Structural Engineering II (CE302) Project

Semester V
B. Tech
Civil Engineering

IIT Guwahati

Submitted by-

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Design and analysis of structural systems using STAAD Pro

1.1 Flow of Work

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1.2 Introduction

In every aspect of human civilization, the need of a civil engineer cannot be understated. Now, more than ever and especially in a growing country like India. In this day and age, multi storied buildings come up in every nook and cranny of the metropolitan cities. What has made this building of such complex structures so quickly possible is the use of computers to aid in the design process.

There are many classical methods to solve design problem, and with time new software's also coming into play. Here in this project work based on software named STAAD Pro has been used.

Two practical problems have been solved to show how STAAD Pro can be used in different scenarios. These problems have also been solved theoretically using the basic concept of loading, analysis, condition as per IS code and results are then compared.

1.3 Scope of work

Following points will be covered in project work

- Introduction of STAAD Pro
- Analysis of the structure in STAAD Pro and interpreting the results
- Using the results to calculate steel reinforcement as may be the case.

1.4 Aim of project

This project aims for relearning the concept of structural design with the help of computer aids. Briefly we have gone through the following points through out of the project work.

- Understanding of design and detailing concept.
- Main objective i.e. learning of STAAD Pro software package.
- Learning of analysis and design methodology which can be very useful in the field.
- Approach for professional practice in the field of structural engineering.

1.5 Introduction of Analysis and design

Analysis **:** Analysis of the structure means to determination of the internal forces like axial compression bending moment, shear force etc. in the component member for which the member are to be designed under the action of given external load.

Design: The design is process of section percussion from the analysis results by using suitable analysis method. The aim of design is to achievement of an acceptable probability that structures being designed will perform satisfactorily during their intended life.

2. Problem statement

- Structural design and analysis of air washer building after the construction of a new water tank on its roof using STAAD PRO

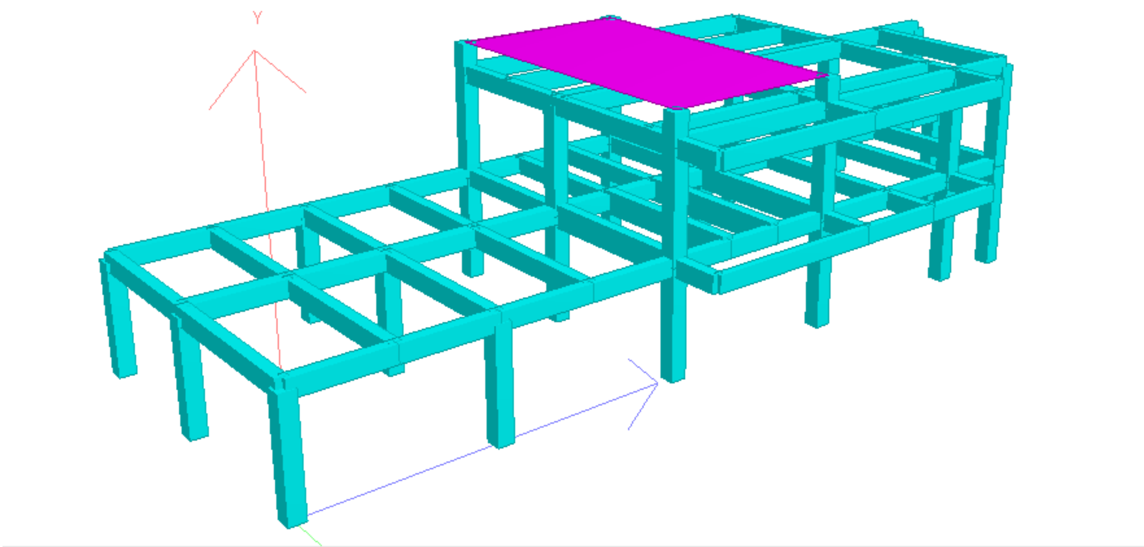
2.1 Introduction:

This report presents the analysis of air washer building in Gurgaon, Haryana. It was designed to meet both strength and serviceability requirements when subjected both to gravity loads and lateral loads.

An additional tank of 20,000 liters was added on the roof of the building. In order to overcome additional requirement of a water tank on roof of the building, STAAD Pro can be used to determine the structural adequacy of the columns and beams of the structure.

2.2 Air Washer Building

3D Rendering of the Air Washer building in STAAD Pro:



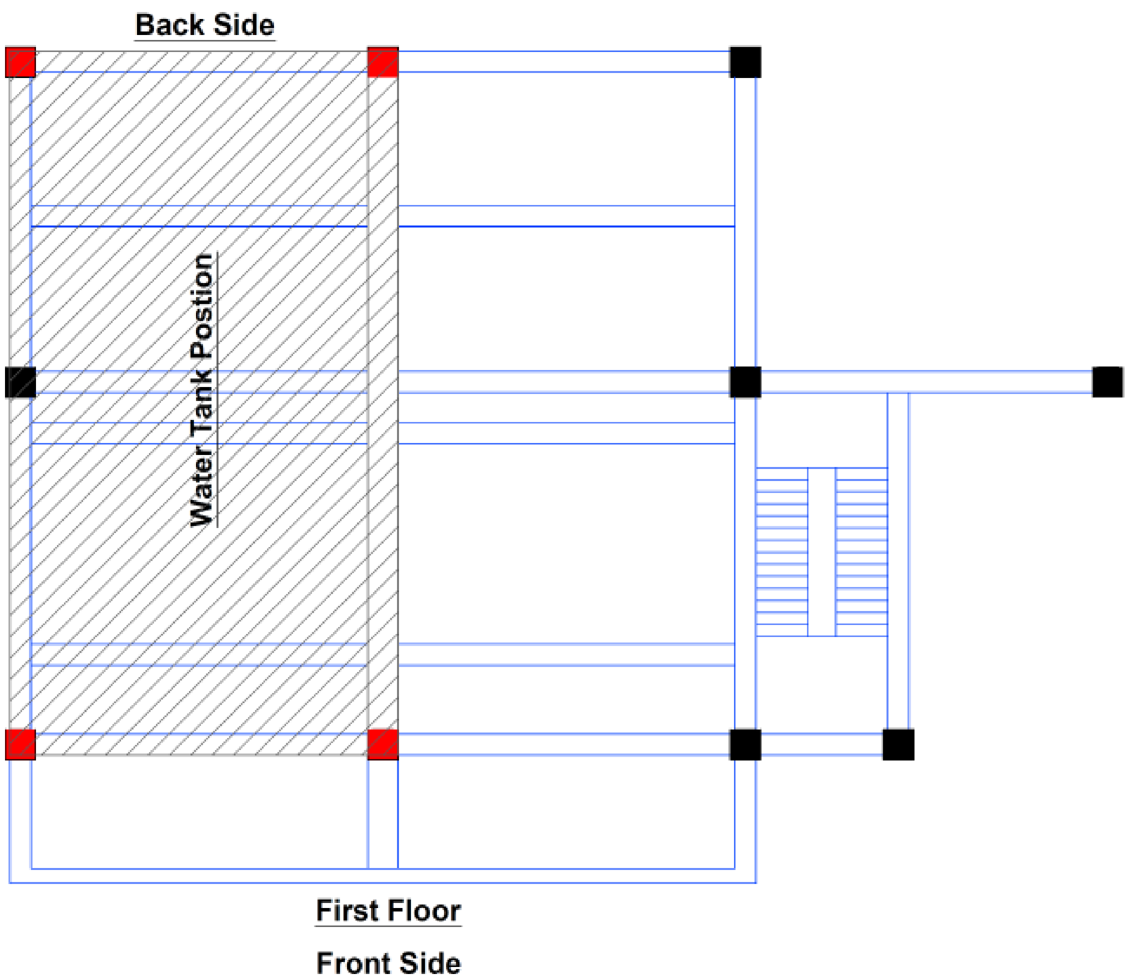
2.3 Specifications for Superstructure and Sub structural elements

1. Grade of Concrete: Columns: M25; Beams: M20
2. Steel: Fe415
3. Slab thickness: 150mm
4. Density of Concrete: 25 kN/m³
5. Density of Brick: 20 kN/m³
6. Earthquake Zone: V
7. Importance factor, I=1, SRF=3
8. Floor to floor height = 3.5m
9. Live load on floors: 2kN/m²
10. Dead Loads: Floor load, Slab dead load (3.75kN/m²) + Floor finish load (0.625kN/m²) = 4.375kN/m²

11. Water Tank load: 3.17 kN/m²

2.4 Location of water tank over first floor column-beam system

Capacity of Tank =20000 liters



2.5 Load combinations

Combination of Loads considered in analysis: The structural design has been carried out in accordance with the provisions of the codes IS 456 – 2000 and IS 1893 – 2002 for Normal design conditions.
Table of Load combinations and load factors as per (Ref. IS: 456 – 2000, CI.18.2.3.1, 36.4.1, and B4.3)

Table 18 Values of Partial Safety Factor γ_f for Loads
(*Clauses* 18.2.3.1, 36.4.1 *and* B-4.3)

Load Combination	Limit State of Collapse			Limit States of Serviceability		
	DL	*IL*	*WL*	*DL*	*IL*	*WL*
(1)	(2)	(3)	(4)	(5)	(6)	(7)
DL + *IL*	1.5		1.0	1.0	1.0	–
DL + *WL*	1.5 or 0.9[1]	–	1.5	1.0	–	1.0
DL + *IL* + *WL*	1.2			1.0	0.8	0.8

NOTES

1 While considering earthquake effects, substitute *EL* for *WL*.

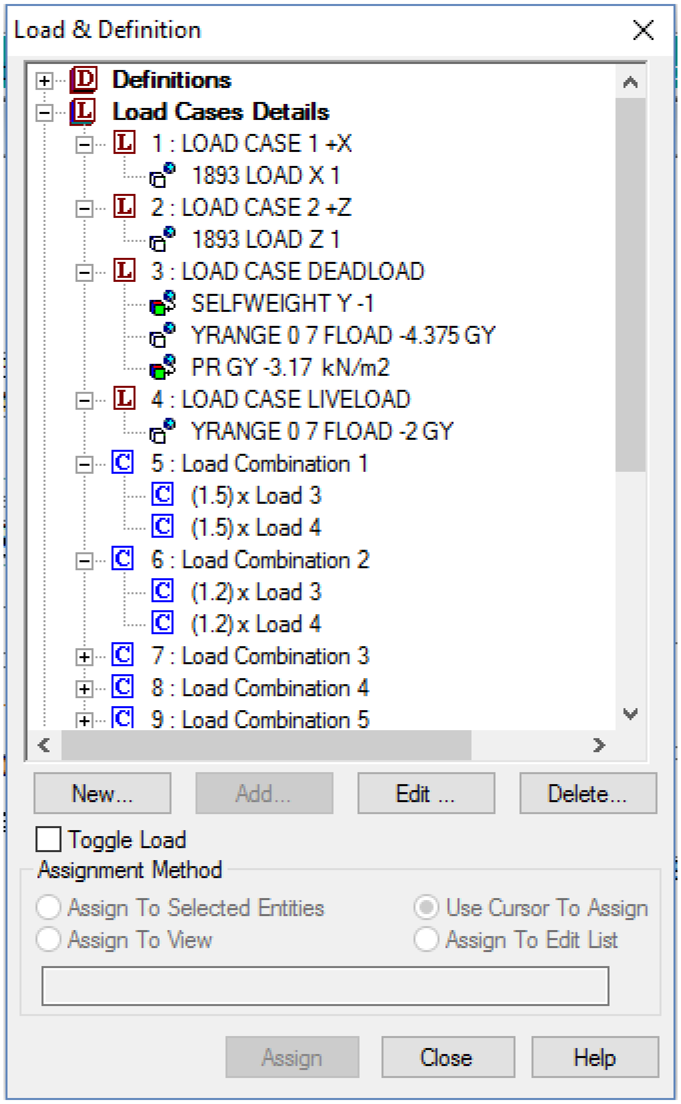
2 For the limit states of serviceability, the values of γ_f given in this table are applicable for short term effects. While assessing the long term effects due to creep the dead load and that part of the live load likely to be permanent may only be considered.

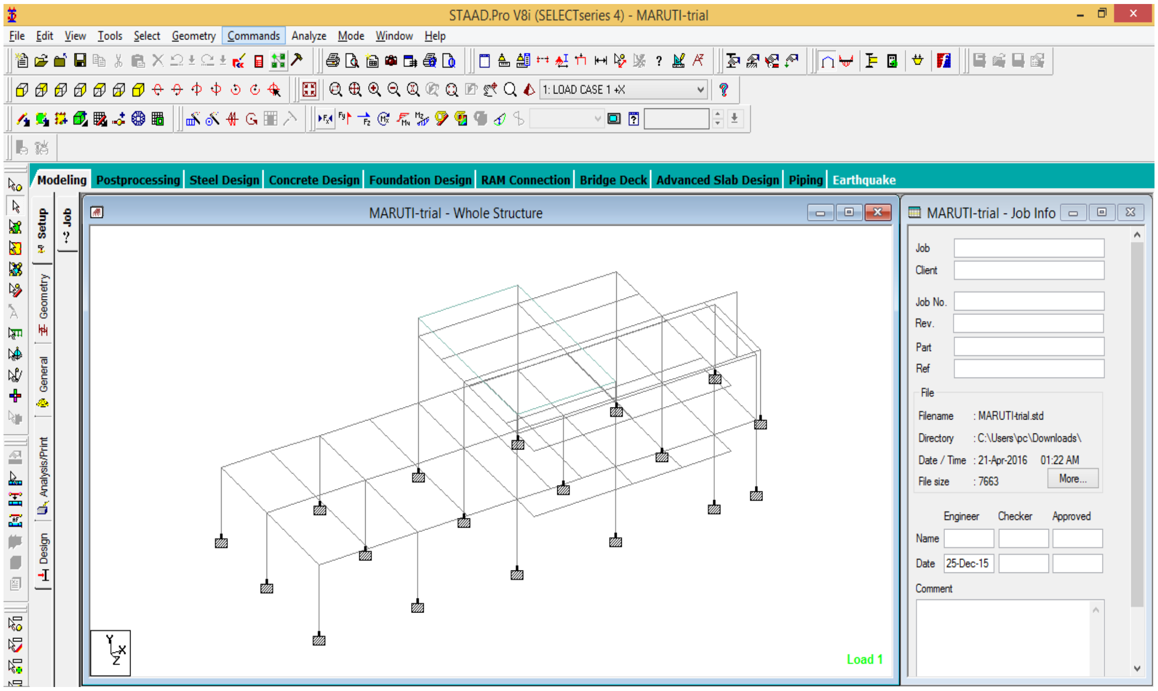
[1] This value is to be considered when stability against overturning or stress reversal is critical.

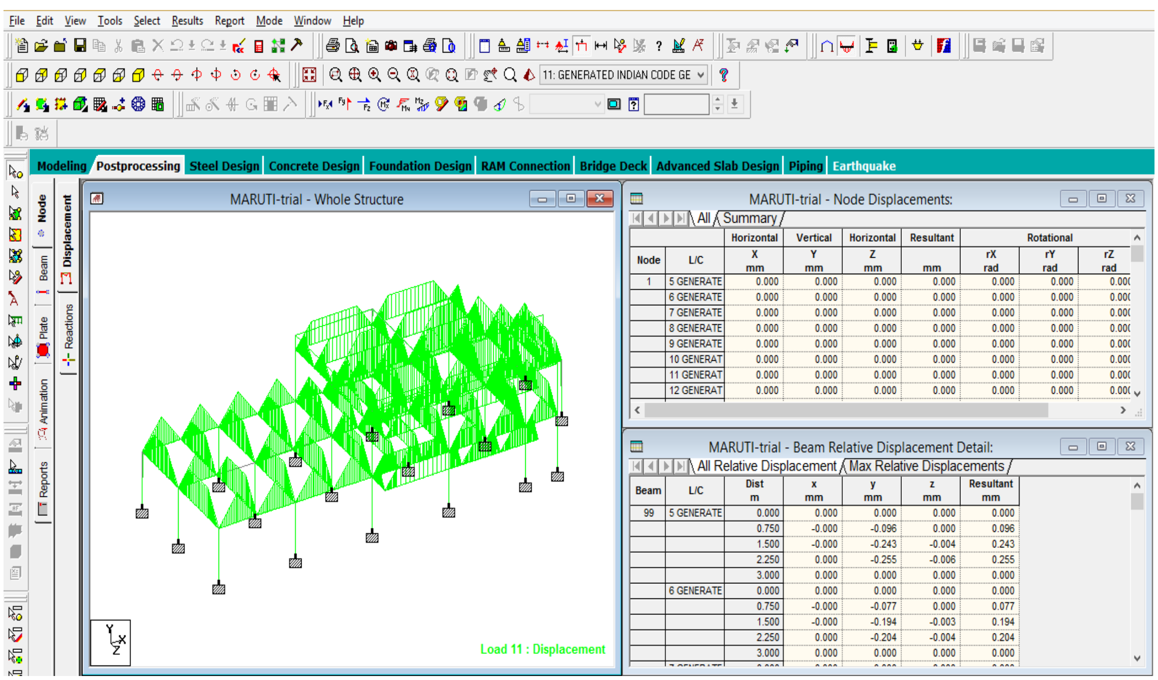
Note: **DL** = Dead Load, **LL** = Live Load/ Superimposed Load, **WL** = Wind Load, **EL** = Earthquake load

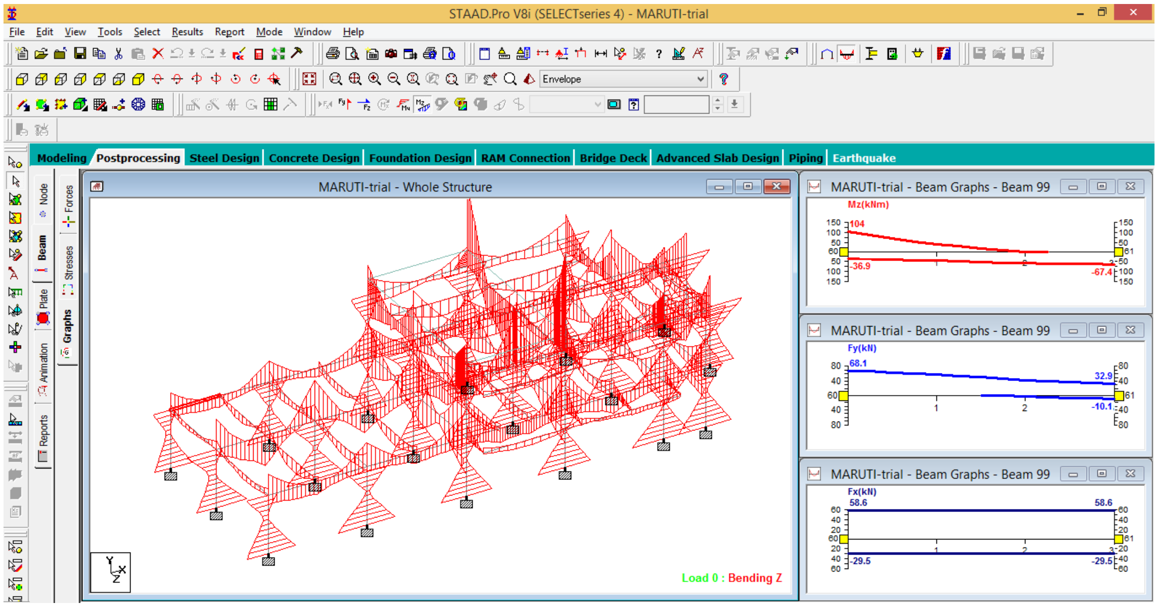
Earthquake loads have been considered in accordance to IS 1893 - 2002. Load case 1 and 2 consists of seismic loads. Water tank load has been included in dead load.

Load case combinations are as shown below:




STAAD.Pro V8i (SELECTseries 4) - MARUTI-trial
Modeling Postprocessing Steel Design Concrete Design Foundation Design RAM Connection Bridge Deck Advanced Slab Design Piping Earthquake
MARUTI-trial - Whole Structure
MARUTI-trial - Job Info
Load 1


Modeling Postprocessing Steel Design Concrete Design Foundation Design RAM Connection Bridge Deck Advanced Slab Design Piping Earthquake
MARUTI-trial - Whole Structure
MARUTI-trial - Node Displacements:
MARUTI-trial - Beam Relative Displacement Detail:
Load 11 : Displacement


STAAD.Pro V8i (SELECTseries 4) - MARUTI-trial
File Edit View Tools Select Results Report Mode Window Help
Modeling Postprocessing Steel Design Concrete Design Foundation Design RAM Connection Bridge Deck Advanced Slab Design Piping Earthquake
MARUTI-trial - Whole Structure
Load 0 : Bending Z
MARUTI-trial - Beam Graphs - Beam 99
Mz(kNm)
Fy(kN)
Fx(kN)

2.6 Formulas Used

1. For Beams

a. $$R \equiv \frac{M_u}{bd^2}$$

b. $$\frac{p_t}{100} \equiv \frac{(A_{st})_{reqd}}{bd} = \frac{f_{ck}}{2f_y}\left[1-\sqrt{1-4.598R/f_{ck}}\right]$$

2. For Columns

a. $$M_u \cong 1.15\sqrt{M_{ux}^2 + M_{uy}^2}$$

b. $$\alpha_n = \begin{cases} 1.0 & \text{for } P_u/P_{uz} < 0.2 \\ 2.0 & \text{for } P_u/P_{uz} > 0.8 \\ 0.667 + 1.667\,P_u/P_{uz} & \text{otherwise} \end{cases}$$

c. $$P_{uz} = 0.45 f_{ck} A_g + (0.75 f_y - 0.45 f_{ck})\, A_{sc}$$

d. $$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \leq 1$$

2.7 Analysis of critical beams

Analysis of the structure was carried out in STAAD Pro. After the analysis, using the moment and shear values, we have designed the critical beam members using limit state method in accordance IS 456 - 2002.

Beam no.	b	D	d	M	R	pt	Ast
101	350	500	525	160	1.658568189	0.5146161014	945.6070863
101	350	500	525	58.8	0.6095238095	0.1753054582	322.1237794
121	350	500	525	127	1.3164885	0.3976750454	730.727896
121	350	500	525	78.8	0.8168448332	0.2381558042	437.6112903
127	350	500	525	133	1.378684807	0.4184262561	768.8582456
127	350	500	525	55.9	0.5794622611	0.1663383041	305.6466338
128	350	500	525	6.4	0.06634272757	0.01845756782	33.91578087
128	350	500	525	63.3	0.6561710398	0.1892916927	347.8234853
144	350	500	525	206	2.135406544	0.6908463104	1269.430095
144	350	500	525	83.8	0.8686750891	0.2541546694	467.009205
145	350	500	525	211	2.187236799	0.7111029203	1306.651616
145	350	500	525	80.6	0.8355037253	0.2439017827	448.1695257
149	350	500	525	1.01	0.01046971169	0.002903429539	5.335051777
149	350	500	525	86.4	0.8956268222	0.2625211837	482.382675
200	350	500	525	181	1.876255264	0.5929474377	1089.540917
200	350	500	525	132	1.368318756	0.414952732	762.475645
201	350	500	525	55.3	0.5732426304	0.1644875017	302.2457844
201	350	500	525	1.68	0.01741496599	0.004831400651	8.877698697

2.8 Analysis of Critical Columns

The columns directly under the water tank have been taken up below:

beam	load case	P	My	Mz	Mu	P/(fck*b*D)	Mu/(fck*b*D^2)	c1	pt	pt(provided)	c2	M(new)	Puz	P/Puz	α	check
275	13	318.3	164.01	39.92	194.1181145	0.050928	0.06211779666	0.032	0.8	0.8792	0.075	234.375	3471.9	0.0916789078	0.8194648463	0.9808172393
259	19	184.43	216.23	5.91	248.7573637	0.0295088	0.07960235637	0.05	1.25	1.256	0.081	253.125	3754.5	0.04912238647	0.7485373108	0.9488212908
258	17	379.6	246.09	1.6	283.0094815	0.060736	0.09056303408	0.048	1.2	1.256	0.09	281.25	3754.5	0.1011053403	0.8351755671	0.907797055
274	13	225	269.15	193.3	381.0764294	0.036	0.1219444574	0.085	2.125	2.355	0.19	593.75	4578.75	0.04914004914	0.7485667486	0.9847646628
260	17	221.74	278.95	6.71	320.8852948	0.0354784	0.1026832943	0.062	1.55	1.6328	0.11	343.75	4037.1	0.05492556538	0.7582092756	0.9040907003
276	13	592.16	300.5	25.15	346.7832055	0.0947456	0.1109706258	0.06	1.5	1.6328	0.12	375	4037.1	0.1466795472	0.9111325787	0.9025307474
261	19	468.4	220.71	12.25	254.2071465	0.074944	0.08134628688	0.042	1.05	1.1304	0.0823	257.1875	3660.3	0.1279676529	0.8799460882	0.9427170911
259	19	184.43	216.23	5.91	248.7573637	0.0295088	0.07960235637	0.048	1.2	1.256	0.0811	253.4375	3754.5	0.04912238647	0.7485373108	0.9479454113
271	14	209.54	37.17	105.48	128.6131913	0.0335264	0.04115622121	0.02	0.5	0.628	0.055	171.875	3283.5	0.06381604995	0.7730267499	0.9917641412
272	15	457.36	337.21	34.36	389.7994403	0.0731776	0.1247358209	0.081	2.025	2.15875	0.131	409.375	4431.5625	0.1032051336	0.8386752227	0.9750745835
255	17	294.59	186.15	7.83	214.2617938	0.0471344	0.06856377401	0.035	0.875	0.8792	0.0687	214.6875	3471.9	0.08484979406	0.8080829901	0.9599924744
254	17	105.19	198.64	7.71	228.6080071	0.0168304	0.07315456228	0.045	1.125	1.1304	0.074	231.25	3660.3	0.02873808158	0.7145634693	0.9850913207

2.9 Foundation Design for columns

Size of footing

- Given: P_u = 1175kN, M_{ux} = 230kNm, q_a = 150kN/m^2 at a depth of 1.25 m.
- As the moment is reversible, the footing should be symmetric with respect to the column. Assuming the weight of the footing plus backfill to constitute about 15 percent of P_u, resultant eccentricity of loading at footing base, $e = 230 \times 10^3 / 1175 \times 1.15 = 171$ mm
- Assuming e < L/6 (i.e., L > 6 x 171 = 1026 mm)

$$1.15 \times 1175/BL + 230 \times 6/BL^2 \leq (150 \times 1.5)\ \text{kN/m}^2$$

$$\Rightarrow 225\ BL^2 - 1351.25L - 1380 \leq 0$$

Various combinations of width B and length L can satisfy the above equation.

Assuming

B = 1.0 m ⇒ L ≥ 6.89 m

B = 2.0 m ⇒ L ≥ 3.808 m

B = 2.5 m ⇒ L ≥ 3.175 m

B = 3 m ⇒ L ≥ 2.75 m

• An economical proportion of the base slab is generally one in which the projection beyond the face of column (or pedestal) is approximately equal in both directions (for effective two-way behaviour, i.e., $(L – a)/2 \approx (B – b)/2$.

• Provide B = 2500 mm and L = 3200 mm; this gives projection of 1000 mm (in the short direction) and 1350 mm (in the long direction).

Thickness of footing based on shear

• Factored (net) soil pressure $q_{u,max}$ = 1175 /2.5x3.2 + 230x6/ 2.5x 3.2^2

= 146.8 + 53.9

= 200.78 kN/m2

• $q_{u,min}$ = 146.8 – 53.9 = 92.975 kN/m^2

(a) One-way shear

• The critical section is located d away from the column face. The average pressure contributing to the factored one-way shear is

q_u = 200.78 – 53.9 × {(1350 – d)/2}/1600

= (178.04 + 0.016d) kN/m2

≈ 190 kN/m2 (assuming d = 600 mm conservatively)

= 0.19 N/mm2

⇒ V_{u1} = 0.19× 2500 × (1350 – d)

= (641250– 475d) N

• Assuming τ_c = 0.36 MPa (for M 20 concrete with nominal p_t = 0.25),

V_{uc} = 0.36 × 2500 × d = (900d) N

$V_{u1} \le V_{uc} \Rightarrow$ 641250– 475d ≤ 900d

⇒ d ≥ 467 mm

(b) Two-way shear

• The critical section is located d/2 from the periphery of the column all around. The average pressure contributing to the factored two-way shear is q_u = 146.8 kN/m^2 = 0.1468 N/mm^2

$\Rightarrow V_{u2}$ = 0.1468 [2500x3200 – (500 +d)(500+d)]

Assuming d = 467 mm (the minimum required for one way shear),

$V_{u2} = 1040 \times 10^3$ N

• For two-way shear resistance, limiting shear stress of concrete

$\tau_{cz} = k_s\,(0.25\, f_{ck})$, where k_s = 0.5 + 500/500, but limited to 1.0.

$\Rightarrow \tau_{cz} = 1.0 \times 0.25\ 20 = 1.118$ MPa

$\Rightarrow V_{uc} = 1.118 \times [(500 + d) + (500 + d)] \times 2 \times d$

$= (2236d + 2.236d^2)$ N

With d = 467 mm $\Rightarrow V_{uc}$ = 2020 kN > V_{u2} = 1040 kN

• Hence, one-way shear governs the footing slab thickness and d ≥ 467 mm. Assuming a clear cover of 75 mm and a bar diameter of 16 mm,

D ≥ 467 + 75 + 16/2 = 550 mm

Provide D = 550 mm

$\Rightarrow$ Effective depth (long span) dx = 550 – 75 – 8 = 467 mm

Effective depth (short span) dy = 467 – 16 = 451 mm

Check maximum soil pressure

• Assuming unit weights of concrete and soil as 24 kN/m^3 and 18 kN/m^3 respectively, at the factored loads,

$q_{max\text{-}gross}$ = 1175/(2.5x3.2) +{(24x0.55)+ 18x (1.25-0.55)} x 1.5 + 230x6/(2.5x3.2^2)

=239.48kN/m^2 ~ 150x1.5 kN/m^2 -Hence, OK.

Design of flexural reinforcement

• The critical sections for moment are located at the faces of the column in both directions (XX and YY) .

(a) Long span

- cantilever projection = 1350mm, width = 2500 mm, d_x = 467 mm, q_u = 0.1553 N/mm^2 at face of column, 0.2007 N/mm^2 at footing edge.
- M_{ux} = (0.1553 × 2500 × 1350^2 /2) + (0.2007 – 0.1553) × 1/ 2 × 2500 × 1350^2 × 2/3
  = (353.8 + 68.26) × 10^6 = 422.06 × 10^6 Nmm

⇒ R ≡ M_u /bd_x^2 = 422.06 x10^6 /2500x 467^2= 0.774MPa

⇒ (p_t) /100 = 20/ (2x415) [1- 1-4.598*0.774/20]= 0.225 × 10^{-2}

p_t assumed for one-way shear = 0.25 > 0.225

⇒ $(A_{st})_{reqd}$ = 0.25 × 2500 × 467/100 = 2918 mm^2

- Using 16 φ bars, number required = 2918/201 = 15

[Corresponding spacing = (2500 – 75 × 2 – 16)/10 = 233 mm]

Provide 15 nos 16 φ bars at uniform spacing in the long direction.

- Development length required = 47.0 φ (for M 20 with Fe 415)
  = 47.0 × 16 = 752 mm
  < 900 mm available — OK.

(b) Short span

- cantilever projection = 1000 mm, width = 3200 mm,

d_y = 451 mm, q_u varies along the section YY, with an average value of 0.1468 N/mm^2 at the middle. Considering a slightly greater value (mean of values at centre and footing edge),

q_u ≈ (0.1468 + 0.2007)/2 = 0.17375 N/mm^2

- M_{uy} = 0.17375 × 3200 × 1000^2 /2) = 278 × 10^6 Nmm

⇒ R ≡ M_u/bd_y^2 =278x10^6/(3200x451^2) = 0.427MPa

⇒ (p_t)/ 100 = 20/ (2 x415) [1 -1-4.598*0.427/20] = 0.121 × 10^{-2}

$\Rightarrow (A_{st})_{reqd} = 0.121 \times 10^{-2} \times 3200 \times 451 = 1752\ mm^2$

$(A_{st})_{min} = 0.0012 \times 3200 \times 550 = 2112\ mm^2 > 1752\ mm^2$

• Number of 12 φ bars required = 2112/113 = 19

As the difference in dimensions between the two sides (B = 2500 mm, L = 3200 mm) is not significant, it suffices to provide these bars at a uniform spacing.

• Provide 19 nos 12 φ bars in the short direction at uniform spacing.

• Development length required = 47.0 × 12 = 564 mm < 775 mm available — Hence, OK

Transfer of forces at column base

• As some of the bars are in tension, no transfer of the tensile force is possible through bearing at the column-footing interface, and these bars may be extended into the footing.

• Required development length of 25 φ bars in tension = 47.0 × 25 = 1175 mm

• Length available (including standard 90o bend on top of upper layer of footing reinforcement) = (550 – 75 – 16 – 12 – 25/2) + 8 × 25 = 634 mm. The balance, 1175 – 634 = 541 mm, can be made up by extending these bars into the footing beyond the bend. A total extension of 4 × 25 + 541 = 641 ≈ 650 mm needs to be provided beyond the bend point. As the moment on the column is reversible, this embedment should be provided for all the column bars.

• Alternatively, a pedestal (with cross-sectional dimensions of, say, 450 mm × 750 mm) may be provided to the column below ground level (or 150 mm below GL), and the longitudinal bars in the pedestal designed to resist the factored axial load-moment combination; small diameter bars (say 16 mm φ) may be selected, with the aim of reducing the development length requirements.

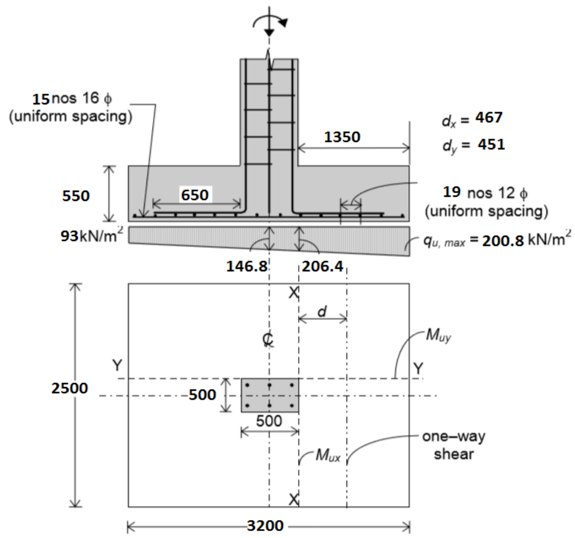


Fig. 14.15 Example 14.5