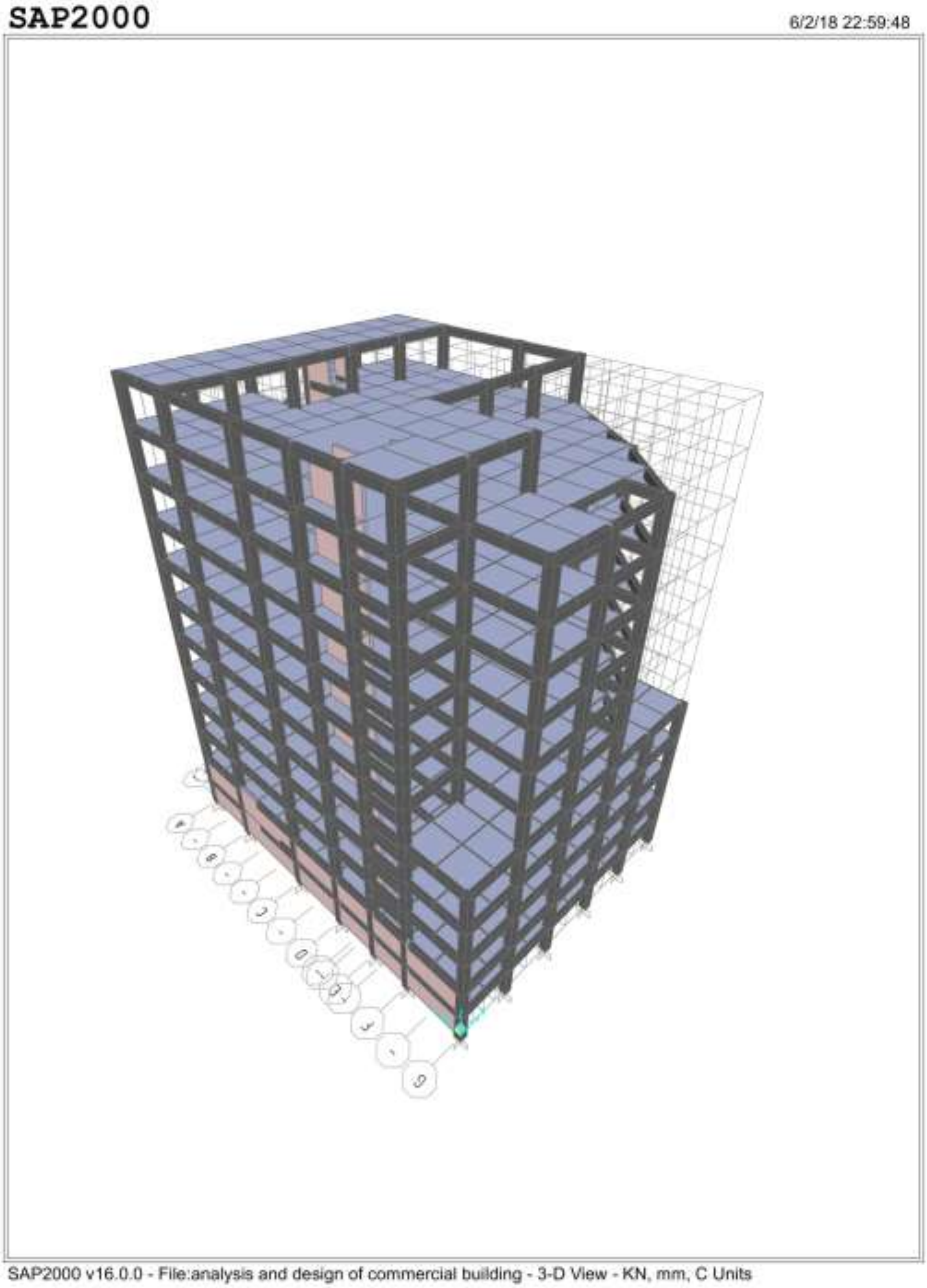
S.N.	Floor	Live load (KN)
1	Lower basement	0
2	Upper basement	5* 33.75* 1= 168.75
3	Ground	4* 33.75* 0.9= 121.5
4	First floor	4* 33.75* 0.8= 108
5	Second floor	3* 33.75* 0.7= 70.875
6	Third floor	4* 33.75* 0.6= 81
7	Fourth floor	4* 33.75* 0.6= 81
8	Fifth floor	4* 33.75* 0.6= 81
9	Sixth floor	4* 33.75* 0.6= 81
10	Seventh floor	4* 33.75* 0.6= 81
11	Eighth floor	4* 33.75* 0.6= 81
12	Ninth floor	4* 33.75* 0.6= 81
13	Roof	1.5* 33.75* 0.5= 25.31

Total LL= 1061.435 KN

Total load = dead load + live load
= 3288.684 +1061.435 = 3051.288 KN

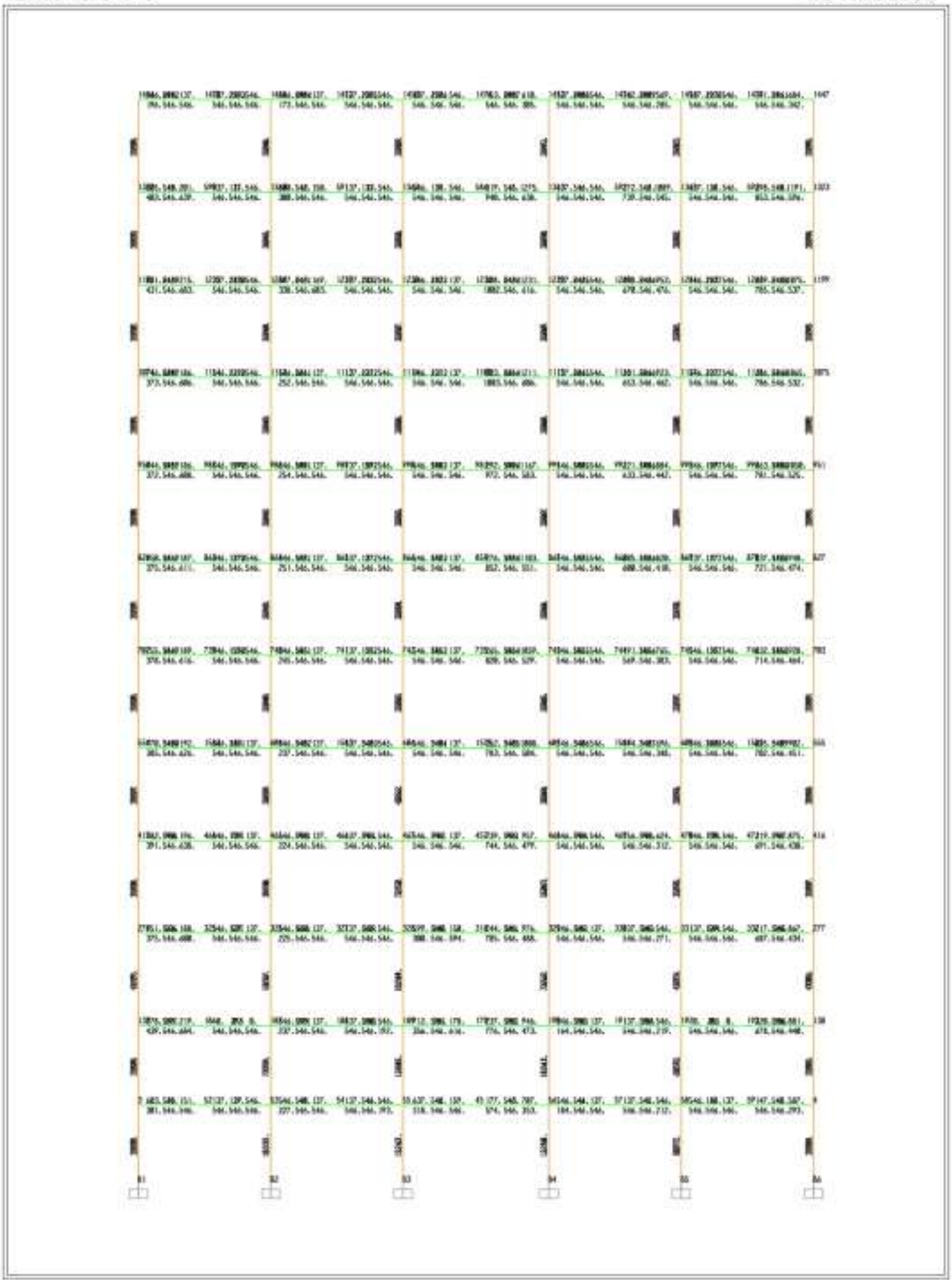
Factored load, P_u =1.5* 3051.288
=6525.178 KN
For earthquake load, 20% addition
Total P_u= 1.2* 6525.178
= 7830.214 KN
Assume 3.5% steel,
From
$P_u / A_g = 0.4f_{ck} + (p/100)\,(0.67f_y - 0.4f_{ck})$
A_g = [7830.214* 10^3] / [0.4* 25+ 0.035*(0.67* 415- 0.4*25)]
= 426366.1094 mm^2
Assuming square column,
$L = B = A_g^{1/2}$
= 648.22 mm; adopt 650 mm.
Size of column =650 mm × 650 mm

4.2 STRUCTURAL DESIGN

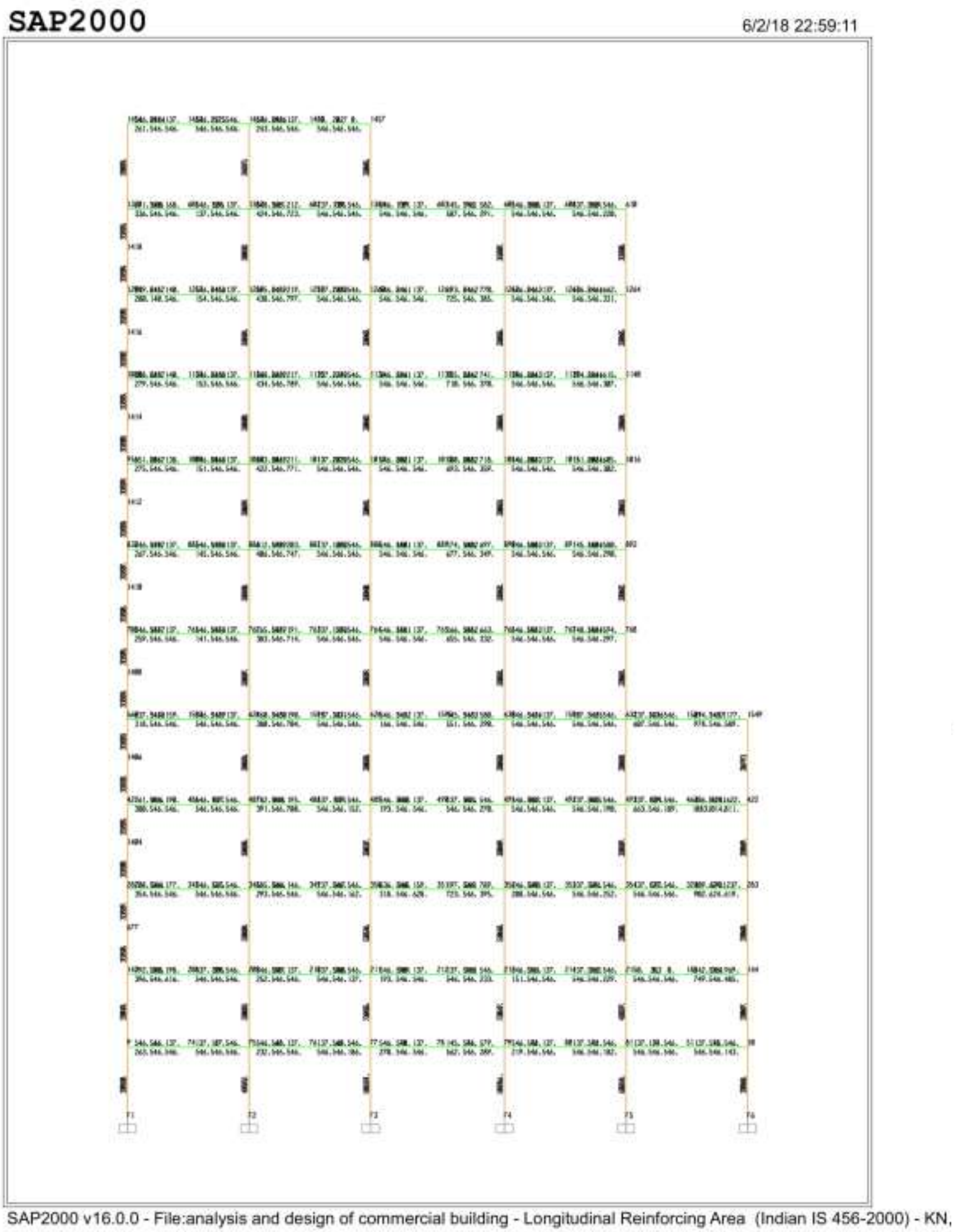


4.2.1 DESIGN OF BEAM

SAP2000 6/2/18 22:58:29

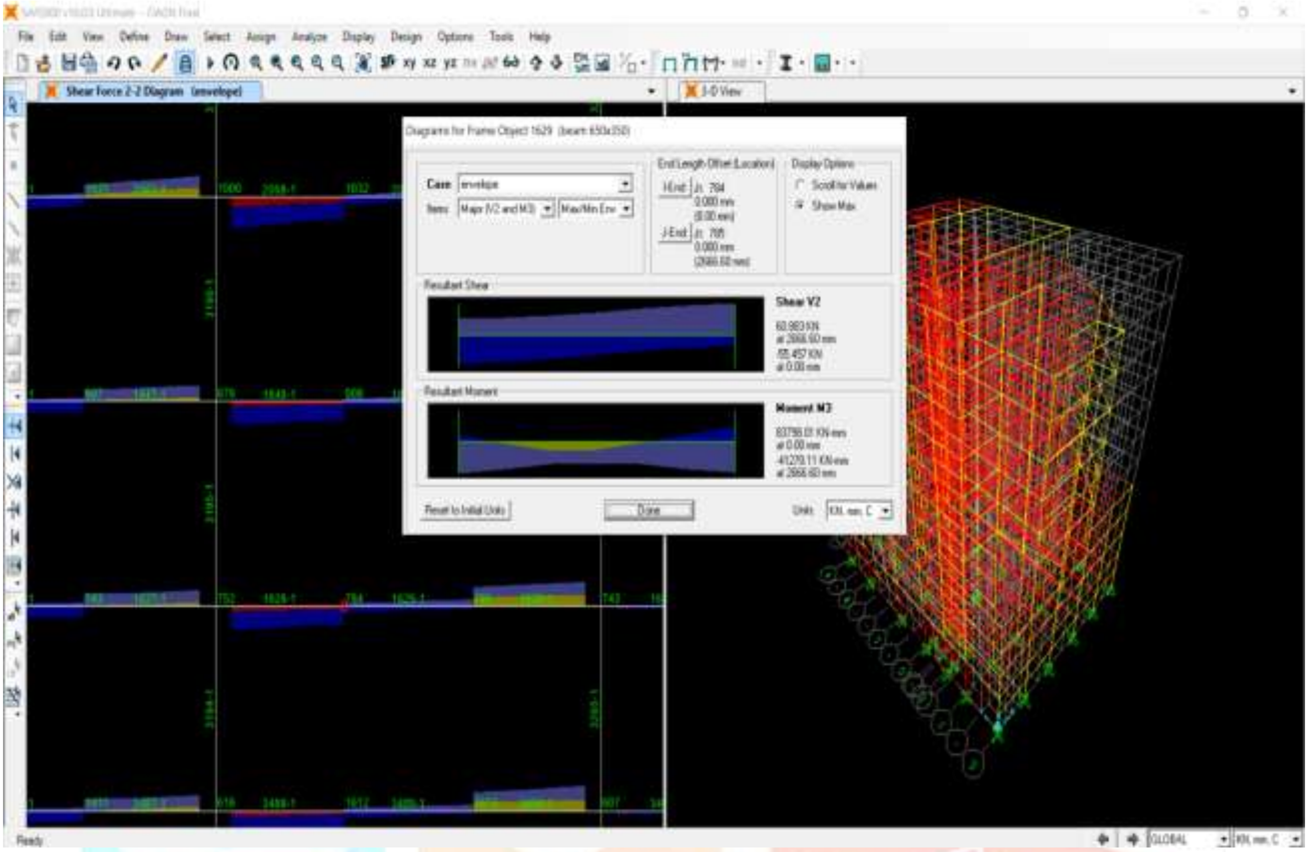


SAP2000 v16.0.0 - File:analysis and design of commercial building - Longitudinal Reinforcing Area (Indian IS 456-2000) - KN,



Calculations:
All the ends of members are designed as rectangular section.
Size of beam= 650mm*350mm
From SP 16, Limiting moment (Mu_{lim})= $0.36*f_{ck}*b*Xu_{max}(d-0.42*Xu_{max})$
If $Mu < Mu_{lim}$, section is designed as singly reinforcement.
If $Mu \geq Mu_{lim}$, section is designed as doubly reinforcement.

BEAM 1629:



HOGGING MOMENT:

From diagram, Mu = 83798.01 KN-mm = 83.798 KN-m

Now,

$Mu_{lim} = 0.36*f_{ck}*b*Xu_{max}(d-0.42*Xu_{max})$

Also,

$Xu_{max} = 0.48*d$

Where d = effective depth =D - cc – Ø/2

$Xu_{max} = 0.48*565 = 271.2$mm

$Mu_{lim} = 0.36*25*350*271.2(565-0.42*271.2)$

= 385362290.9 N-mm

= 385.362 KN-m

Here Mu < Mu_{lim} so section design is singly beam section.

Again, d′/d = 35/565 = 0.062, taking higher value of 0.1, from table 7.1 IS: 456- 1978

$f_{sc} = 353$ N/mm^2

Now,

Area of tension steel (A_{st}):

$A_{st} = Mu_{lim}/0.87f_y(d-0.42Xu_{max})$

$A_{st} = 385.362*10^6/0.87*415(565-0.42*271.2)$

= 2366.1 mm^2

Let us provide 20mm dia. bars of 8 bars.

From IS: 456- 1978 Table no. 95

A_{st} provided = 2512 mm2

CHECK FOR DEFLECTION:

l/d ≤ αβγ∂

Where α = 20 for simply supported beam

For modification factor γ:

$P_t = 100*A_{st}/ (b*d)$

= 100* 2512/ (350*565)

= 1.27%

$f_s = 0.58f_y(A_{st, req}/ A_{st, provided})$

= 0.58*415*(2366.1/2512) = 226.72 N/mm^2

Using P_t = 1.27% and f_s = 226.72 N/mm^2, from IS 456 – 1978

γ = 1.02

Then,
$l/d = 8650/565 \leq 20*1.02 = 20.4$
$15.31 < 20.4$ (ok)

SHEAR CHECK:
Shear force, $V_u = 60.883$ KN
Now, shear stress $(\tau_u) = V_u/bd$
$= 60.883*10^3/(350*565)$
$= 0.308$ N/mm^2
From IS 456- 2000 Table 20:
$\tau_{cmax} = 3.1$ N/mm$^2 > \tau_u = 0.308$ N/mm^2 (ok)
A_{st} at supports = 2512 mm^2
We have, percentage of steel $P_t = 1.27\%$
For M25 concrete and $p_t = 1.27\%$, interpolate in table for τ_c in IS 456- 2000 Table 19

P	τ_c (N/mm^2)
1.25	0.70
1.27	x
1.50	0.74

After interpolation, $\tau_c = x = 0.7032$ N/mm^2
Thus, $\tau_u = 0.308$ N/mm$^2 < \tau_c = 0.7032$ N/mm^2
Hence, minimum shear reinforcement is to be provided.
Therefore, provide 8mm dia. 2- legged vertical stirrups at 150mm spacing.

CHECK FOR DEVELOPMENT LENGTH (L_d):
$(M_1)/V_u + L_0 = L_d$
Also, $L_d = 0.87f_y\emptyset/(4\tau_{bd})$
$= 0.87*415*\emptyset/(4*2.24)$
$= 40.296\emptyset$
Here, $L_o = 16\emptyset$ (provide U- bent at the end of bars at centre of supports.)
$M_1 = 0.87f_yA_{st}(d-(f_yA_{st}/f_{ck}b))$
$= 0.87*415*2512*(565-(415*2512/25*350))*10^{-6}$
$= 403.468$ KN-m
Then
$(403.468*10^6/60.883*10^3) + 16\emptyset \geq 40.296\emptyset$
$6626.94 + 16\emptyset \geq 40.296\emptyset$ (ok)
$L_d = 0.87*415*25/(4*2.24)$
$= 1007.394 \text{ mm} \approx 1010\text{mm}$

FOR SAGGING MOMENT:
From diagram, Mu = 41270.11 KN-mm = 41.27 KN-m
Now,
$Mu_{lim} = 0.36*f_{ck}*b*Xu_{max}(d-0.42*Xu_{max})$
Also, $Xu_{max} = 0.48*d$
Where d = effective depth =D - cc – $\emptyset/2$
$Xu_{max} = 0.48*565 = 271.2$mm
$Mu_{lim} = 0.36*25*350*271.2(565-0.42*271.2)$
$= 385362290.9$ N-mm
$= 385.362$ KN-m
Here Mu < Mu_{lim} so section design is singly beam section.
Again, $d'/d = 35/565 = 0.062$, taking higher value of 0.1, from table 7.1 IS: 456- 1978
$f_{sc} = 353$ N/mm^2
Now,
Area of tension steel (A_{st}):
$A_{st} = Mu_{lim}/0.87f_y(d-0.42Xu_{max})$
$A_{st} = 385.362*10^6/0.87*415(565-0.42*271.2)$
$= 2366.1$ mm^2
Let us provide 20mm dia. bars of 8 bars.
From IS: 456- 1978 Table no. 95

A_{st} provided = 2512 mm2

CHECK FOR DEFLECTION:

$l/d \leq \alpha\beta\gamma\partial$

Where α = 20 for simply supported beam

For modification factor γ:

$P_t = 100*A_{st}/ (b*d)$

$= 100* 2512/ (350*565)$

$= 1.27\%$

$f_s = 0.58f_y(A_{st, req}/ A_{st, provided})$

$= 0.58*415*(2366.1/2512)$

$= 226.72$ N/mm^2

Using P_t = 1.27% and f_s = 226.72 N/mm^2, from IS 456 – 1978

γ = 1.02

Then,

$l/d = 8650/565 \leq 20*1.02 = 20.4$

$15.31 < 20.4$ (ok)

SHEAR CHECK:

Shear force, V_u = 55.457 KN

Now, shear stress (τ_u) = V_u/bd

$= 55.475*10^3/(350*565)$

$= 0.281$ N/mm^2

From IS 456- 2000 Table 20:

τ_{cmax} = 3.1 N/mm^2 > τ_u = 0.281 N/mm^2 (ok)

A_{st} at supports = 2512 mm^2

We have, percentage of steel P_t = 1.27%

For M25 concrete and p_t = 1.27%, interpolate in table for τ_c in IS 456- 2000 Table 19

P	τ_c (N/mm^2)
1.25	0.70
1.27	x
1.50	0.74

After interpolation, τ_c = x = 0.7032 N/mm^2

Thus, τ_u = 0.281 N/mm^2 < τ_c = 0.7032 N/mm^2

Hence, minimum shear reinforcement is to be provided.

Therefore, provide 8mm dia. 2- legged vertical stirrups at 150mm spacing.

CHECK FOR DEVELOPMENT LENGTH (L_d):

$(M_1)/V_u + L_0 = L_d$

Also, $L_d = 0.87f_y Ø/(4\tau_{bd})$

$= 0.87*415*Ø/(4*2.24)$

$= 40.296Ø$

Here, L_o = 16Ø (provide U- bent at the end of bars at centre of supports.)

$M_1 = 0.87f_yA_{st} (d-(f_yA_{st}/f_{ck}b))$

$= 0.87*415*2512*(565-(415*2512/25*350))*10^{-6}$

$= 403.468$ KN-m

Then

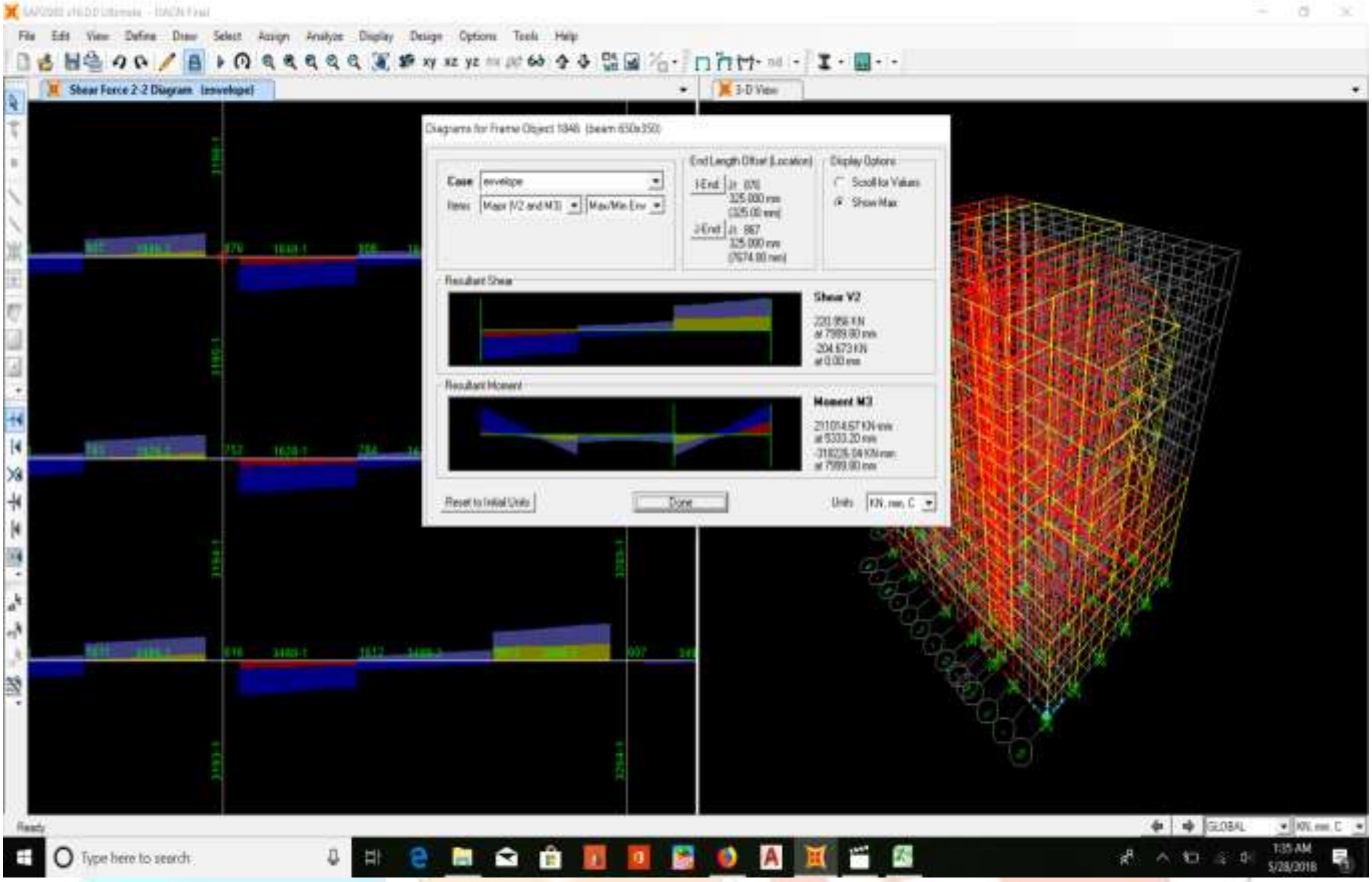
$(403.468*10^6/55.475*10^3) + 16Ø \geq 40.296Ø$

$7272.97 + 16Ø \geq 40.296Ø$ (ok)

$L_d = 0.87*415*25/ (4*2.24)$

$= 1007.394$ mm $\approx$ 1010mm

BEAM 1848: (length = 5 m)



HOGGING MOMENT:

From diagram, Mu = 211014.67 KN-mm = 211.015 KN-m

Now,

$Mu_{lim} = 0.36*f_{ck}*b*Xu_{max}(d-0.42*Xu_{max})$

Also,

$Xu_{max} = 0.48*d$

Where d = effective depth =D - cc – Ø/2

$Xu_{max} = 0.48*565 = 271.2$mm

$Mu_{lim} = 0.36*25*350*271.2(565-0.42*271.2)$

= 385362290.9 N-mm = 385.362 KN-m

Here Mu < Mu_{lim} so section design is singly beam section.

Again, d′/d = 35/565 = 0.062, taking higher value of 0.1, from table 7.1 IS: 456- 1978

$f_{sc} = 353$ N/mm^2

Now,

Area of tension steel (A_{st}):

$A_{st} = Mu_{lim}/0.87f_y(d-0.42Xu_{max})$

$A_{st} = 385.362*10^6/0.87*415(565-0.42*271.2)$

= 2366.1 mm^2

Let us provide 20mm dia. bars of 8 bars.

From IS: 456- 1978 Table no. 95

A_{st} provided = 2512 mm2

CHECK FOR DEFLECTION:

$l/d \leq \alpha\beta\gamma\partial$

Where α = 20 for simply supported beam

For modification factor γ:

$P_t = 100*A_{st}/ (b*d)$

= 100* 2512/ (350*565)

= 1.27%

$f_s = 0.58f_y(A_{st, req}/ A_{st, provided})$

= 0.58*415*(2366.1/2512)

$= 226.72\ N/mm^2$
Using $P_t = 1.27\%$ and $f_s = 226.72\ N/mm^2$, from IS 456 – 1978
$\gamma = 1.02$
Then,
$l/d = 5650/565 \leq 20*1.02 = 20.4$
$10 < 20.4$ (ok)

SHEAR CHECK:

Shear force, $V_u = 204.673$ KN
Now, shear stress $(\tau_u) = V_u/bd$
$= 204.673*10^3/(350*565)$
$= 1.035\ N/mm^2$
From IS 456- 2000 Table 20:
$\tau_{cmax} = 3.1\ N/mm^2 > \tau_u = 1.035\ N/mm^2$ (ok)
A_{st} at supports = 2512 mm^2
We have, percentage of steel $P_t = 1.27\%$
For M25 concrete and $p_t = 1.27\%$, interpolate in table for τ_c in IS 456- 2000 Table 19

P	τ_c (N/mm^2)
1.25	0.70
1.27	x
1.50	0.74

After interpolation, $\tau_c = x = 0.7032\ N/mm^2$
Thus, $\tau_u = 1.035\ N/mm^2 > \tau_c = 0.7032\ N/mm^2$
Hence, shear reinforcement is to be provided.
Using 8mm dia. 2- legged vertical stirrups.
$A_{sv} = 2*\pi*8^2/4$
$= 100.5\ mm^2$
Shear resistance of the reinforcement:
$V_{us} = V_u - \tau_c *b*d$
$= 204.673*10^3 - 0.7032*350*565$
$= 65615.2\ N = 65.615KN$
Now, Spacing of vertical stirrups:
$S_v = 0.87*f_y*A_{sv}*d/V_{us}$
$= 0.87*415*100.5*565/65615.2 = 312.448$ mm
Spacing as per the nominal reinforcement:
$S_v = 0.87f_y*A_{sv}/(0.4*b)$
$= 0.87*415*100.5/(0.4*350)$
$= 259.182$ mm
Maximum spacing should be less than following:

i. 0.75*d = 0.75*565 = 423.75 mm
ii. 300 mm

So provide 2-legged 8 mm Ø vertical stirrups at 200 mm spacing.

CHECK FOR DEVELOPMENT LENGTH (L_d):

$(M_1)/V_u + L_0 = L_d$
Also, $L_d = 0.87f_y\text{Ø}/(4\tau_{bd})$
$= 0.87*415*\text{Ø}/(4*2.24)$
$= 40.296\text{Ø}$
Here, $L_o = 16\text{Ø}$ (provide U- bent at the end of bars at centre of supports.)
$M_1 = 0.87f_yA_{st}(d-(f_yA_{st}/f_{ck}b))$
$= 0.87*415*2512*(565-(415*2512/25*350))*10^{-6}$
$= 403.468$ KN-m
Then
$(403.468*10^6/204.673*10^3) + 16\text{Ø} \geq 40.296\text{Ø}$
$1971.281 + 16\text{Ø} \geq 40.296\text{Ø}$ (ok)
Thus,
$L_d = 0.87*415*25/(4*2.24)$
$= 1007.394\ mm \approx 1010mm$

FOR SAGGING MOMENT:
From diagram, Mu = 318226.04 KN-mm = 318.226 KN-m
Now,
$Mu_{lim} = 0.36*f_{ck}*b*Xu_{max}(d-0.42*Xu_{max})$
Also, $Xu_{max} = 0.48*d$
Where d = effective depth =D - cc – Ø/2
$Xu_{max} = 0.48*565 = 271.2$mm
$Mu_{lim} = 0.36*25*350*271.2(565-0.42*271.2)$
= 385362290.9 N-mm
= 385.362 KN-m
Here Mu < Mu_{lim} so section design is singly beam section.
Again, d′/d = 35/565 = 0.062, taking higher value of 0.1, from table 7.1 IS: 456- 1978
$f_{sc} = 353$ N/mm^2
Now,
Area of tension steel (A_{st}):
$A_{st} = Mu_{lim}/0.87f_y(d-0.42Xu_{max})$
$A_{st} = 385.362*10^6/0.87*415\ (565-0.42*271.2)$
= 2366.1 mm^2
Let us provide 20mm dia. bars of 8 bars.
From IS: 456- 1978 Table no. 95
A_{st} provided = 2512 mm2

CHECK FOR DEFLECTION:
$l/d \leq \alpha\beta\gamma\partial$
Where α = 20 for simply supported beam
For modification factor γ:
$P_t = 100*A_{st}/\ (b*d)$
= 100* 2512/ (350*565)
= 1.27%
$f_s = 0.58f_y(A_{st,\ req}/\ A_{st,\ provided})$
= 0.58*415*(2366.1/2512)
= 226.72 N/mm^2
Using P_t = 1.27% and f_s = 226.72 N/mm^2, from IS 456 – 1978
γ = 1.02
Then,
$l/d = 5650/565 \leq 20*1.02 = 20.4$
10.0 < 20.4 (ok)

SHEAR CHECK:
Shear force, V_u = 220.956 KN
Now, shear stress (τ_u) = V_u/bd
= 220.956*10^3/(350*565)
= 1.117 N/mm^2
From IS 456- 2000 Table 20:
τ_{cmax} = 3.1 N/mm^2 > τ_u = 1.117 N/mm^2 (ok)
A_{st} at supports = 2512 mm^2
We have, percentage of steel P_t = 1.27%
For M25 concrete and p_t = 1.27%, interpolate in table for τ_c in IS 456- 2000 Table 19

P	τ_c (N/mm^2)
1.25	0.70
1.27	x
1.50	0.74

After interpolation, τ_c = x = 0.7032 N/mm^2
Thus, τ_u = 1.117 N/mm^2 > τ_c = 0.7032 N/mm^2
Hence, shear reinforcement is to be provided.
Using 8mm dia. 2- legged vertical stirrups.
$A_{sv} = 2*\pi*8^2/4$
= 100.5 mm^2

Shear resistance of the reinforcement:
$V_{us} = V_u - \tau_c *b*d$
$= 220.956*10^3 - 0.7032*350*565$
$= 81898.2$ N = 81.898 KN
Now, Spacing of vertical stirrups:
$S_v = 0.87*f_y*A_{sv}*d/V_{us}$
$= 0.87*415*100.5*565/81898.2$
$= 250.327$ mm
Spacing as per the nominal reinforcement:
$S_v = 0.87f_y*A_{sv}/(0.4*b)$
$= 0.87*415*100.5/(0.4*350)$
$= 259.182$ mm
Maximum spacing should be less than following:
i. $0.75*d = 0.75*565 = 423.75$ mm
ii. 300 mm

So provide 2-legged 8 mm Ø vertical stirrups at 200 mm spacing.

CHECK FOR DEVELOPMENT LENGTH (L_d):
$(M_1)/V_u + L_0 = L_d$
Also, $L_d = 0.87f_y\text{Ø}/(4\tau_{bd})$
$= 0.87*415*\text{Ø}/(4*2.24)$
$= 40.296\text{Ø}$
Here, $l_o = 16\text{Ø}$ (provide U- bent at the end of bars at centre of supports.)
$M_1 = 0.87f_yA_{st}(d-(f_yA_{st}/f_{ck}b))$
$= 0.87*415*2512*(565-(415*2512/25*350))*10^{-6}$
$= 403.468$ KN-m
Then,
$(403.468*10^6/220.956*10^3) + 16\text{Ø} \geq 40.296\text{Ø}$
$1826.01 + 16\text{Ø} \geq 40.296\text{Ø}$ (ok)
Thus, $L_d = 0.87*415*25/(4*2.24)$
$= 1007.394$ mm ≈ 1010mm

4.2.2 DESIGN OF SLAB
Slab is a flexural element and there are mainly two types of slab based on the ratio of longer to shorter span of room. They are as follow:
i. One way slab: It is a slab with the ratio of longer to shorter span greater than 2 and the coefficient for it can be used from Table 26. b (IS 456:2000).
ii. Two way slab: It is the slab with the ratio of longer to shorter span less than or equal to 2 and the coefficient for it can be used from table 26. a (IS456:2000).
There are ten types of two way continuous slab depending upon the length and the discontinuous edge. The conditions to be satisfied for use of these conditions are
a) The loading of the adjacent span should be the same.
b) The span in each direction should be approximately equal.

The span moment per unit width (which are considered as positive in sign) and the negative moments at continuous edge for these slabs are calculated from the equation:
$M_x=\alpha_x w l_x^2$ from span l_x
$M_y=\alpha_y w l_x^2$ from span l_y
Spacing of bars on slab:
A. Maximum spacing in main bar :
1. 3 times the effective depth
2. 300 mm, ; whichever is less

B. Maximum spacing in distribution bars
1. 5 times the effective depth
2. 450 mm, ; whichever is less

Reinforcement requirement in slab:
i. Maximum reinforcement:
$A_{st\ max}$ =4% of area of slab
ii. Minimum reinforcement:
$A_{st\ min}$ =0.12% of area of slab

DESIGN OF SLAB (ONE LONG EDGE DISCONTINUOUS)
l_y (long span) = 8 m

l_x (short span) = 5 m
$l_y/l_{x=}$ = 1.58<2
Hence, it is a two way slab. Slab is one edge continuous.
For bending moment coefficients,
From table 26 IS 456:2000,

$$M_x = \alpha_x W_u l_x^2$$

$$M_y = \alpha_y W_u l_x^2$$

α_{x-}= 0.0702	α_{y-}=0.037
α_{x+}= 0.0535	α_{y+}= 0.028

M_{x-}	= 0.043*11.7*3.49^2	= 22.208 KNm
M_{x+}	= 0.032*11.7*3.49^2	= 16.925 KNm
M_{y-}	= 0.037*11.7*3.49^2	= 29.548 KNm
M_{y+}	= 0.028*11.7*3.49^2	= 22.361 KNm

Maximum bending moment, M_u= 29.548 KNm

DESIGN LOAD
Dead load:
Self-weight = 25*0.125*1 = 3.125 KN/m
Floor finish = 1 KN/m^2
Total dead load= 4.125 KN/m
Live load = 4 KN/m
Total design load= 8.125 KN/m
Factored load (W_u) = 1.5*8.125KN/m =12.187KN/m

BENDING MOMENT:
Maximum bending moment, M_u= 29.548 KNm
D = 125mm
d = 95mm
$L_{ex} = L_{cx}$ + d/2 + d/2 = 5.095 m
$L_{ey} = L_{cy}$ + d/2 + d/2 = 8.095 m

CHECK FOR EFFECTIVE DEPTH FOR MAXIMUM BENDING MOMENT:
For f_y= 415 MPa & f_{ck}= 25 MPa
M_{max}= 0.138*f_{ck}*b*d^2
29.548*10^6= 0.138*25*1000*d^2
d = 92.54 mm< 95 mm
Then, adopt d = 95 mm, D = 125 mm and b = 1000mm.

STEEL CALCULATIONS:
We have,
B.M. = 0.87*f_y*A_{st}*(d - $\frac{fy*Ast}{fck*b}$)

1. **For shorter span,**

$(B.M)_{x,max}$ = 22.208 KNm
22.208*10^6 = 0.87*415*A_{stx}*(120 – $\frac{415*Astx}{25*1000}$)
A_{stx} = 739.59 mm^2
Provide 10 mm bars,
Area of one bar (A_o)= 78.54 mm^2
Spacing (S_x)= 1000*A_o/A_{stx}
= 106.19 mm > 100 mm (OK)
Adopt 106 mm as spacing in shorter span.
Also S_x < 3*d = 3*95 = 285 mm > 106 mm (OK)
Provide 10 mm diameter bars@ 106 mm c/c.
Thus, (A_{st}) provided = 1000* 78.54/106
= 740.94 mm^2

Use 10 mm ϕ bars @ 106 mm along short span
Minimum reinforcement, $(A_{st})min$ = 0.12 % of b*d
= (0.12/100) *1000 *125
= 150 mm^2 < 714 mm^2 Hence OK

2. Along longer span,

d' = d - ϕ_s = 95-10 = 85 mm
$(BM)_{y,max}$ = 29.584 KNm
29.584*10^6=0.87*415*A_{sty} (85-415*A_{sty}/25*1000)
(A_{sty}) required =1265.39 mm^2
Use 10 mm bars,
S_y = 1000* 78.54/ 1265.39 = 62.06 < 100 mm
So adopt S_y = 150 mm
>100 mm
< 5*d = 5*95 = 475 mm, Hence okay.
Provide ϕ–10 mm @ 150 mm c/c along longer span in middle strip
(A_{st}) provided = 1000* 78.54 / 150
= 523.6 mm^2 > $(A_{st})_{min}$ (OK)

CHECK FOR SHEAR:
We have factored shear force,
V_u= W_u L_x / 2
= 12.187*5 /2
= 30.467 KN.
Nominal shear stress (τ_v) = V_u/b*d
= 30.467/(1000*95) = 0.32 MPa
Also, p= (100* A_{sty}/2)/ b*d
= (100* 523.6/2)/ (1000*95)
= 0.275%
Therefore, shear strength of concrete is:
τ_c = 3.1 MPa (for M25 concrete)
Also, modified τ_c'= k* τ_c
= 1.3* 3.1 (k= 1.3 for d ≤ 150 mm; from B-5.2.1.1 of IS 456: 2000)
= 4.03 MPa
Since, τ_v< τ_c', no shear reinforcement is required.

CHECK FOR DEVELOPMENT LENGTH:
We know,
L_d= [(1.3M_1)/ V_u] + L_o
Development length of bar: (IS: 456-2000, clause 26.2.1)
For 90° bent, L_o= 8ϕ
M_1= 0.87* fy* A_{sty}[d-(f_y*A_{sty})/ (f_{ck}*b)]
=0.87*415*523.6[85-(415*523.6)/(25*1000)]
= 14.425 KN-m
Also, L_d= (0.87* f_y* ϕ)/ (4*τ_{bd})
For f_{ck}=25 MPa, τ_{bd}= 2.24 MPa [τ_{bd}=1.4 MPa but increased by 60% due to deformed bars]
Then,

L_d = 1.3*M_1/V_u+ 8ϕ

(0.87*415* ϕ)/ (4*2.24) = (1.3*14.425* 10^6)/(30.467*10^3)+8* ϕ

ϕ = 19.06 mm > $\phi_{provided}$ (OK)
Hence, it is safe in development length.
Then, L_d= (0.87* 415*10)/ (4*2.24)
= 402.95 mm
Adopt L_d= 405 mm.

CHECK FOR DEFLECTION
Minimum effective depth (d)= $l_x/(\alpha\beta\gamma\delta\lambda)$

Here, α = (20+ 26)/2 = 23 (simply supported plus continuous)
β = 1 (span < 10 m)
for γ,
f_s =0.58* f_y *(area of steel required / area of steel provided)
f_s =0.58*415*(739.59 / 740.94) = 240.26 MPa
p% = $(A_{st})_{provided}$*100)/(b*d)=(740.94*100)/(1000*95)=0.77%
From fig. (4), IS 456:2000, γ=1.1
δ= 1 and λ= 1 for slab.
Then,
d_{req} > span/αβγ$\delta\lambda$ = (5*1000)/(23*1*1.1*1*1)=117.628 mm > 125 mm.
Hence it is not safe in deflection.
So adopt d= 125 mm and provide secondary beam of size 300 mm* 500 mm with singly reinforced bars in 4 no.s -16 mm diameter bars.

4.2.3 DESIGN OF COLUMN
A column may be classified as follows based on types of loading:

a. Axially loaded column
b. A column subjected to axial load and uniaxial bending and
c. A column subjected to axial load and biaxial bending

The design of column section for given axial load and biaxial moments can be made by pre-assigning the section geometry from axially loaded consideration and then checking adequacy for given eccentricity and moments.

The minimum eccentricity specified by the **IS 456-2000(clause 39.2)** is:

$$e_{min}=\frac{L_o}{500}+\frac{D}{300}$$

Where, L_o=unsupported length of column
D = lateral dimension in plane of bending
If e_{min} is less than 0.05*D, then column can be designed as axially loaded column. Else it is to be designed for both moment and axial load.

COLUMN NO. 3248 [LOAD COMBINATION: 1.2(DL+LL)
From sap analysis,
P_u= 6118.73 KN
M_{ux} = 135.85 KNm
M_{uy} = 151.68 KNm
We have,
f_y= 415 N/mm^2
f_{ck} = 25 N/mm^2
Column size = 650 mm *650 mm

Check for short or long column,
L_{ex}/D and L_{ey} /d < 12 for short column
L_{ex}/D and L_{ey}/d > 12 for long column
As per clause 25.1.3 IS 456 – 2000
Unsupported length (L) = 3000 – 650
= 2350 mm
Effective length (L_{eff}) = 0.65 * 2350
= 1527.5 mm
L_{eff}/D = 1527.5/650
= 2.35 < 12 so the column is short column.

Calculated minimum eccentricity,
e_{min} = L/500 +D/30 > 20 mm
= 2350/500 +650/30 > 20 m
= 26.36 mm > 20 mm
Also,
$e_{min} \ngtr$ 0.05* 650 = 32.5 mm
Hence the column is designed for axial load only.
As per clause 39.3, IS 456:2000, if minimum eccentricity is not greater than 0.05D then factored load is written as,
$P_u = 0.4* f_{ck}* A_c + 0.67* f_y* A_{sc}$
Where, A_c= A_g - A_{sc}
= 650* 650 - A_{sc}= 422500 - A_{sc}
Then,

$6118.73 * 10^3 = 0.4 * 25 * (422500 - A_{sc}) + 0.67 * 415 * A_{sc}$
$6118730 = 4225000 - 10\ A_{sc} + 278.05\ A_{sc}$
$268.05\ A_{sc} = 1893730$
$A_{sc} = 7064.83\ mm^2$
As SAP analysis provide around 14000 mm^2 of reinforcement, lets us provide 16 numbers of 32mm ϕ bar .

DESIGN OF TRAVERSE REINFORCEMENT:-
As per **Clause 26.5.3.2** of **IS 456-2000**
Diameter of lateral ties (ϕt) ≥ϕ L/4=32/4=8 mm
Hence provide 8 mm ϕ lateral ties.
Pitch distance ≤ least lateral dimension =350mm
≤ 16*smallest diameter of longitudinal reinforcement
=16*20 =320 mm
≤ 300 mm
Hence, provide lateral ties & hooks of ϕ-8 mm @300 mm.

4.2.4 DESIGN OF FOUNDATION
Raft foundation is designed for this building because the loads transmitted by the columns in this structure are so heavy and the allowable soil bearing pressure so small that individual footing would cover more than about one half of the area. The raft foundation is divided into series of continuous strip. The shear and bending moment diagrams may be drawn using continuous beam analysis or coefficients for each strip. The depth is selected to satisfy shear requirements. The steel requirements will vary from strips. This method generally gives a conservative design since the interaction of adjacent strips is neglected.

Load Calculation
The loads from the column footings are calculated from SAP, the loads are assumed to be distributed uniformly by the slab to the foundation soil.
Data Obtained From Sap Analysis:

TABLE: Joint Reactions			
Joint	**OutputCase**	**CaseType**	**F3 (KN)**
A1	UDCON2	Combination	5118.481
A2	UDCON2	Combination	5360.121
A3	UDCON2	Combination	2902.891
A4	UDCON2	Combination	4090.279
A5	UDCON2	Combination	4466.677
A6	UDCON2	Combination	5192.556
B1	UDCON2	Combination	6556.209
B2	UDCON2	Combination	6929.087
B3	UDCON2	Combination	8370.574
B4	UDCON2	Combination	7565.529
B5	UDCON2	Combination	6303.152
B6	UDCON2	Combination	6486.414
C1	UDCON2	Combination	6453.461
C2	UDCON2	Combination	6936.941
C3	UDCON2	Combination	8017.887
C4	UDCON2	Combination	7781.535
C5	UDCON2	Combination	6714.603
C6	UDCON2	Combination	6542.008
D1	UDCON2	Combination	4163.216
D2	UDCON2	Combination	3650.559
D3	UDCON2	Combination	7263.141
D4	UDCON2	Combination	7712.675
D5	UDCON2	Combination	6735.076
D6	UDCON2	Combination	6130.163
F1	UDCON2	Combination	3566.829

F2	UDCON2	Combination	5277.387
F3	UDCON2	Combination	6989.556
F4	UDCON2	Combination	6914.215
F5	UDCON2	Combination	5940.435
F6	UDCON2	Combination	3744.594
G1	UDCON2	Combination	2099.817
G2	UDCON2	Combination	3653.785
G3	UDCON2	Combination	4404.259
G4	UDCON2	Combination	4210.839
G5	UDCON2	Combination	2449.691
G6	UDCON2	Combination	1518.701

INPUT DATA

Bearing capacity of soil = 150 KN/m^2

Compressive strength of concrete used = 25 KN/m^2

Strength of reinforcement use = 415 KN/m^2

It is decided to select Raft footing since the area covered by the individual isolated footing is more than 50% of the plinth area.

CALCULATIONS:

$\overline{y}$

=

[(6556.209+6929.087+8370.574+76565.529+6303.152+6486.414)*5.5+6543.461+6936.941+8017.857+7781.535+6714.503+6552.008)*13.5+(4163.216+3650.559+7263.141+7712.675+6735.076+6130.163)*19+(3566.829+5277.38+6989.556+6914.215+5940.465+3744.594)*27+(2099.81+3653.785+4404.259+4210.839+2449.691+1518.701)*32.5/198213.3

= 2954275/198213.3

=14.90

e_y=16.245-14.90

=1.35

$\overline{x}$

=

[(5360.121+6929.08+6936.941+3650.559+5277.38+3653.785)*5+(2902.891+8370.574+8017.887+7263.141+6989.556+4404.259*10.5+(490.279+7565.529+7781.535+7712.675+6914.215+4210.8369)*15.5+(4466.67+63.3.152+6714.603+6735.076+5940.435+2449.691)*20.5+(5192.556+6486.414+6542.008+6130.163+3744.594+1518.701)*25.5/198213.3

e_x=12764-12.75

=0.014

$I_y = 33.8*26.8^3/12 = 54217.54$

A = 26.8*33.8 = 905.84

$M_x = P*e_y$ = (198213.3*1.35) = 267587.955 KNm

M_y = 198213.3*0.014 = 2774.9862 KNm

Soil pressure at different points as follows

$\sigma = \frac{P}{A} \pm \frac{My}{Iy} * x \pm \frac{Mx}{Ix} * y$

= 198213.3/905.84 ± (267587.955/54217.54)* x ± (267587.955/86238.98)*y

=218.81 ± 4.93x ± 3.10y

Calculation Table

S.N.	Column	P/A	My/Iy	Mx/Ix	X	Y	Stress
1	A1	218.18	4.93	3.1	-13.4	-16.25	335.247
2	B1	218.18	4.93	3.1	-13.4	-10.75	318.197
3	C1	218.18	4.93	3.1	-13.4	-2.75	293.397

4	D1	218.18	4.93	3.1	-13.4	2.75	276.347
5	F1	218.18	4.93	3.1	-13.4	10.75	251.547
6	G1	218.18	4.93	3.1	-13.4	16.25	234.497
7	A6	218.18	4.93	3.1	13.4	-16.25	203.123
8	B6	218.18	4.93	3.1	13.4	-10.25	184.523
9	C6	218.18	4.93	3.1	13.4	-2.75	161.273
10	D6	218.18	4.93	3.1	13.4	2.75	144.223
11	F6	218.18	4.93	3.1	13.4	10.25	120.973
12	G6	218.18	4.93	3.1	13.4	16.25	102.373

Maximum soil pressure is at, A1 = 335.247 KN/m^2
B_6 = 335.247 KN/m^2
Moment at A-A=838.11=335.247*0.5^2/10
Moment at B-B= (335.4+318.197)*0.5*5^2/10 =795.49
Moment at C-C= (318.197+293.397)0.5*5^2/10=764.4925
Moment at D-D = (293.397 + 276.347)*0.5*5^2/10=712.18
Moment at E-E= (276.347+251.547)*0.5*5^2/10 =659.86
Moment at F-F= (251.547 +234.497) *5^2/10 =607.555
Moment at Strip A-A=335.247*5.5^2/10=1014.12
τ'=0.25$\sqrt{25}$
=1.25 N/mm^2
For corner load
Perimeter: b_o =2*(0.5d+0.975)
=d + 1950
τ_v=V_x /(bo +d)
1.25 =(5118.481*1000)/(d+1950)*d
X1=1271.1
D=1271.1 mm
In raft foundation d should not be less than 600mm so d=1271.1 mm is okay.
Let us adopt d=1300mm
D=1300+50mm= 1350mm
For edge column B6,
Perimeter (b_o) = 2*(0.5*d+ 1200) + (400+d)
= 2d+2800

REINFORCEMENT:
Maximum moment= 1014.122 KNm
1014.122* 10^6= 0.87* 415* A_{st}* [1300- {(415*A_{st})/ (25*1000)}]
1014.122* 10^6=361[1300-415Ast/25*1000]
= 361Ast [1300-0.0166]
= 469300Ast-5.9926Ast^2
X1 = 76089.16 mm2
X2 = 2224.08
Let us adopt value of 76089.16 mm^2, then no. of longitudinal bar required;
Total area / 25 mm dia. bar area gives nearly 155 of bars along 26.88 m.
Adopting 180 mm c/c (from general spacing calculation)

Similarly along 33.8 m, 154 no. of bar is provided with 220 mm c/c distance.

4.2.5 DESIGN OF STAIRCASE
4.2.5.1 OPEN WELL STAIRCASE
Floor height= 3500 mm
Size of stair hall= 5100 mm * 4900 m
Assume 2 flights,
Height of one flight= total floor height/ no. of flight
= 3500/2

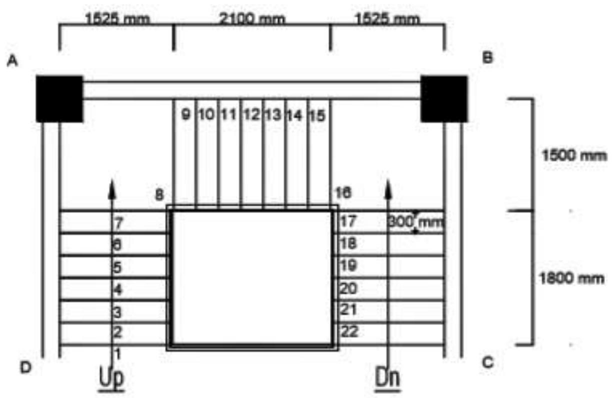
=1750 m
As this building is a commercial complex, let us adopt the height of riser of 150 mm and tread of 300 mm.
Keeping two flights, no. of riser in each flight= 1/2 *(3500)/150
= 12
Therefore, no. of treads in each flight= 12-1= 11
Let us take width of landing A and B= 1525 mm.
Horizontal distance required to accommodate these= (300* 11+ 1525)
= 4825 mm
Width of passage= 5100- 4825 = 275 mm, which is not sufficient. Also, in commercial building, no. of treads in one flight is limited to 9.

Therefore, open well staircase is preferable.
Hence, let us provide 7 treads in the landing portion, which can be easily accommodated in a width= 7* 300= 2100 mm, which will be equal to the width of well.
Provide 6 treads in each flight. Thus, there will be total of 6+6+7= 19 treads.

The stairs will be of quarter landing type. Total no. of risers to accommodate 19 treads in three flights will be = 19+ 3= 22 risers.
Height of riser= 3500/22 = 159 mm.

Thus, the steps will have risers of 159 mm and treads of 300 mm. Horizontal space required for 6 treads = 300* 6= 1800 mm.
Therefore, width of passage left= 5100-(1800+ 1525)
= 1775 mm.

ARRANGEMENT OF STAIRS:



Plan of open well staircase

Stair hall dimension= 5100 mm* 4900 mm
Floor height= 3500 mm
Riser (R)= 159 mm
Tread (T)= 300 mm
No. of treads= 19
No. of riser= 22
Landing size= 1525 mm* 1500 mm
Type of staircase= open well staircase
Grade of concrete used= M25
Grade of steel= Fe415

LOAD CALCULATIONS:

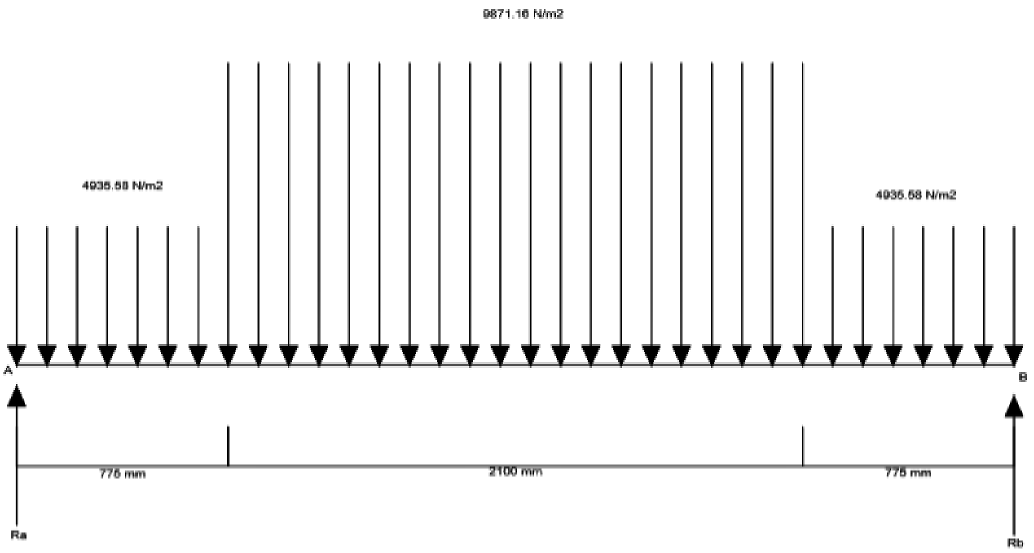
Let us provide waist slab of 150 mm thickness.
Dead load of waist slab = b*D*l*25 [density of concrete= 25 KN/m^3]
= 150*25
=3750 N/m^2 in inclined form
Ceiling finish= 300 N/m^2
Total load from waist slab, W_s= 3750+ 300= 4050 N/m^2

Corresponding load per sq. m. on plan (Load in horizontal slab),
W= $W_s*\sqrt{(R^2+T^2)}/ T$
= 4583.66 N/m²

Load per sq. m. on plan:
Waist and ceiling finish= 4583.66 N/m^2
Dead load of step (159/2 mm average) = 79.5* 25
= 1987.5 N/m^2
Top finish= 300 N/m^2
Live load= 3000 N/m^2

Total load= 4583.66+ 1987.5+ 300+ 3000 = 9871.16 N/m^2
Since the landing slab is two way slab, the load on the landing slab may be taken as
= 9871.16/2
= 4935.58 N/m^2

SHEAR FORCE AND BENDING MOMENT
Flight AB:



Loading on face AB

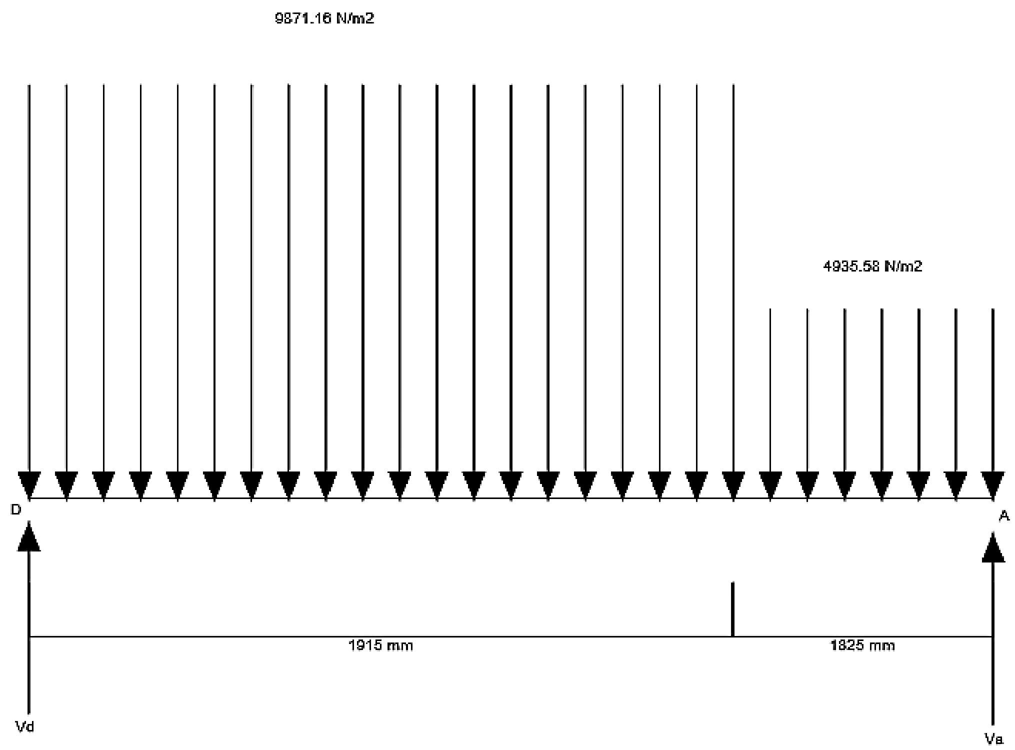
Effective span= centre line of landing A to the centre line of landing B
= 775+ 2100+ 775 = 3650 mm
Consider a 1000 mm wide strip of flight,
Reaction at supports= (4935.58* 1550+ 9871.16*2100)/2
= 14189.79 N
Maximum bending moment,
M= (14189.79* 3.650)/2 + 4938.58*0.775*(0.755+ 2.1)/2 + 9871.16*(1.05^2/2)
= 15004.158 Nm
Ultimate moment, M_u= 1.5* 15004.158 = 22506.237 Nm
Also, M_u= 0.138* f_{ck}*b*d^2
22506.237* 1000= 0.138*25*1000*d^2
d= 80.76 mm
Providing 10 mm diameter bars at a clear span of 25 mm,
Effective depth available= d= 150- 30= 120 mm.
Now,
M_u/b*d^2= 22506.237*1000/(1000* 120^2) = 1.56

Percentage of steel, P_t= $50*\frac{1-\sqrt{1-\frac{4.6*Mu}{fck*b*d^2}}}{\frac{fy}{fck}}$
= 0.468%

A_{st}= (0.468*1000*120)/100

= 561.6 mm^2
Area of one 10 mm diameter bar= 78.54 mm^2
Spacing of 10 mm dia. Bars= (78.54*1000)/ 561.6 = 140 mm
Hence provide 10 mm diameter bars @ 140 mm c/c.

Flight DA= distance from centre of beam to centre of bearing on wall
= (325+ 1500+ 1800+ 115)mm = 3740 mm



Loading on face DA

Load on landing= 4935.58 N/m^2
Load on waist slab= 9871.16 N/m^2
Consider a 1000 mm strip width of the flight. Fig. 2 shows the loading on the strip.
Taking moment about D,
V_a* 3.74= 9871.16*1.915^2/2 + 4935.58*(1.825/2)* {1.915+(1.825/2)}
V_a= 8244.42 N

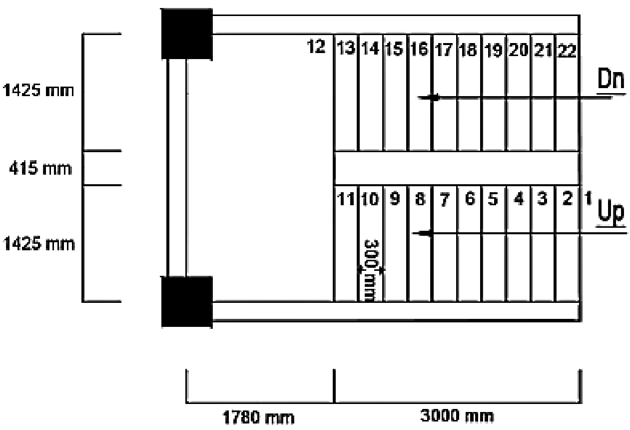
+↑$\sum F_y$=0
V_d+ V_a= 9871.16* 1.915+ 4935.58* 1.825
V_d= 19666.28 N

Let the shear force be zero at a distance of x m from D.
Then, 19666.28- 9871.16* x= 0
Therefore, x= 1.99 m.
Maximum bending moment = 19666.28* 1.99- 8244.42* 1.99^2/2
= 22811.53 Nm
Ultimate moment, M_u= 1.5* 22811.53= 34217.30 Nm
$M_u/b*d^2$ = $(34217.30*1000)/(1000*120^2)$= 2.376

Percentage of steel, $P_t=50*\frac{1-\sqrt{1-\frac{4.6*Mu}{fck*b*d^2}}}{\frac{fy}{fck}}$
= 0.75%

A_{st}= (0.75*1000*120)/100
=900 mm^2
Spacing of 10 mm diameter bars= (78.54*1000)/900= 87.26 mm
Hence provide 10 mm diameter bars @ 87.26 mm c/c.

4.2.5.2 DOG LEGGED STAIRCASE



Plan of dog legged staircase

Stair hall dimension= 3740 mm * 4780 mm
Height of each flight= 3500/2= 1750 mm
Choosing the dimension of riser and tread as used in open well staircase, we get
Riser (R)= 159 mm
Tread (T)= 300 mm
Then, no. of risers= 1750/159= 11 risers
No. of treads= 11-1=10 treads
Now, width of landing= 4780-(300*10)= 1780 mm

Design of flight AB:
Let the bearing for the flight be,
Effective horizontal span= 3000+ 1780= 4780 mm
LOADS:
Dead load of 150 mm waist slab= 25*150 = 3750 N/m^2
Ceiling finish (12.5 mm)= 24*12.5= 300 N/m^2
Total= 4050 N/m^2
Corresponding load per sq. m. on plan= 4050* $\sqrt{(R^2+T^2)}/T$
= 4583.66 N/m^2
Hence the actual load per sq. m. of plan area will consist of following:
Waist and ceiling finish= 4583.66 N/m^2
Dead load of steps (159/2 average)= 79.5*25= 1987.5 N/m^2
Top finish (12.5 mm)= 12.5*24= 300 N/m^2
Live load= 3000 N/m^2
Total load= 9871.16 N/m^2

Maximum bending moment per metre width of stairs= $(9871.16*4.78^2)/2$
= 28192.53 N/m^2
Ultimate moment, M_u= 1.5* 28192.53= 42288.79 Nm
That is, $0.138*f_{ck}*b*d^2$= 42288.79* 10^3
That is, d= 110.71 mm
Providing 10 mm diameter bars with 30 mm clear cover,
Overall depth required= 110.71+ 30= 140.71 mm
Hence provide an overall depth of 150 mm.
Therefore, effective depth, d= 150-30= 120 mm.

Now,
$M_u/b*d^2= 42288.79*10^3/(1000*120^2)= 2.936$

Percentage of steel, $P_t= 50*\frac{1-\sqrt{1-\frac{4.6*Mu}{fck*b*d^2}}}{\frac{fy}{fck}}$

= 0.96%

$A_{st}= (0.96*1000*120)/100 = 1152\ mm^2$

Spacing of 10 mm diameter bars= 78.54*1000/1152
= 68.17 mm

Hence provide 10 mm diameter bars @ 68.17 mm.

V. CONCLUSION

This project was chosen because of the recent earthquake's disastrous damages to the houses and buildings in Nepal which caused tremendous loss of human life as well. Most of the loss of lives was because of the collapse of the building. Due to this catastrophic incident, we, as the future engineers, have to make the effect of the earthquake minimum. The goal being to try and make the buildings completely earthquake resistant so we can at lessen the loss of lives in future disasters.

From the analysis and design of the building that we did in this project, we came in conclusion that a building can withstand substantial amount of earthquake forces to let us stay safe at the moment earthquake strikes and allows insiders to prepare for safety measures. There is no technology that can guarantee that the building can withstand the earthquake forces to the fullest but we can at least make time to be safe from the damages if we design the building as per the proposed model.

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