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Analysis and Design of G+15 Building With Connecting Skywalk in Gorakhpur

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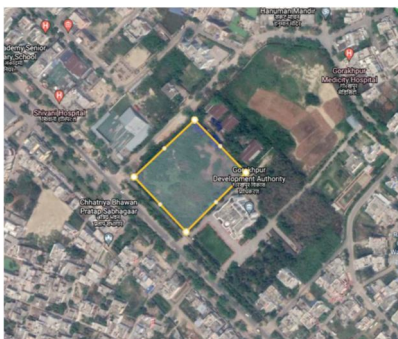
Abstract: The structural analysis and design of G+15 building with the connection skywalk is done using software StaddPRO V8i. The planning is done in AUTOCAD 2019. The G+15 building with the connection skywalk which connect the two buildings to facilitates the movement of the people inside the building. The plan of the G+15 building will be drawn. The analysis of the speed, quality and construction aspects will be done. The building will be designed as per the IS codes. The various loads acting on the G+15 building will be calculated based on the Indian Standard Code books. It gives a platform to work hand to hand with the people of other department such IT department. The building will be designed keeping the economic aspect in mind with sufficient durability and strength.

I. INTRODUCTION

The high rise building concept which is the demand of the future generation. We are moving toward the era where the land is less and the population demand is quite high. To meet the basic demand of each individual human being was residence which can be fulfilled by the making the high rise building. The tall building can fulfill the demand. They save space and accommodate more residents as compared to individual houses. Tall buildings provide a aesthetic and a modern look to the city. Due to much space inside of the building it could fit more than one company in it. Hence saves much space in the city. Another advantage of the tall building is that it can saves more land for agricultural purpose. Hence in this project we have planed, analysed and designed the G+15 building with the connection skywalk which connect the two buildings to facilitates the movement of the people inside the building. The successful construction of G+15 buildings are based upon clear understanding of the conduct of the structure, relevant analysis theory and methodologies, use of software, and explicit design principles.

II. PLANNING

This building was being made in the location of taramandal which is located in the Gorakhpur, Uttar Pradesh. The location coordinates of the site is $26^{\circ}43'54.6''\text{N}$ $83^{\circ}22'53.2''\text{E}$ and the altitude is 73.7176827. The total plot area required for this peoject was 9000m². The total height of the building is 48 m. The building connecting skywalk at the 8th floor of the building which is of 9 m in length. The building project was economical and this project help us to understand the various components of the building and the loads acting on the building and their effects on the building. And to design the structural components like slab, beam, column, foundation of the G+15 building. The plan of the G+15 building will be drawn. The analysis of the speed, quality and construction aspects will be done. The various loads acting on the G+15 building will be calculated based on the Indian Standard Code books. It gave us the opportunity to work with the other department students which help us to gain knowledge.



A. Data Collection

TABLE I

Dead Load Value	4.125KN/m
Live Load Value	5.5KN/m
Loading Cases	DL, WL, EQL, LL
External Loads	Wind, Earthquake
Wind Speed	47 m/s
Earthquake Zone	III
Foundation Type	Isolated Footing

B. Building Details

TABLE II

Total Length	50m
Total Width	26m
Height of each floor	3m
Total floor height	48m
Total no. of floors	G+15
Length of skywalk	9m
Location of skywalk	8th floor

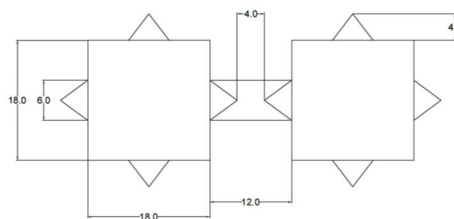


Fig 1. Shown the top view of the building on AutoCAD

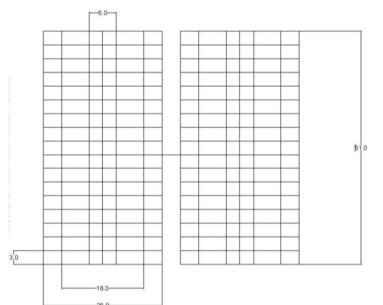


Fig 2. Shown the front view of the building on AutoCAD

C. Analysis

Based on the information obtained from many relevant sources, the design was examined in STAAD.Pro. The structure was examined in order to determine its limit and, as a result, a critical value that could be used to build the structure. The structure was tested under self-weight, axial load, shear along the Y , Z axes, and bending along the Y ,Z axes. The results of the investigation are displayed below.

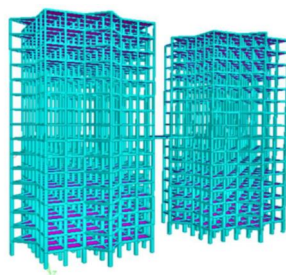


Fig 2. Rendered view

1) Bending Moment

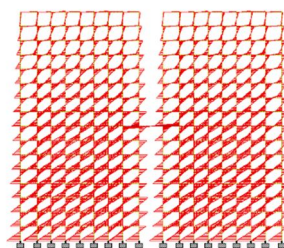


Fig. 3. Shows the bending moment in the building.

TABLE III

Direction of bending moment	Maximum positive BM (Kn-m)	Maximum negative BM (Kn-m)	Load combination (Kn-m)
My	48.419	48.546	1.5(DL+LL)
Mz	73.559	53.433	1.5(DL+LL)

2) Shear Force

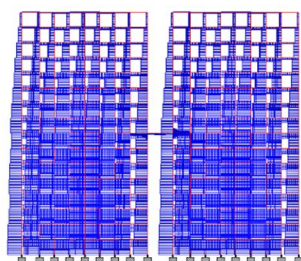


Fig 4. Shows the shear force in the building

TABLE IV

Direction of shear force	Maximum positive SF (kN)	Maximum negative SF (kN)	Load combinations (kN)
F _y	36.869	37.316	1.5(DL+LL)
F _z	36.869	49.381	1.5(DL+LL)

3) Torsion

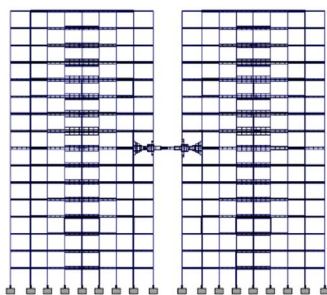


Fig 5. Shows the torsional force in the building

TABLE V

Direction of Torsion	Maximum torsion	Minimum Torsion	Load combination
M_x	23.770	24.644	1.5(DL+LL)

D. Design

The design of the components was done based on the data obtained from analysis.

1) Design Of Slab

$$\frac{l_y}{l_x} = \frac{3}{3} = 1 < 2 \text{ two-way slab}$$

Effective depth of slab = 125 mm

D = 150mm

Step 1: Load Calculation

$$\text{Self weight (slab)} = 0.125 \times 25 = 3.125 \frac{\text{KN}}{\text{m}^2}$$

$$\text{Floor finish} = 1 \frac{\text{KN}}{\text{m}^2}$$

$$\text{L.L.} = 5.5 \frac{\text{KN}}{\text{m}^2}$$

$$\text{Total load} = 12.125 \frac{\text{KN}}{\text{m}^2}$$

$$\text{Factored load} = 12.125 \times 1.5 = 18.187 \frac{\text{KN}}{\text{m}}$$

seismic zone- zone = 3

Response factor = 3

Zone factor = 0.16

Importance factor = 1

$$\begin{aligned} \text{Factored Moment (M}_{ux}) &= \alpha_x \times W_u \times l_x^2 \\ &= 0.0672 \times 18.187 \times 3^2 \\ &= 10.9 \text{ KN-m} \end{aligned}$$

$$\begin{aligned} \text{Factored Moment (M}_{uy}) &= \alpha_y \times W_u \times l_y^2 \\ &= 0.056 \times 18.187 \times 3^2 = 9.166 \text{ kN-m} \end{aligned}$$

Step 2: Check for depth

$$M_{u \text{ limit}} = 0.133 f_{ck} b d^2$$

$$10.9 \times 10^6 = 0.133 \times 30 \times 1000 \times d^2$$

$$d = 52.26 < 150$$

Depth of 150 mm is adequate

Step 3: Calculation of ast

$$\frac{M}{bd^2} = \frac{10.9 \times 10^6}{1000 \times 150^2} = 0.4 \text{ mm}^2$$

$$\text{sp16 table 4 pt} = 0.344$$

$$A_{st} = \frac{\text{pt bd}}{100} = \frac{0.344 \times 1000 \times 150}{100} = 516 \text{ mm}^2$$

$$\text{Diameter of bar} = 10 \text{ mm}$$

$$\text{Area of bar} = \frac{\pi}{4} \times 10^2 = 80 \text{ mm}^2$$

$$\text{Spacing} = \frac{80 \times 1000}{516} = 155 \text{ mm}$$

Use 10 mm diameter bar @ 150 mm c/c

LONGER DIRECTION:

$$d = 150 - 10 = 140 \text{ mm}$$

$$\frac{M}{bd^2} = \frac{9.16 \times 10^6}{1000 \times 140^2} = 0.467$$

$$\text{from sp16 table 4 pt} = 0.342$$

$$A_{st} = \frac{0.342 \times 1000 \times 140}{100} = 479 \text{ mm}^2$$

$$\text{spacing} = \frac{80 \times 1000}{480} = 167 \text{ mm}$$

use 10mm diameter bar @ 160 mm c/c

2) Design Of Column (1 TO 8 FLOOR)

$$L = 3 \text{ m}$$

$$\text{Yield strength} = 600 \frac{\text{N}}{\text{mm}^2}$$

$$\text{Compressive strength} = 40 \frac{\text{N}}{\text{mm}^2}$$

$$\text{Dimension of column} = 500 \times 500$$

$$P_u (\text{factored load}) = 3000 \text{ KN}$$

Step 1: Calculation of Minimum Eccentricity

$$e_{\min} = \frac{L}{500} + \frac{D}{30} = \frac{3000}{500} + \frac{500}{30} = 22.66 \text{ m}$$

$$e_{\max} = 0.05 \times D = 0.05 \times 500 = 25 \text{ m}$$

$$e_{\min} < e_{\max}$$

Step 2: Check for short or long column

$$\frac{L_{ex}}{D} = \frac{3000}{500} = 6$$

$$\frac{L_{ey}}{D} = \frac{3000}{500} = 6$$

$$\frac{L_{ex}}{D} \text{ and } \frac{L_{ey}}{D} < 12$$

∴ The column is short column.

Step 3: Reinforcement calculation

$$P_u = 0.4 \times f_{ck} \times A_c + 0.67 f_y \times A_{sc}$$

$$A_c = A_g - A_{sc}$$

$$P_u = 0.4 \times f_{ck} \times (A_g - A_{sc}) + 0.67 \times f_y \times A_{sc}$$

$$P_u = 0.4 \times f_{ck} \times A_g + (0.67 f_y - 0.4 f_{ck}) A_{sc}$$

$$A_{sc} = 3715.17 \text{ mm}^2$$

assuming 16 mm diameter bars

$$A_{st} = \frac{\pi}{4} \times 16^2 = 200 \text{ mm}^2$$

$$\text{No. of rods} = A_{sc} \times \frac{A_{st}}{A_{st}} = \frac{3715.17}{200} = 19$$

∴ Take 20 no. of rods

$$A_{sc} \text{ provided} = 20 \times 200 = 4000$$

Step 4: Design of Lateral ties

1. Tie diameter = $\frac{1}{4} \times \text{diameter of bar} = 4 \text{ mm}$

2. $\leq 16 \text{ mm}$

Provide 8 mm diameter ties.

Ties spacings: -

Provide minimum of below three conditions.

(a) Size of column i.e, 500 mm

(b) $16 \times \text{longitudinal diameter} = 16 \times 16 = 256 \text{ mm}^2$

(c) 300 mm

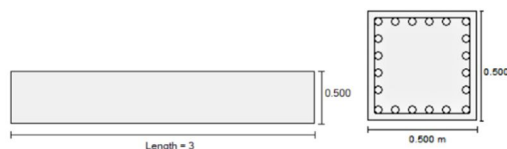


Figure 6: Design of Column (1-4 floor)

3) Design Of Column (9 to 16 floor)

$$L = 3 \text{ m}$$

$$\text{Yield strength} = 600 \frac{\text{N}}{\text{mm}^2}$$

$$\text{Compressive strength} = 40 \frac{\text{N}}{\text{mm}^2}$$

$$\text{Dimension of column} = 400 \times 400$$

$$P_u (\text{factored load}) = 3000 \text{ kN}$$

Step 1: Calculation of Minimum Eccentricity

$$E_{\min} = \frac{L}{500} + \frac{D}{30} = \frac{3000}{500} + \frac{400}{30} = 19.33 \text{ m}$$

$$E_{\max} = 0.05 \times D = 0.05 \times 400 = 20 \text{ m}$$

$$E_{\min} < E_{\max}$$

Step 2: Check for short or long column

$$\frac{L_{\text{ex}}}{D} = \frac{3000}{400} = 7.5$$

$$\frac{L_{\text{ey}}}{D} = \frac{3000}{400} = 7.5$$

$$\frac{L_{\text{ex}}}{D} \text{ and } \frac{L_{\text{ey}}}{D} < 12$$

∴ The column is short column.

Step 3: Reinforcement calculation

$$P_u = 0.4 \times f_{ck} \times A_c + 0.67 f_y \times A_{sc}$$

$$A_c = A_g - A_{sc}$$

$$P_u = 0.4 \times f_{ck} (A_g - A_{sc}) + 0.67 \times f_y \times A_{sc}$$

$$P_u = 0.4 \times f_{ck} \times A_g + (0.67 f_y - 0.4 f_{ck}) A_{sc}$$

$$A_{sc} = 1322.47 \text{ mm}^2$$

assuming 12 mm diameter bars

$$A_{st} = \frac{\pi}{4} \times 12^2 = 114 \text{ mm}^2$$

$$\text{No. of rods} = A_{sc} \times \frac{A_{sc}}{A_{st}} = \frac{1300.4}{114} = 11.6$$

∴ Take 12 no. of rods

$$A_{sc} \text{ provided} = 12 \times 114 = 1368$$

Step 4: Lateral ties design

1. Tie diameter = $\frac{1}{4} \times \text{diameter of bar} = 3 \text{ mm}$

2. $\leq 16 \text{ mm}$

Provide 8 mm diameter tie.

Ties spacings: -

Provide minimum of three condition.

(a) Size of column i.e, 400 mm

(b) $16 \times \text{longitudinal diameter} = 16 \times 12 = 192 \text{ mm}^2$

(c) 200 mm



Figure 7: Design of Column (5-8 floor)

4) Design Of Beam (1 -8 Floor)

Beam design for 3 m span

Effective length of beam = 3 m

Width of beam $b = 500 \text{ mm}$

Grade of concrete $M40 = 40 \frac{N}{\text{mm}^2}$

Grade of steel = $600 \frac{N}{\text{mm}^2}$

Step 1: Cross Sectional Area

Assume span depth ratio as 15

Effective depth $d = \frac{\text{span}}{15} = \frac{3000}{15} = 200 \text{ mm}$

Adopt $d = 450 \text{ mm}$, clear cover as 50 mm

Total depth $D = 500 \text{ mm}$

Step 2: Load Calculation

Self weight of beam = $0.23 \times 0.5 \times 25 = 2.875 \text{ kN/m}$

D.L. = $25.8 \frac{\text{kN}}{\text{m}^2}$

Total dead load = $28.8 \frac{\text{kN}}{\text{m}^2}$

Live load = $12 \frac{\text{kN}}{\text{m}^2}$

Total load = $40.8 \frac{\text{kN}}{\text{m}^2}$

Factored load = $1.5 \times 40.8 = 61.2 \text{ kN-m}$

Ultimate bending moment and shear force

$M_u = \frac{wl^2}{8} = \frac{61.2 \times 3^2}{8} = 68.85 \text{ kN-m}$

$V_u = \frac{wl}{2} = \frac{61.2 \times 3}{2} = 96.3 \text{ kN}$

Step 3: Limiting Moment Resistance

$M_u \text{ limit} = 0.133 f_{ck} b d^2$

$= 0.133 \times 40 \times 500 \times 500^2$

$= 665 \text{ kN m}$

$M_u < M_{u \text{ lim}}$ (section is underreinforced)

Step 4: Main Reinforcement

$M_u = (0.87 \times f_y \times A_{st} d) [1 - \frac{A_{st} f_y}{b d f_{ck}}]$

$68.85 \times 10^6 = 1744000 A_{st} - 12.16 A_{st}^2$

$A_{st} = 421 \text{ mm}^2$

Provide 6 bars of 12mm diameter bar $A_{st} = 6 \times \frac{\pi}{4} \times 12^2 = 678.24 \text{ mm}^2$

Step 5: Shear Reinforcements

$\tau_v = \frac{V_u}{b d} = \frac{96.3 \times 10^3}{500 \times 500} = 0.385$

$$p_t = \frac{100 A_{st}}{b d} = \frac{100 \times 678.24}{500 \times 500} = 0.271$$

from table 19 of IS 456 $\tau_c = 0.5612$

Since $\tau_c < \tau_v$, shear reinforcement is to resist the balance shear computed below

$$V_{us} = V_u - (\tau_c \times b d)$$

$$V_{us} = 96.3 \times 10^3 - (0.56 \times 500 \times 500) = 43.7 \text{ kN}$$

Using 8 mm dia bar @ 2 legged stirrups

$$S_v = \frac{0.877 F_y \times A_{svd}}{V_{us}}$$

$$S_v = 191$$

$$S_v \geq 0.75 \times d$$

$$= 0.75 \times 400 = 300$$

$$S_v \geq 300$$

Step 6: Check for Deflection

$$P_t = 0.22, k_t = 1.2, K_c = k_f = 1$$

$$\left(\frac{L}{d}\right)_{\max} = \left(\frac{L}{d}\right)_{\text{basic}} \times k_t \times k_c \times k_f$$

$$= \left(\frac{3000}{400}\right) \times 1.2 \times 1 \times 1 = 9$$

$$\left(\frac{L}{d}\right)_{\text{actual}} = \frac{3000}{400} = 7.5 < 9$$

Hence deflection control is satisfied.

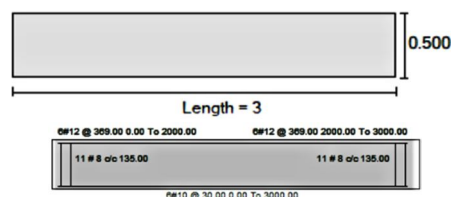


Figure 10: Design of Beam (1-8 floor)

5) Design Of Beam (9 TO 16 Floor)

Beam design for 3 m span

Effective length of beam = 3 m

Width of beam $b = 400 \text{ mm}$

Grade of concrete $M40 = 40 \frac{N}{\text{mm}^2}$

Grade of steel = $600 \frac{N}{\text{mm}^2}$

Step 1: Cross Sectional Area

Assume span depth ratio as 15

$$\text{Effective depth } d = \frac{\text{span}}{15} = \frac{3000}{15} = 200 \text{ mm}$$

Adopt $d = 450 \text{ mm}$, clear cover as 50 mm

Total depth $D = 500 \text{ mm}$

Step 2: Load Calculation

$$\text{Self-weight of beam} = 0.23 \times 0.5 \times 25 = 2.875 \text{ kN m}^2$$

$$D.L. = 25.8 \frac{\text{kN}}{\text{m}^2}$$

$$\text{Total dead load} = 28.8 \frac{\text{kN}}{\text{m}^2}$$

$$\text{Live load} = 12 \frac{\text{kN}}{\text{m}^2}$$

$$\text{Total Load} = 40.8 \frac{\text{kN}}{\text{m}^2}$$

$$\text{Factored load calculation} = 1.5 \times 40.8 = 61.2 \text{ KN m}$$

Ultimate bending moment and shear force

$$M_u = \frac{Wl^2}{8} = \frac{61.2 \times 3^2}{8} = 68.85 \text{ kN-m}$$

$$V_u = \frac{Wl}{2} = \frac{61.2 \times 3}{2} = 96.3 \text{ kN}$$

Step-3: - Limiting Moment Resistance

$$M_{u \text{ limit}} = 0.133 f_{ck} b d^2$$

$$= 0.133 \times 40 \times 400 \times 400^2 = 340 \text{ kN m}$$

$$M_u < M_{u \text{ lim}} \text{ (underreinforced)}$$

Step 4: Main Reinforcement

$$M_u = (0.87 \times f_y \times A_{st} \times d) \left[1 - \frac{A_{st} f_y}{b d f_{ck}} \right]$$

$$68.85 \times 10^6 = 1744000 A_{st} - 12.16 A_{st}^2$$

$$A_{st} = 421 \text{ mm}^2$$

$$\text{Provide 6 bars of 10mm diameter bar } A_{st} = 6 \times \frac{\pi}{4} \times 10^2 = 471 \text{ mm}^2$$

Step5: Shear Reinforcements

$$\tau_v = \frac{V_u}{b d} = \frac{96.3 \times 10^3}{400 \times 400} = 0.601$$

$$p_t = \frac{100 A_{st}}{b d} = \frac{100 \times 471}{400 \times 400} = 0.348$$

from table no. 19 of IS 456 the value of $\tau_c = 0.2937$

Since $\tau_c < \tau_v$, the balance shear computed as

$$V_{us} = V_u - (\tau_c \times b d)$$

$$V_{us} = 96.3 \times 10^3 - (0.56 \times 400 \times 400) = 67 \text{ kN}$$

Using 8 mm dia bar @ 2 legged stirrups

$$S_v = \frac{0.877 F_y \times A_{sv} d}{V_{us}}$$

$$S_v = 191$$

$$S_v \geq 0.75 \times d$$

$$= 0.75 \times 400 = 300$$

$$S_v \geq 300$$

Step-6: - Check for Deflection

$$P_t = 0.22, k_t = 1.2, K_c = k_f = 1$$

$$\left(\frac{L}{d}\right)_{\text{max}} = \left(\frac{L}{d}\right)_{\text{basic}} \times k_t \times k_c \times k_f$$

$$= \left(\frac{3000}{400}\right) \times 1.2 \times 1 \times 1 = 9$$

$$\left(\frac{L}{d}\right)_{\text{actual}} = \frac{3000}{400} = 7.5 < 9$$

Hence deflection control is satisfied.

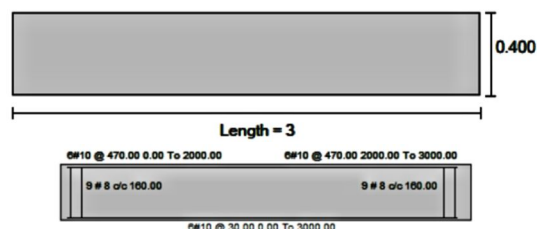


Figure 11: Design of Beam (9-16 floor)

6) Design Of Skywalk

Step: 1

$$\frac{l_y}{l_x} = \frac{9}{6} = 1.5 < 2$$

Hence, Two-way slab

Step 2: Depth of slab

$$\text{Depth} = \frac{\text{Span}}{28} = 214.28$$

$$d \approx 220 \text{ mm}$$

$$\text{F.O.S} = 1.5$$

Adapt clear cover of 20 mm and using 10 mm dia. Bar

Total depth is computed as

$$= 20 + 220 + 5 = 245 \text{ mm}$$

Step 3: Effective span

$$\text{Effective span} = \text{Clear span} + \text{Effective depth}$$

$$= 6 + 0.220$$

$$= 6.22 \text{ m}$$

Step 4: Calculation of load

$$\text{Self wt. of the slab} = 0.245 \times 25$$

$$= 6.125 \frac{\text{kn}}{\text{m}^2}$$

$$\text{L.L. on slab} = 4 \frac{\text{kn}}{\text{m}^2}$$

$$\text{Floor finish} = 0.6 \frac{\text{kn}}{\text{m}^2}$$

$$\text{Total working load} = 10.725 \frac{\text{kn}}{\text{m}^2}$$

$$\text{Ultimate load} = 1.5 \times 10.725$$

$$= 16.08 \frac{\text{kn}}{\text{m}^2}$$

Step 5: Moment and shear force

$$M_x = \alpha_x \times w l^2$$

$$M_y = \alpha_y \times w l^2$$

$$\alpha_x = 0.0885, \alpha_y = 0.057$$

$$M_{ux} = 0.0885 \times 16.08 \times 6.220^2 = 55.05 \text{ Kn.M}$$

$$M_{uy} = 0.059 \times 16.08 \times 6.220^2 = 36.70 \text{ Kn.M}$$

$$V_{ux} = \frac{W_x \times l_x}{2} = 50 \text{ Kn}$$

Step 6: Check for depth

$$M_{u_{lim}} = 0.138 \times f_{ck} \times b d^2$$

$$d = \sqrt{\frac{55.05 \times 10^6}{0.138 \times 20 \times 1000}} = 141.22 \text{ mm} < 220 \text{ mm}$$

Hence the effective depth is sufficient

$$A_{st(\min)} = 12\%$$

$$= \frac{0.12}{100} \times 1000 \times 245$$

$$= 294 \text{ mm}^2$$

Step 7: Reinforcement

$$M_u = 0.87 \times f_y \times A_{st} \times d \left[1 - \sqrt{\frac{A_{st} \times 415}{b d \times f_{ck}}} \right]$$

$$= 450 \text{ mm}^2$$

Step 8: Spacing

$$S = \frac{1000 \times a_{st}}{A_{st}}$$

$$= 174 \text{ mm}$$

Adapt 10mm bar @ 180 mm c/c

Use 10 mm bar in the long spam

$$\text{Effecive depth} = 220 - 10 = 210 \text{ mm}$$

$$M_u = 0.87 \times f_y \times A_{st} \times d \left[1 - \sqrt{\frac{A_{st} \times 415}{b d \times f_{ck}}} \right]$$

$$A_{st} = 315 \text{ mm}^2$$

$$\begin{aligned} \text{Spacing} &= \frac{1000 \times a_{st}}{A_{st}} \\ &= \frac{1000 \times 0.785 \times 102}{315} = 249 \text{ mm} \end{aligned}$$

Hence, provide 10mm dia. Bar @ 249 mm c/c

Step 9: Check for shear

$K\tau_c > \tau_v$ (Safe)

$K\tau_c < \tau_v$ (Unsafe)

$$\tau_v = \frac{Vu}{bd} = \frac{50 \times 103}{1000 \times 220} = 0.227$$

$$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 0.5 \times 450}{1000 \times 220} = 0.102$$

Permissible shear stress in slab is computed

$a_s = K\tau_c$ for the value of K refer 40.2.1.1 for 245

$$x = 0.08$$

$$\tau_c = 0.327$$

$$K\tau_c = 0.402 > \tau_v$$

Hence the slab is safe

Step 10: Check for deflection control

$$\left(\frac{L}{d}\right)_{\max} = \left(\frac{L}{d}\right)_{\text{basic}} \times k_t \times k_c \times k_f$$

$$k_t = 1.4$$

$$k_c = 1$$

$$k_f = 1$$

$$\left(\frac{L}{d}\right)_{\max} = 20 \times 1.4 = 28$$

$$\left(\frac{L}{d}\right)_{\text{provided}} = \frac{4150}{220} = 18.8 < 30$$

Hence the deflection control is satisfied.

Step 11: Torsion reinforcement (corners)

Area of the reinforcement in each layer

$$0.75 \times 415 = 307.5 \text{ mm}^2$$

$$\text{Divide} = \frac{\text{Shore tr span}}{5} = \frac{6000}{5} = 1200$$

$$\text{Spacing} = \frac{100 a_{st}}{A_{st}} = \frac{100 \times 0.785 \times 62}{307.5 \text{ mm}^2} = 90$$

Use 6 mm dia bar @ 100 mm c/c

Length of 1200 mm at all 4 corners in four layer.

Step 12: Reinforcement of edge stirrup

$A_{st} = 0.12\%$ of crosssection area

$$\frac{0.12}{100} \times 100 \times 170 = 204 \text{ mm}^2$$

Provide 10 mm dia bar at 300 mm c/c

7) Design Of Footing

Step 1: Footing Size

Loading on columns = 1000 Kn

$$\text{Weight of the footings and backfill @ } 10\% = \frac{10}{100} \times 1000 = 100$$

Total load = 1000 KN

$$\text{Area of footing} = \frac{1100}{200} = 5.5 \text{ m}^2$$

$$\text{Size of footing} = L = B = \sqrt{5.5} = 2.3 \text{ m}$$

Adopt 2.4×2.4 m (square footing)

$$Q_u = \frac{1000}{(2.4 \times 2.4)} \times 1.5 = 260 \frac{\text{kn}}{\text{m}^2}$$

Step 2: One-way shear

Critical section is at 'd' distance from the column face

$$\begin{aligned} \text{Factorial shear } V_u &= 0.26 \times 2400 \times (1000 - d) \\ &= 162 \times (1000 - d) \end{aligned}$$

Assuming reinforcement in the footing

Pt% = 0.25% fy M20 concrete

Permissible shear stress from table no. 19 IS5456 2000

$$\tau_c = 0.36 \frac{\text{N}}{\text{mm}^2}$$

$$\begin{aligned} \text{One way shear resistance } V_c &= 0.36 \times 2400 \times d \\ &= 864d \end{aligned}$$

$$624(1000 - d) = 864d$$

$$d = 722 \text{ mm} \approx 725 \text{ mm}$$

Step 3: Two-way shear

Assuming d=722mm

Two way shear at critical section $\frac{d}{2}$ from face of column.

$$\begin{aligned} V_u &= 0.26 [2400^2 - (400 + d)^2] \\ &= 0.26 [2400^2 - (400 + 725)^2] \\ &= 1170290 \text{ N} \end{aligned}$$

Two way shear

$$V_c = K_s \times \tau_c [4(400 + d)d]$$

$$\text{where } K_s = 1.0 \text{ and } \tau_c = 0.25 = 1.118 \frac{\text{N}}{\text{mm}^2}$$

$$\begin{aligned} V_c &= 1 \times 1.118 [4(400 + d)d] \\ &= 1788.8d + 4.722d^2 \\ &= d = 349 \text{ mm} \end{aligned}$$

Hence, one way shear is critical.

Adapting effective depth 'd' = 725 mm

And overall depth = 800 mm.

Step 4: Reinforcement Designs

Ultimate moment $M_u = 260 \times 1 \times 0.5 = 130 \text{ KNm}$

$$\frac{M_u}{bd^2} = \frac{130 \times 106}{1000 \times 725^2} = 0.247$$

$$A_{st} = \frac{P_t d}{1000} = \frac{0.25 \times 1000 \times 725}{1000} = 1813 \text{ mm c/c}$$

Use 20mmØ

$$\text{Spacing } S = \frac{1000 \times 12.56 \times 202}{1813} = 173 \text{ mm c/c}$$

Use 20 mmØ @ 170 mm c/c

Step 5: Transfer of force at column face

Ultimate load on the column base = $1.5 \times 1000 = 1500 \text{ KN}$

$$F_{br} = 0.45 F_{ck} \sqrt{\frac{A_1}{A_2}}$$

1. Column Face

$$F_{ck} = 20 \text{ N/mm}^2$$

$$\begin{aligned} A_1 = A_2 &= 400 \text{ mm}^2 \\ &= 0.45 \times 20 \times 1 = 9 \text{ N/mm}^2 \end{aligned}$$

2. At footing face

$$F_{ck} = 20 \text{ N/mm}^2$$

$$A_1 = 2400 \text{ mm}^2$$

$$A_2 = 4002$$

$$F_{br}(\text{footing}) = 0.45 \times 20 \times 2 = 18 \frac{N}{mm^2}$$

Hence govern by column face

$$F_{br} = 9 \text{ N/mm}$$

$$\text{Check for stress} = f_{br} = \frac{9 \times 4002}{1000} = 14000 \text{ Kn} < 15000 \text{ Kn}$$

Hence Safe

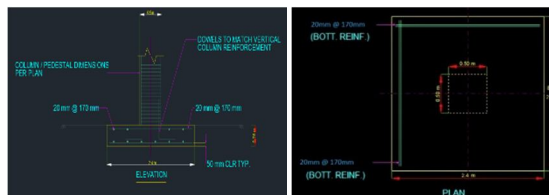


Figure 4.15: Design of foundation

III. CONCLUSION

The Planning, analysis and design of G + 15 building is done successfully. The methodology and plan for a G + 15 building is presented in this project. Limit state of method has been used as the method for the design of the building. The successful construction of G+15 buildings are based upon clear understanding of the conduct of the structure, relevant analysis theory and methodologies, use of software. They take longer to build and are generally 25-40% more expensive per square meter than a low-rise building. It will increase the pollution caused during the construction of a building. Hence followed the NBC guide lines during the construction. The design of joints connecting slabs and RC walls is designed constrain.

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