iJRASET
International Journal For Research in
Applied Science and Engineering Technology

INTERNATIONAL JOURNAL FOR RESEARCH
IN APPLIED SCIENCE & ENGINEERING TECHNOLOGY

Volume: 11 Issue: VII Month of publication: July 2023

DOI: https://doi.org/10.22214/ijraset.2023.55082

International Journal for Research in Applied Science & Engineering Technology (IJRASET)
ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor: 7.538
Volume 11 Issue VII Jul 2023- Available at www.ijraset.com

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

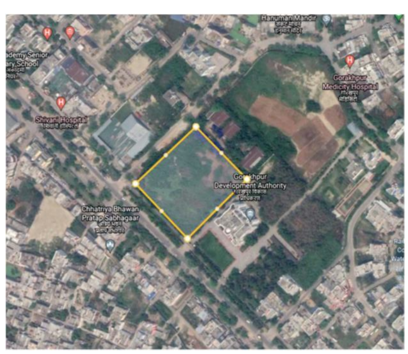
The high rise building concept which is the demand of the future generation. We are moving toward the era where the land is less and the population demand is quite high. To meet the basic demand of each individual human being was residence which can be fulfilled by the making the high rise building. 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°43'54.6"N 83°22'53.2"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.



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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

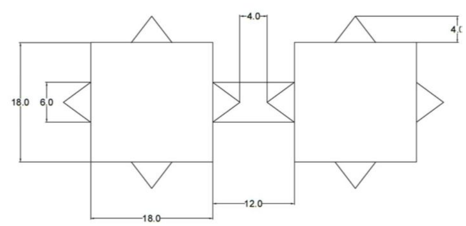

Fig 1. Shown the top view of the building on AutoCAD

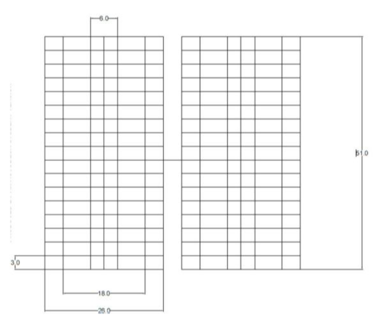

Fig 2. Shown the front view of the building on AutoCAD

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.

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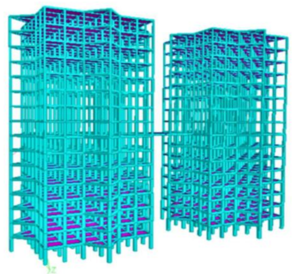

Fig 2. Rendered view

1) Bending Moment

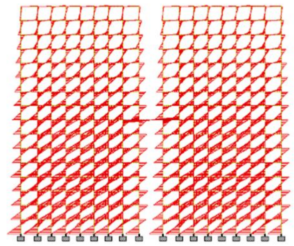

Fig. 3. Shows the bending moment in the building.

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

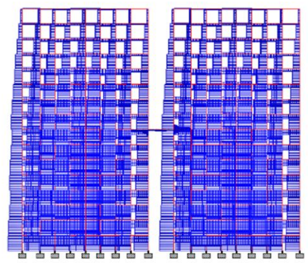

Fig 4. Shows the shear force in the building

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)

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3) Torsion

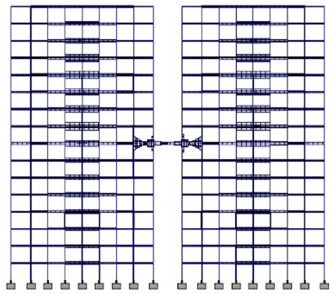

Fig 5. Shows the torsional force in the building

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

$\frac{ly}{lx} = \frac{3}{3} = 1 < 2$ two-way slab

Effective depth of slab = 125 mm

D = 150mm

Step 1: Load Calculation

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

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

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

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

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

seismic zone- zone = 3

Response factor = 3

Zone factor = 0.16

Importance factor = 1

Factored Moment (M_{ux}) = $\alpha_x \times W_u \times l_x^2$

$= 0.0672 \times 18.187 \times 3^2$

$= 10.9$ KN-m

Factored Moment (M_{uy}) = $\alpha_x \times W_u \times l_y^2$

$= 0.056 \times 18.187 \times 3^2 = 9.166$ kN-m

Step 2: Check for depth

$M_{u\ limit} = 0.133$ fck bd^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

$\frac{M}{bd2} = \frac{10.9 \times 106}{1000 \text{ x}1502} = 0.4 \text{ mm}^2$

sp16 table 4 pt = 0.344

Ast = $\frac{\text{pt bd}}{100} = \frac{0.344 \times 1000 \times 150}{100}$ =516 mm2

Diameter of bar = 10 mm

Area of bar =$\frac{\Pi}{4} \times 10^2$ = 80 mm2

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

Use 10 mm diameter bar @ 150 mm c/c

LONGER DIRECTION:

d = 150 – 10 = 140 mm

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

from sp16 table 4 pt = 0.342

Ast = $\frac{0.342 \times 1000 \times 140}{100}$ = 479 mm2

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

use 10mm diameter bar @160 mm c/c

2) Design Of Column (1 TO 8 FLOOR)

L = 3 m

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

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

Dimension of column = 500 × 500

Pu (factored load) = 3000 KN

Step 1: Calculation of Minimum Eccentricity

Emin = $\frac{L}{500} + \frac{D}{30} = \frac{3000}{500} + \frac{500}{30}$ = 22.66 m

Emax = 0.05 × D = 0.05 × 500 = 25 m

Emin < Emax

Step 2: Check for short or long column

$\frac{Lex}{D} = \frac{3000}{500} = 6$

$\frac{Ley}{D} = \frac{3000}{500} = 6$

$\frac{Lex}{D}$ and $\frac{Ley}{D} < 12$

∴ The column is short column.

Step 3: Reinforcement calculation

Pu = 0.4×fck×Ac+0.67fy×Asc

Ac = Ag - Asc

Pu = 0.4×fck×(Ag – Asc) + 0.67×fy×Asc

Pu = 0.4 × fck × Ag + (0.67 fy - 0.4 fck) Asc

Asc = 3715.17 mm2

assuming 16 mm diameter bars

Ast = $\frac{\Pi}{4} \times 16^2 = 200 \text{ mm}^2$

No. of rods = Asc × $\frac{Asc}{Ast} = \frac{3715.17}{200}$ = 19

∴ Take 20 no. of rods

Asc provided = 20 × 200 = 4000

Step 4: Design of Lateral ties

1. Tie diameter = $\frac{1}{4}$ × diameter of bar = 4 mm
2. ≤ 16 mm

Provide 8 mm diameter ties.
Ties spacings: -
Provide minimum of below three conditions.
(a) Size of column i.e, 500 mm
(b) 16 × longitudinal diameter = 16 ×16 =256 mm^2
(c) 300 mm

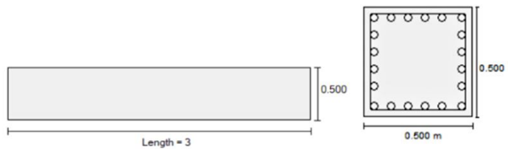

Figure 6: Design of Column (1-4 floor)

3) Design Of Column (9 to 16 floor)
L = 3 m
Yield strength =600 $\frac{N}{mm^2}$
Compressive strength = 40 $\frac{N}{mm^2}$
Dimension of column = 400 × 400
Pu (factored load) = 3000 kN
Step 1: Calculation of Minimum Eccentricity
Emin = $\frac{L}{500} + \frac{D}{30} = \frac{3000}{500} + \frac{400}{30}$ = 19.33 m
Emax = 0.05 × D = 0.05 × 400 = 20 m
Emin < Emax
Step 2: Check for short or long column
$\frac{Lex}{D} = \frac{3000}{400}$ = 7.5
$\frac{Ley}{D} = \frac{3000}{400}$ = 7.5
$\frac{Lex}{D}$ and $\frac{Ley}{D}$ < 12
∴ The column is short column.
Step 3: Reinforcement calculation
Pu = 0.4×fck×Ac+0.67fy×Asc
Ac = Ag - Asc
Pu = 0.4 × fck (Ag - Asc) + 0.67 × fy × Asc
Pu = 0.4 × fck × Ag + (0.67 fy - 0.4 fck) Asc
Asc = 1322.47 mm2
assuming 12 mm diameter bars
Ast =$\frac{\Pi}{4} \times 12^2$ = 114 mm^2
No. of rods = Asc×$\frac{Asc}{Ast} = \frac{1300.4}{114}$ = 11.6
∴ Take 12 no. of rods
Asc provided = 12 × 114 = 1368
Step 4: Lateral ties design

1. Tie diameter = $\frac{1}{4}$ × diameter of bar = 3 mm
2. ≤ 16 mm

Provide 8 mm diameter tie.
Ties spacings: -

Provide minimum of three condition.
(a) Size of column i.e, 400 mm
(b) 16 × longitudinal diameter = 16 ×12 =192 mm^2
(c) 200 mm

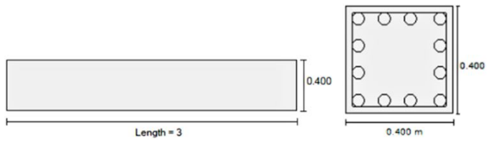

Figure 7: Design of Column (5-8 floor)

4) Design Of Beam (1 -8 Floor)

Beam design for 3 m span
Effective length of beam = 3 m
Width of beam b = 500 mm
Grade of concrete M40 = 40 $\frac{N}{mm^2}$
Grade of steel = 600 $\frac{N}{mm^2}$
Step 1: Cross Sectional Area
Assume span depth ratio as 15
Effective depth d = $\frac{span}{15} = \frac{3000}{15}$ =200 mm
Adopt d = 450mm, clear cover as 50 mm
Total depth D = 500 mm
Step 2: Load Calculation
Self weight of beam = 0.23 ×0.5 ×25 = 2.875 kN m2
D.L. = 25.8 $\frac{kN}{m^2}$
Total dead load = 28.8 $\frac{kN}{m^2}$
Live load = 12 $\frac{kN}{m^2}$
Total load = 40.8 $\frac{kN}{m^2}$
Factored load = 1.5 × 40.8 = 61.2 kN-m
Ultimate bending moment and shear force
Mu = $\frac{Wl^2}{8} = \frac{61.2 \times 3\ 2}{8}$ = 68.85 kN-m
Vu = $\frac{Wl}{2} = \frac{61.2 \times 3}{2}$ = 96.3 kN
Step 3: Limiting Moment Resistance
Mu limit = 0.133 fck bd^2
= 0.133 × 40 ×500 ×500^2
= 665 kN m
Mu < Mu_{lim} (section is underreinforced)
Step 4: Main Reinforcement
Mu = (0.87 × fy × Ast d) [1 $-\frac{Ast\ fy}{bd\ fck}$]
68.85 ×10^6 = 1744000Ast – 12.16 Ast^2
Ast =421 mm^2
Provide 6 bars of 12mm diameter bar Ast = 6 × $\frac{\Pi}{4}$ ×12^2 = 678.24 mm2
Step 5: Shear Reinforcements
$\tau v = \frac{Vu}{bd} = \frac{96.3 \times 10^3}{500 \times 500}$ = 0.385

pt = $\frac{100 \text{ Ast}}{\text{bd}} = \frac{100\times678.24}{500\times500}$ = 0.271
from table 19 of IS 456 τc= 0.5612
Since τc < τv, shear reinforcement is to to resist the balance shear computed below
Vus = Vu - (τc × bd)
Vus = 96.3 × 10^3 – (0.56 × 500 × 500) = 43.7 kN
Using 8 mm dia bar @ 2 legged stirrups
Sv = $\frac{0.877\text{Fy} \times \text{Asvd}}{\text{Vus}}$
Sv = 191
Sv ≯ 0.75 × d
= 0.75 × 400 = 300
Sv ≯ 300
Step 6: Check for Deflection
Pt = 0.22, kt = 1.2, Kc= kf = 1
$(\frac{L}{d})$max = $(\frac{L}{d})$ basic × kt × kc × kf
= $(\frac{3000}{400})$ × 1.2×1×1= 9
$(\frac{L}{d})$actual= $\frac{3000}{400}$ = 7.5 < 9
Hence deflection control is satisfied.

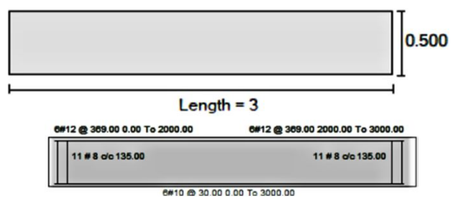

Figure 10: Design of Beam (1-8 floor)

5) Design Of Beam (9 TO 16 Floor)

Beam design for 3 m span
Effective length of beam = 3 m
Width of beam b = 400mm
Grade of concrete M40 = 40 $\frac{N}{mm^2}$
Grade of steel = 600 $\frac{N}{mm^2}$
Step 1: Cross Sectional Area
Assume span depth ratio as 15
Effective depth d = $\frac{\text{span}}{15} = \frac{3000}{15}$ =200 mm
Adopt d = 450mm, clear cover as 50 mm
Total depth D = 500mm
Step 2: Load Calculation
Self-weight of beam = 0.23 ×0.5 ×25 = 2.875 kN m^2
D.L. = 25.8 $\frac{kN}{m^2}$
Total dead load = 28.8 $\frac{kN}{m^2}$
Live load = 12 $\frac{kN}{m^2}$
Total Load = 40.8 $\frac{kN}{m^2}$
Factored load calculation = 1.5×40.8 = 61.2 KN m
Ultimate bending moment and shear force

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

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

Step-3: - Limiting Moment Resistance

Mu limit = 0.133 fck bd^2

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

$Mu < Mu_{lim}$ (underreinforced)

Step 4: Main Reinforcement

$Mu = (0.87 \times fy \times Ast \times d)[1 - \frac{Ast\ fy}{bd\ fck}]$

$68.85 \times 10^6 = 1744000Ast - 12.16\ Ast^2$

Ast= 421 mm^2

Provide 6 bars of 10mm diameter bar $Ast = 6 \times \frac{\Pi}{4} \times 10^2 = 471mm^2$

Step5: Shear Reinforcements

$\tau v = \frac{Vu}{bd} = \frac{96.3 \times 103}{400 \times 400} = 0.601$

$pt = \frac{100\ Ast}{bd} = \frac{100 \times 471}{400 \times 400} = 0.348$

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

Since $\tau c < \tau v$, the balance shear computed as

$Vus = Vu - (\tau_c \times bd)$

$Vus = 96.3 \times 10^3 - (0.56 \times 400 \times 400) = 67$ kN

Using 8 mm dia bar @ 2 legged stirrups

$Sv = \frac{0.877Fy \times Asvd}{Vus}$

Sv = 191

$Sv \ngtr 0.75 \times d$

$= 0.75 \times 400 = 300$

$Sv \ngtr 300$

Step-6: - Check for Deflection

Pt = 0.22, kt = 1.2, Kc= kf = 1

$(\frac{L}{d})max = (\frac{L}{d})basic \times kt \times kc \times kf$

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

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

Hence deflection control is satisfied.

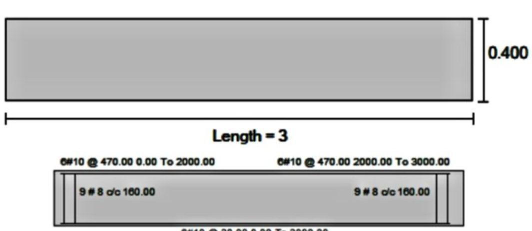

Figure 11: Design of Beam (9-16 floor)

6) Design Of Skywalk

Step: 1

$\frac{ly}{lx} = \frac{9}{6} = 1.5 < 2$

Hence, Two-way slab

Step 2: Depth of slab

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

d ≈220 mm

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 mm

Step 3: Effective span

Effective span = Clear span + Effective depth

=6 + 0.220

=6.22m

Step 4: Calculation of load

Self wt. of the slab = 0.245×25

=6.125$\frac{kn}{m2}$

L.L. on slab = 4 $\frac{kn}{m2}$

Floor finish = 0.6 $\frac{kn}{m}$

Total working load = 10.725 $\frac{kn}{m2}$

Ultimate load = 1.5 × 10.725

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

Step 5: Moment and shear force

Mx = ∝x × wl^2

My = ∝y × wl^2

∝x = 0.0885 , ∝y = 0.057

Mux = 0.0885 × 16.08 × 6.220^2 =55.05Kn.M

Muy = 0.059 × 16.08 × 6.220^2 = 36.70Kn.M

Vux = $\frac{\text{Wx × lx}}{2}$ = 50 Kn

Step 6: Check for depth

Mu_{lim} = 0.138×fck×bd^2

d = $\sqrt{\frac{55.05 \times 106}{0.138 \times 20 \times 1000}}$ = 141.22 mm < 220 mm

Hence the effective depth is sufficient

Ast(min) = 12%

= $\frac{0.12}{100}$ × 1000 × 245

= 294 mm^2

Step 7: Reinforcement

Mu = 0.87 × fy × Ast×d [1 - $\sqrt{\frac{Ast \times 415}{bd \times fck}}$]

= 450 mm^2

Step 8: Spacing

S=$\frac{1000 \times ast}{Ast}$

= 174 mm

Adapt 10mm bar @ 180 mm c/c

Use 10 mm bar in the long spam

Effecive depth = 220 - 10 =210 mm

Mu = 0.87×fy×Ast×d [1 - $\sqrt{\frac{Ast \times 415}{bd \times fck}}$]

Ast=315 mm^2

Spacing = $\frac{1000 \times ast}{Ast}$

= $\frac{1000 \times 0.785 \times 102}{315}$= 249 mm

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)

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

$Pt = \frac{100\, Ast}{bd} = \frac{100 \times 0.5 \times 450}{1000 \times 220} = 0.102$

Permissible shear stress in slab is computed

as = Kτc for the value of K refer 40.2.1.1 for 245

x = 0.08

τc = 0.327

$K\tau c = 0.402 > \tau_v$

Hence the slab is safe

Step 10: Check for deflection control

$(\frac{L}{d})max = (\frac{L}{d})bacic \times kt \times kc \times kf$

kt= 1.4

kc= 1

kf= 1

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

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

Hence the deflection coltrol is satisfied.

Step 11: Torsion reinforcement (corners)

Area of the reinforcement in each layer

0.75 × 415 = 307.5mm^2

Divide = $\frac{Shoretr\ span}{5} = \frac{6000}{5} =$ 1200

Spacing = $\frac{100\, ast}{Ast} = \frac{100 \times 0.785 \times 62}{307.5\, mm2}$ = 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

Ast = 0.12% of crosssection area

$\frac{0.12}{100} \times 100 \times 170$ = 204 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

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

Total load = 1000 KN

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

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

Adopt 2.4×2.4 m (square footing)

$Q_u = \frac{1000}{(2.4 \times 2.4)} \times 1.5 = 260 \frac{kn}{m2}$

Step 2: One-way shear

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

Factoral shear V_u=0.26×2400x(1000-d)

=162× (1000-d)

Assuming reinforcement in the footing

Pt%= 0.25% fy M20 concrete

Permissible shear stress from table no. 19 IS5456 2000

$\tau c = 0.36 \frac{N}{mm2}$

One way shear resistance Vc=0.36× 2400 × d

= 864d

624(1000-d) = 864d

d= 722 mm ≈ 725 mm

Step 3: Two-way shear

Assuming d=722mm

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

$Vu = 0.26 [\ 2400^2 - (400+d)^2]$

$= 0.26 [\ 2400^2 - (400+725)^2]$

= 1170290 N

Two way shear

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

where Ks = 1.0 and τ_c =0.25 =$1.118 \frac{N}{mm2}$

Vc = 1×1.118[4(400+d)d]

$=1788.8d + 4.722d^2$

= d = 349 mm

Hence, one way shear is critical.

Adapting effective depth 'd' = 725 mm

And overall depth = 800 mm.

Step 4: Reinforcement Designs

Ultimate moment Mu=260×1×0.5=130 KNm

$\frac{Mu}{bd2} = \frac{130 \times 106}{1000 \times 725^2} = 0.247$

$Ast = \frac{Pt\ d}{1000} = \frac{0.25 \times 1000 \times 725}{1000}$ = 1813 mm c/c

Use 20mmØ

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

Use 20 mmØ @170 mm c/c

Step 5: Transfer of force at column face

Ultimate load on the column base = 1.5×1000 = 1500 KN

$$Fbr = 0.45Fck \sqrt{\frac{A1}{A2}}$$

1. <u>Column Face</u>

   $Fck=20\ N/mm^2$

$A1 = A2 = 400mm^2$

$=0.45 \times 20 \times 1 = 9\ N/mm^2$

2. <u>At footing face</u>

Fck = 20 N/mm2

A1=2400 mm2

A2= 4002

Fbr(footing) =0.45×20×2 =18 $\frac{N}{mm2}$

Hence govern by column face

Fbr = 9 N/mm

Check for stress = fbr =$\frac{9\times400^2}{1000}$=14000 Kn<1500 Kn

Hence Safe

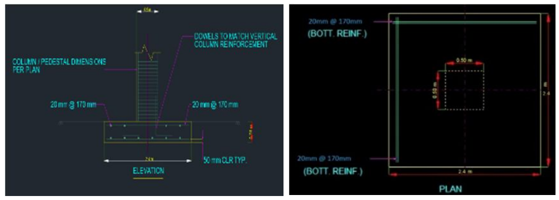

Figure 4.15: Design of foundation

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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