COMPARISON BETWEEN MANUAL AND SOFTWARE DESIGN OF A BUILDING

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Abstract – *STAAD.PRO is a structural analysis and design software application originally developed by Research Engineers International in 1997. In late 2005, Research Engineers International was bought by Bentley Systems. It is one of the most widely used structural structural analysis design software product in mainstream civil industry. It helps structural engineers to automate their tasks by eliminating the complex and tedious procedures which they have to undergo while designing manually and allows to perform designing and analysis virtually of any type of structure.*

Key Words: Structural Analysis and Designing Program (STAAD.PRO)

1. INTRODUCTION

In this respective project we are going to design a (G+3) commercial building for bending moments, shear forces, deflections and reinforced details with the help of STAAD.PRO. As it provides us with fast, efficient and an accurate platform to perform the actions required for analysis and designing of complex structures.

1.1 PROBLEM STATEMENT

It is necessary from the end user to check the model and perform analysis and design action manually side by side to overcome and to spot the errors in the outcome. In the software design, while designing the structure the beams and columns are denoted as beams only. Bill of Quantity (BOQ) of a single element cannot be found out as it gives the BOQ of all the elements combined of a structure.

1.2 OBJECTIVES

1. Comparing the results of both STAAD.PRO and Manual design.
2. Detailing in STAAD.PRO
3. Cost (BOQ) comparison of components between STAAD.PRO and manual design.
4. Designing of structure manually by referring IS 456:2000.

1.3 SCOPE OF PROJECT WORK

1. As STAAD.PRO gives the estimate quantity of a particular structure after the design to reduce manual calculations. Therefore, this project can be further used to determine the economic structure by comparing the quantity.
2. STAAD.PRO is an advance software which provides with an efficient and accurate platform for designing the structure.
3. Through STAAD.PRO, we will come to know the cost difference between manual and software design.

1.4 EXPECTED OUTCOMES

After referring the journals, it is observed that difference between manual and software design varies with each other. But for safety standards, the highest value of loads among both the design should be considered. Time consumption in software designing is comparatively lesser than manual designing using all the IS code standards.

2. PROCEDURE

At the first stage of the project, plan layout and elevation of the building has been finalized.

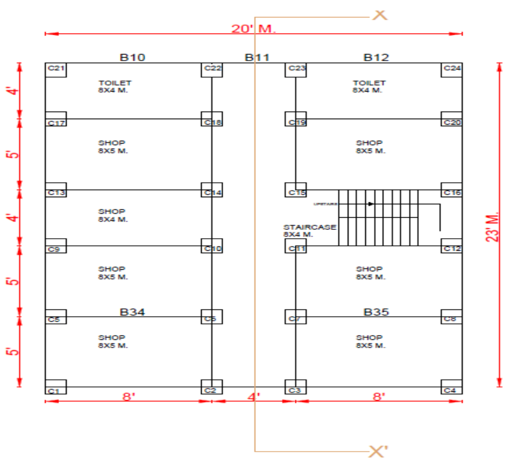


Fig -1: Plan layout

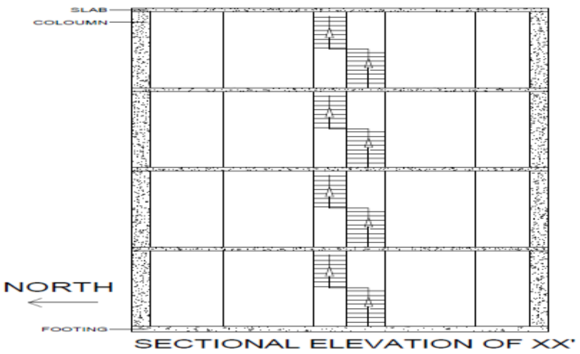


Fig -2: Elevation

2.1 MANUAL AND SOFTWARE DESIGN

MANUAL DESIGN
FOOTING OF COLUMN NO. 21

Column size = $300 \times 750\ mm^2$

STEP-1 (SIZE OF FOOTING)

Factored load on column = 1755 KN

Self weight of footing is 10% of load on column = 175.5 KN

Total factored load = 1930.5 KN

Assume self bearing capacity of soil = $200\ \mathrm{KN/m^2}$

Ultimate bearing capacity of the soil

$= 2 \times 200 = 400\ \mathrm{KN/m^2}$

Area of footing = $\frac{1930}{400} = 4.82\ \mathrm{m^2}$

For square shape, each side = $\sqrt{4.82}$ = 2.19
$\approx 3.5m$

Provide 3.9×3.9 footing size.

STEP-2 (UPWARD SOIL PRESSURE)

Upward soil pressure

$$= \frac{Factored\ load}{size\ of\ footing} = \frac{1930.5}{3.9 \times 3.9} = 126.92\ KN/m^2$$

STEP-3 (DEPTH OF FOOTING)

$\tau_v = K_s \tau_c$, where $K_s = 0.5\ \beta_c$

$\frac{b}{D} = \beta_c = \frac{300}{750} = 0.4$

I.e. K_s = 0.4+0.5 = 0.9

$\tau_c = 0.25\sqrt{f_{ck}} = 1.25$

$\tau_v = K_s \tau_c = 0.9 \times 1.25 = 1.125$

CASE-1 (DEPTH OF FOOTING FOR ONE WAY SLAB)

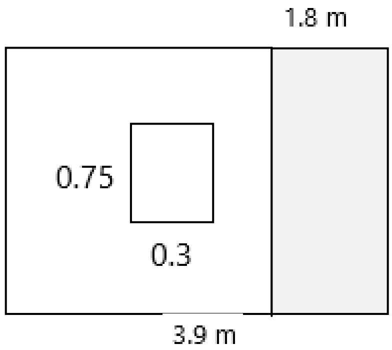


Fig -3: Depth of footing

$V_u = \text{uplift pressure} \times \text{shaded area}$

$V_u = 126.92 \times \left[\frac{3.9-0.3}{2} - d\right] \times 3.9$

$= 494.98[1.8 - d] \longrightarrow$ [1]

$V_u = \tau_c . B . d$

$= 1250 \times 3.9 \times d \longrightarrow$ [2]

d = 165 mm $\longrightarrow$ by equating [1] & [2]

CASE-2 (DEPTH OF FOOTING FOR TWO WAY SHEAR)

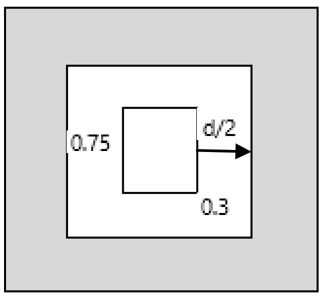


Fig -4: Depth of footing

Critical section at ${}^{d}/_{2}$

$V_u = p[B^2 - (b + d^2)]$
$V_u = 126.92[3.9^2 - (0.3 + d) \times (0.75 + d)] \longrightarrow$ [1]

Shear force resisted by concrete,

$V_u = \tau_c b' d$

$b' = 2 \times [0.3 + d + 0.75 + d]$

$$V_u = 1250 \times 3.9 \times d \times [2.1 + 4d] \longrightarrow [2]$$

By equating [1] and [2],

$$d = 396.68 \approx 400\ mm$$

Check for depth for bending moment

$$M_x = P\left[\frac{(B-b)^2}{8}\right] = \frac{[3.9-0.3]^2 \times 126.92}{8} = 205.61\ KN.m \rightarrow [1]$$

$$M_y = \frac{[3.9-0.75]^2 \times 126.92}{8} = 157.42\ KN.m$$

$$M_d = 0.138\ f_{ck}\, bd^2$$

$$M_d = 0.138 \times 25 \times 1000 d^2 \longrightarrow [2]$$

Equate [1] & [2] as M_x is maximum

$$d_{required} = 244.12\ mm$$

By considering above 3 conditions;

$$d_{provided} = 400\ mm \quad \therefore \text{OK}$$

Assume cover 50 mm and bar 20 mm.

$$D = 400 + 50 + \frac{20}{2} = 460 \approx 500\ mm$$

Provide depth of 500 mm.

STEP-4 (REINFORCEMENT PER METER)

$$A_{st} = \frac{0.5 \times 25}{415} \times \left[1 - \sqrt{1 - \frac{4.6 \times 205.61 \times 10^6}{25 \times 3900 \times 400^2}}\right] \times 3900 \times 400$$

$$A_{st} = 1446.67\ mm^2$$

$$Provide\ 14\ mm\ \emptyset\ bar\ / a_{st} = 153.93\ mm^2$$

$$Number\ of\ bars = \frac{1446.67}{153.93} = 9.39 \approx 10\ bars$$

Provide 10 number of 14 mm diameter bars in both the directions.

<u>Footing column- 21</u>

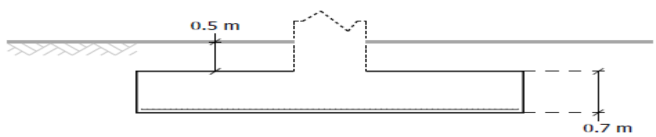


Fig -5: Elevation

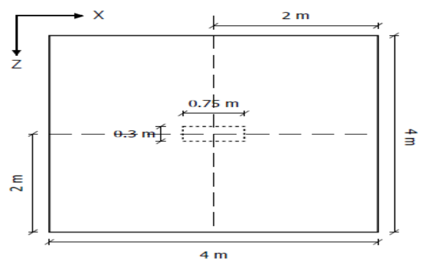


Fig -6: Plan

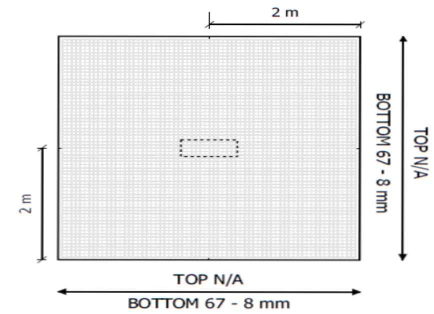


Fig -7: Plan

COLUMN DESIGN NO– 22

MANUAL CALCULATIONS

Floor to floor height = 3m

Height of plinth above ground level = 1m

Depth of foundation below ground level = 1.5m

Live load on slab = 2 KN/m^2

Floor finish = 0.5 KN/m^2

Thickness of slab = 170 mm

Thickness of wall = 300 mm

Size of beam = 300mm × 500mm

Material: M25 and Fe 415

1. LOAD ON SLAB

Dead load of slab = 25×0.17 = 4.25 KN/sq.m

F.F = 0.5 KN/sq.m

W.P. Load = 1 KN/sq.m

L.L = 1 KN/sq.m

Total load on roof slab = 6.75 KN/sq.m

2. FIRST, SECOND & THIRD FLOOR SLAB

Dead load of slab = 25×0.17= 4.25 KN/sq.m

F.F = 0.5 KN/sq.m

L.L = 2 KN/sq.m

Total load on roof slab = 6.75 KN/sq.m

STEP-2 (LOAD FROM SLAB TO COLUMN)

1. LOAD ON COLUMN FROM ROOF SLAB.

P = Intensity of load × shaded area of floor

P = 6.75 × 2 × (4+2) = 81 KN

2. LOAD ON COLUMN FROM GROUND, FIRST, SECOND, THIRD FLOOR SLABS.

P = Intensity of load × shaded area of floor

P = 6.75 × 12 = 81 KN.

Total load from slab to column= 81×5=405 KN.

STEP-3 (BEAM LOAD TO COLUMN)

Wall load = $\gamma_w \times b_w \times$ [illegible]

Wall load = 19 × 0.3 × 3 = 17.1 KN

Self weight of beam = γ[illegible]$_c \times b \times d$

Self weight of beam = 25 × 0.3 × 0.5 = 3.75 KN

BEAM LOADS TO COLUMNS:

Roof beam load to columns = 3.75 × (2+4+2)

Roof beam load to columns = 30 KN.

Third floor beam load to column

= (3.75+17.1) × 8 = 166.8 KN

Second floor beam load to column

= (3.75+17.1) × 8 = 166.8 KN

First floor beam load to columns = 166.8 KN

Ground floor beam load to columns = 166.8 KN

Plinth beam load to columns = 3.75 × 8 = 30 KN

Total wall load including self weight of beams

= 30 + (4 × 166.8) + 30 = 727.2 KN

STEP-4 (WEIGHT OF COLUMN)

Assume column size = 0.3m × 0.5m

For roof floor, self weight = 0.3 × 0.5 × 1 × 25

= 3.75 KN

Third floor, second floor, first floor, ground floor, self weight of columns each floor = 0.3 × 0.5 × 3 × 25

= 11.25 KN

From ground to plinth, self weight = 0.3 × 0.5 × 1 × 25

= 3.75 KN

From ground to footing, self weight= 0.3 × 0.5 × 1.5 × 25

= 5.625 KN

Total column load = 3.75+ (11.25×4)+3.75+5.625

=58.125 KN

STEP-5 (TOTAL LOAD ON GROUND FLOOR COLUMN)

P = Slab load + beam load including wall load + column load.

P = 405 + 727.2 + 58.125

$P = 1190.325 \approx 1190\ KN.$

Design load = P + 10% of P for accidental increase in load = 1190+119 = 1309 KN

Design load ≈ 1310 KN

STEP-6 (DESIGN OF GROUND FLOOR COLUMN)

P = 1310 KN

Factored design load (P_u) = 1.5 × 1310 = 1965 KN

$= 1965 \approx 1960\ KN$

Assume 2.5 % of steel,

∴ Area of steel (A_{sc}) = 0.025 A_g

Area of concrete $A_c = A_g - A_{sc}$

∴ $A_c = A_g - 0.025\,A_g = 0.975\,A_g$

$P_u = (0.4 f_{ck} A_c) + (0.67 f_y A_{sc})$

$1960 \times 10^3 = (0.4 \times 25 \times 0.975 A_g) + (0.67 \times 415 \times 0.025 A_g)$

$A_g = 117356.4852\ mm^2$

(Assuming rectangular column of width = 300 mm)

$$Depth = \frac{A_g}{300} = \frac{117356.4852}{300} = 391.18\ mm$$

Depth = 391.18 mm < 500 mm ($\therefore ok$)

Size of column = 300mm × 500mm

Area of steel = 0.025 A_g

A_{sc} = 0.025 × 117356.4852

A_{sc} = 2933.912 $mm^2 \approx 2930\ KN$

$$Number\ of\ steel\ bars = \frac{2930}{\frac{\pi}{4} \times 25^2} = 5.97 \approx 6\ bars$$

Provide 6 bars of 25 mm ∅ giving area of steel (2930mm^2)

Check for minimum eccentricity

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

e_{min} = 22.66 mm

0.05 D = 0.05 × 500 = 25 mm

22.66 < 25 mm ($\therefore ok\ for\ minimum\ eccentricity$)

DESIGN OF LATERAL TIES:

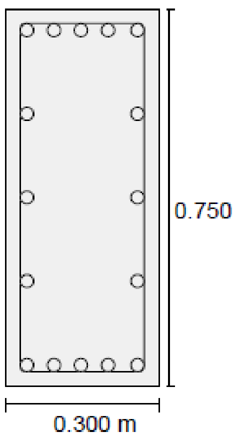
Assume diameter of link = 8 mm

1. Least lateral dimension – 300 mm
2. 16 × diameter of longitudinal steel (16×25) = 400 mm
3. 300 mm ∴ provide pitch = 300 mm ≈ 200 mm

SUMMARY:

1. Column size = 300 mm × 500 mm
2. Longitudinal steel 6 bars of 25 mm ∅ ; lateral ties 8 mm ∅ @ 200 mm c/c.

SOFTWARE DESIGN



Design Load

Load	1
Location	End 1
Pu(Kns)	-144.679993
Mz(Kns-Mt)	101.540001
My(Kns-Mt)	4.330000

Design Results

Fy(Mpa)	415
Fc(Mpa)	25
As Reqd(mm²)	1800.000000
As (%)	0.804000
Bar Size	12
Bar No	16

Fig -8: COLUMN DESIGN NO.-22

BEAM – 34

Length = 8000 mm

Width = 300 mm

Depth = 500 mm

Total depth = 550 mm

Cover = 40 mm

Bar diameter = 25 mm

Fy = 415

Total load = 29.34 KN/m

$$B.M = \frac{Wl^2}{8} = \frac{29.34 \times 8^2}{8} = 234.72\ KN.m$$

Mu lim = 0.138×300×500²×25 = 258.75 KN.m

Mu lim > Mu (∴ Design beam as single R/F beam)

$$\frac{0.5 \times f_{ck}}{f_y} \times \left[1 - \sqrt{1 - \frac{M_u \times 4.6}{f_{ck}.bd^2}}\right] \times bd$$

$$= \frac{0.5 \times 25}{415} \times \left[1 - \sqrt{1 - \frac{234.72 \times 10^6 \times 4.6}{25 \times 1000 \times 500^2}}\right] \times 1000 \times 500$$

= 1362.48 mm²

Number of bars = $\frac{1362.48}{314.15}$ $[a_{st} = 314.15;\ \emptyset = 20\ mm]$

= 4.33≈ 5

Provide 5 no. of 20 mm diameter bars.

Ast provided = 5×314.15 = 1570.75 mm².

SHEAR CHECK

$$V_u = \frac{wl}{2} = \frac{29.34 \times 8}{2} = 117.36\ KN.$$

$$\tau_v = \frac{V_u}{bd} = \frac{117.36 \times 10^3}{300 \times 500} = 0.78\ N/mm^2$$

$$P_t = \frac{A_{st} \times 100}{bd} = \frac{1362.48 \times 100}{300 \times 500} = 0.9\%$$

From table – 19; IS 456-2000

∴ τ_c = 0.64

$\tau_v > \tau_c \cdots$ Shear bars required.

Vus = Vu – τc bd =213.36 KN.

Let provide 8mm #stirrups -2 legged

Asv =2 $\times \frac{\pi}{4} \times 8^2$ = 100.53 mm^2

$$Spacing\ S_v = \frac{0.87 \times f_y \times A_{sv} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times 100.53 \times 500}{213.36 \times 10^3}$$

$$= 85.05 \approx 90mm$$

Provide 8 mm diameter 2 legged stirrups @ c/c 90 mm.

Provided Reinforcement

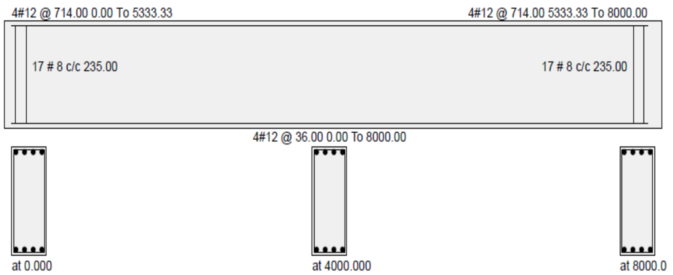
Provide 5 numbers of 20 mm diameter bars at supports.

Provide 3 numbers of 20 mm diameter bars at mid span.

$$A_{st}\ after\ curtailment = 3 \times 20^2 \times \frac{\pi}{4}$$

$$= 942.47\ mm^2$$

SOFTWARE DESIGN OF BEAM



Design Load

Mz(Kn Met)	Dist.et	Load
71.050003	4.000000	4
-79.559998	0.000000	4
-89.470001	8.000000	2

Design Parameter

Fy(Mpa)	415.000000
Fc(Mpa)	25.000000
Depth(m)	0.750000
Width(m)	0.300000
Length(m)	8.000000

Fig -9: Beam design no. 34

<u>SLAB: S1</u>

STEP-1 (DEPTH OF SLAB)

Longer span= 8m.

Shorter span= 5m.

$$\frac{ly}{lx} = \frac{8}{5} = 1.6 \therefore (TWO\ WAY\ SLAB)$$

Since it is a continuous slab, l/d = 26xM.F

Fs = (0.58 fy) = (0.58 x 415) = 215 N/mm2

Assume Pt = 0.4%

∴ M.F = 1.3 (From Fig.4 IS 456:2000)

I.e. (5000/d) = 26 x 1.3

d= 147.92 mm ≈ 150mm.

Assuming effective cover 20 mm

D = (150+20) = 170 mm.

Effective span (lxe) = (lx+d) = (5000+170) =5170.

STEP-2 (LOAD CALCULATION)

Self weight = (0.170 x 25 x 1) = 4.25 KN/m

Live load = 4 KN

Floor Finish = 1 KN

Total Factored load= (9.25 x 1.5) = 13.875 KN/m

STEP-3 (DESIGN MOMENT)

$M_x = \alpha x W l x^2$, $M_y = \alpha y W l y^2$

At mid span- α_x= 0.063, α_y= 0.035

At supports- α_x= 0.084, α_y= 0.047

STEP-4 (REINFORCMENT)

ALONG SHORTER SPAN

Width of the middle strip= ¾ ly= ¾ x 8= 6m.

$$M_{ux} = 23.86\ KN.m$$

$$Ast = 0.5\, fck \left[1 - \sqrt{\frac{1 - Mux4.6}{2afck \times bd^2}}\right] \times bd = 454.40mm^2$$

Let bar diameter be 8mm,

∴ ast= 50.24 mm^2

Spacing= $\frac{50.24}{454.40} \times 1000 = 110.56 \approx 110$

∴ Providing 8mm bar diameter with 1100 mm c/c distance at mid span of 6m.

ALONG LONGER SPAN

Width of the middle strip = $\frac{3}{4}$ lx = $\frac{3}{4}$×5 = 3.75

M_{UY}= 12.98 KN/m.

$$\therefore Ast=0.5\times\frac{25}{415}\left[1-\sqrt{\frac{1-4.6\times12.98\times10^6}{25\times1000\times190^2}}\right]\times1000\times150$$

∴Ast= 246.1422

Providing 6mm diameter bar.

Spacing $\frac{28.26}{216.1422}\times 1000 = 130.75 \approx 120mm$

Providing 6mm diameter bars with 120mm c/c distance at mid span of 3.75m.

STEP-5 (DISTRIBUTION OF STEEL)

Reinforcement for edge strip along shorter span and longer span.

Provide Pt.min = 0.12%

∴ Ast= $\frac{0.12}{100}\times 1000\times 170 = 120mm^2$

Spacing= $c\times 1000 = 246.27 \approx 240mm$

∴ Provide 8mm dia. Bars at 240 c/c for edge strip along shorter and longer span.

Number of bars= $\frac{204}{50.24} = 4\ bars$.

STEP-6 (TORSION REINFORCMENT)

Size of mesh= $\frac{lx}{5} = \frac{5000}{5} = 1000mm$ (For both directions).

Required area of steel= 0.75 Ast = 340.8 mm^2

Provide 8mm diameter bars with 140 mm c/c distance.

STAAD MODELS

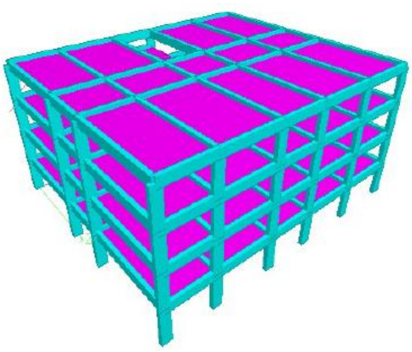


Fig –10: 3D Rendered view

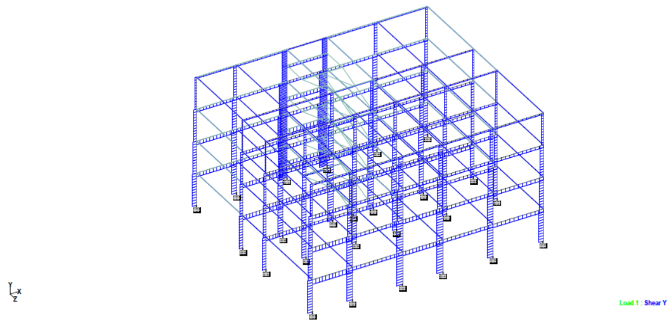


Fig -11: Shear Force Diagram

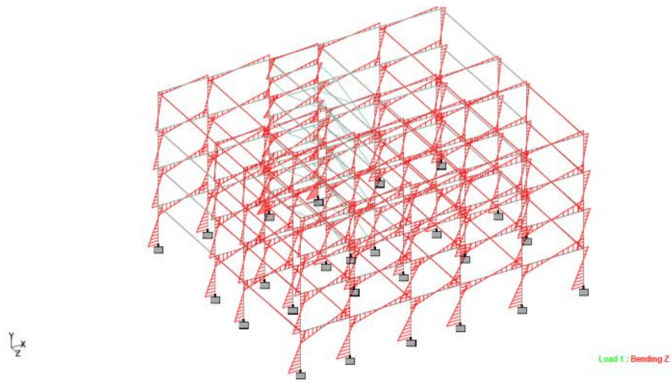


Fig -12: Bending Moment Diagram

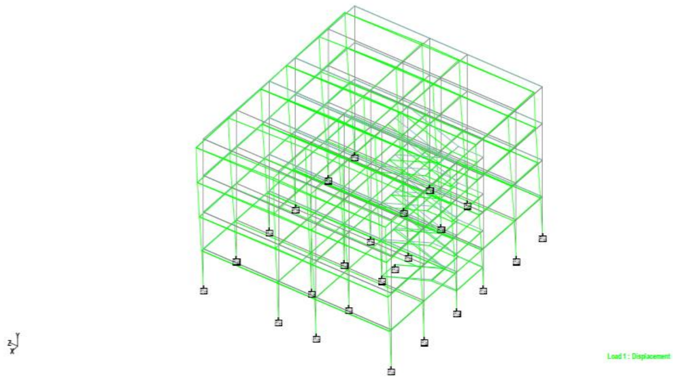


Fig -13: Deflection diagram

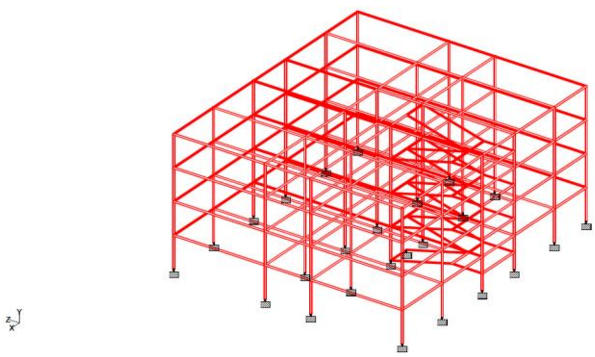


Fig -14: Dead load

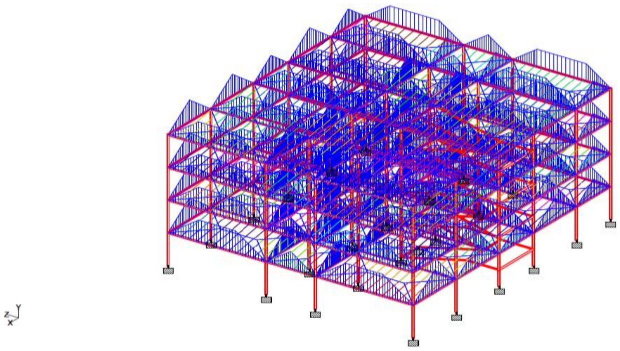


Fig -15: Live load

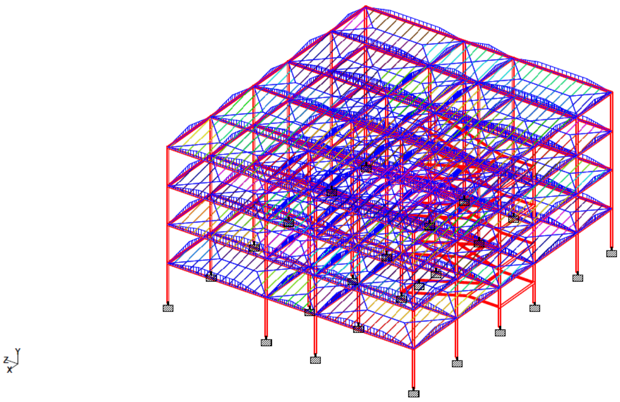


Fig -16: Floor load

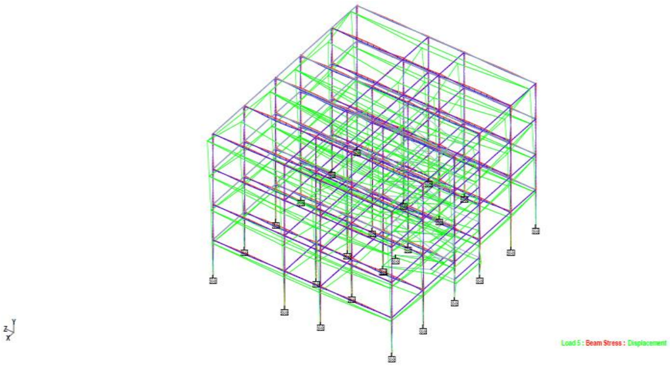


Fig -17: Beam stress

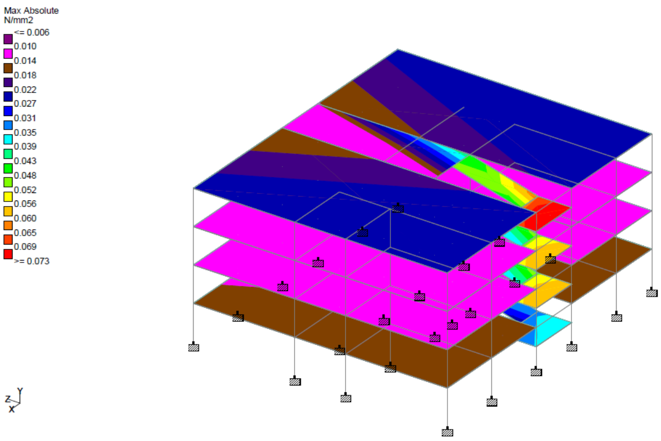


Fig -18: Plate stress

3. PROJECT RESULTS

Table -1: Comparing area of concrete and steel from both the outcomes. (M- Manual outcome, S- Software outcome)

COMPONENTS	A_{st} AREA OF STEEL (mm²)		F_{ck} AREA OF CONCRETE (mm²)	
	M	S	M	S
FOOTING	3078	6534	9	15.21
COLUMN	2930	1810	0.15	0.225
BEAM	942	905	0.15	0.225

Table -2: Cost comparison of both the outcomes.

Components	Total cost		% Difference
	M	S	
FOOTING	₹ 42,375	₹52,184	18.79
COLUMN	₹ 4,748	₹4,577	3.6
BEAM	₹ 9,063	₹9,766	7.19

4. CONCLUSIONS

1. The cost difference of footing of manual calculation and STAAD.PRO is 19 % and is more in STAAD.PRO.
2. The cost difference of column of manual calculation and STAAD.PRO is 3.6 % and is more in manual calculation.
3. The cost difference of beam of manual calculation and STAAD.PRO is 7.19 % and is more in STAAD.PRO.

REFERENCES

[1] A. P. Patil, A. A. Choudhary, P. A. Mudhole, V. V. Patole, A. D. Dange, S. K. Chendake "Design and Analysis of Multi Storeyed Building (G+10) by using Staad Pro V8i" IJAEMS, ISSN: 2454-1311 (March 2017)

[2] Dr. P. D. Hiwase, Aditi Joshi, Akash Keshariya "Comparison Between Manual and Software Approach towards design of structural Elements" IJES, ISSN (e): 2319-1813 ISSN (p): 23-19 – 1805, 2018

[3] Aju. Joy, Akhila A. B., Rosmary K. Eldhose, Sruthi K.B., Jisanto MDR "Comparison between Manual analysis and Software analysis of a shopping complex" IJARTET, ISSN 2394-3785 (March 2017)

[4] Kavya B.R., Rahul R., Sahitya C. N., Suchitha T. J., Vijay N. J. "ANALYSIS AND DESIGN OF RC BEAM UNDER DIFFERENT END CONDITIONS USING STAAD.PRO" IEJSE, E-ISSN NO: 2581-6195 (MAY-JUNE 2019)

[5] Prashanth P., Anshuman S., Pandey R. K., Arpan Herbert "Comparison of design results of a structure designed using Staad and Etabs spftware" IJCSE, ISSN 0976-4399 (February 2012)

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