# Design Calculation Report-
# Faculty Residence, IISER Tirupati
# CE514L-Term Project

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## Contents

# 1 Introduction

The design calculation report is a companion to the design basis report. In this report, a manual design of a typical slab, beam, column and footing is performed and appropriate detailed drawings of the sections are produced. This report also summarises the analysis of sections based on STAAD.pro software. The basis for the design is reported in the design basis report.

## 1.1 Details of Building

The proposed structural design is for the faculty residential building in the Indian Institutes of Science Education and Research (IISER) at Tirupati. The structural design satisfies the functional and non-structural constraints imposed by the architectural plan prepared by Arcop Associates Pvt Ltd. Based on the architectural drawings, the building is a G+3 building with an accessible roof. The height of the building is 18.2m. The horizontal dimensions of the building span 15.35×42.9m.

## 1.2 Details of Structural Model

The structural skeleton frame model is presented in Fig-1. The vertical plates indicate the shear wall.

### 1.2.1 Preliminary Design Considerations

The frame model of the structure is indeterminate, thus the results of the force analysis would depend on the stiffness(indirectly sectional dimensions) of the members. To initiate force analysis all vertical members are assumed to be of dimension 400×400mm, the horizontal members are assumed 450×300mm, the slabs are considered to be 150mm thick and the shear walls are considered to be 300mm thick.

# 2 Manual Design of Typical Slab

The selected slab for manual design is highlighted in Fig-2. The dimensions of the slab are 3900×3900×150mm. The load details of the slab are presented in Table-1, The governing load combination in the design

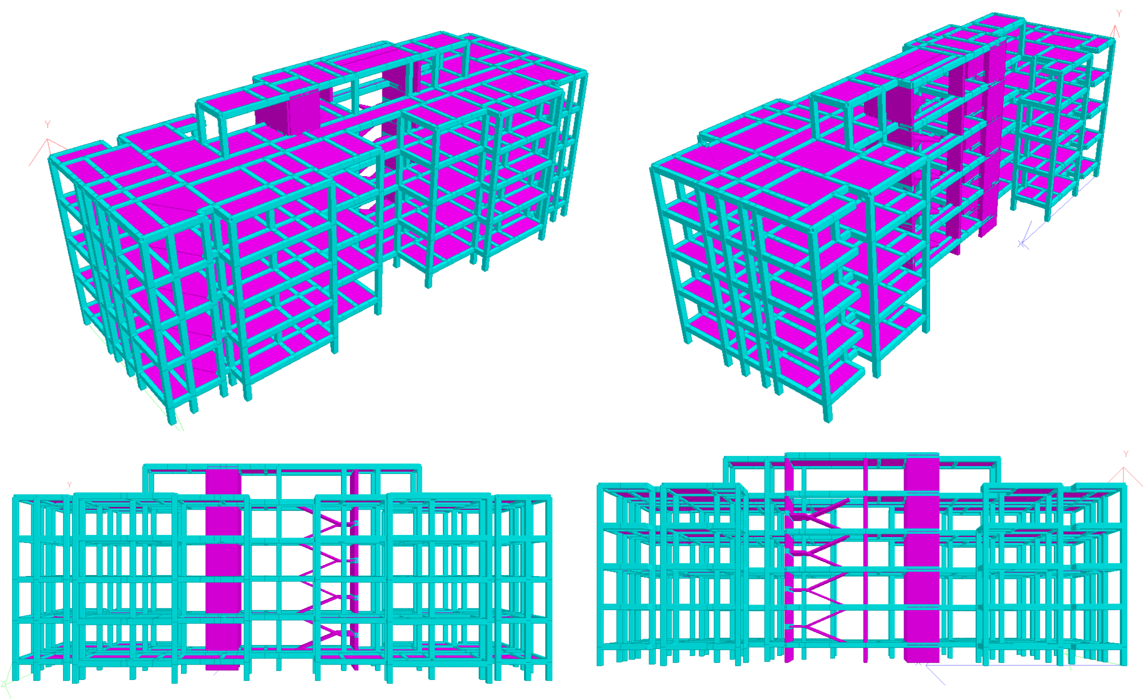


Figure 1: 3D rendered views of skeletal frame of the structure

of the slab is

$$1.5DL + 1.5LL = 10.125KN/m^2 \tag{1}$$

The ratio of the longitudinal dimensions of the slab is $l_x/l_y = 1 < 2$, the slabs are expected to behave as two-way slabs. The slab has two adjacent discontinuous edges. Based on Table 26 of IS 456:2000, the positive moment per width in the midspan is $5.39KN$ and the Negative moment per width at the continuous edge is $7.24KN$.
The l/d ratio of the member=3900/150=26==26(criterion), eventually based on the appropriate modifications, can elevate the member's stiffness.

### 2.1 Flexural design

Assuming longitudinal reinforcement to be performed with 8mm$\phi$ rebars. Assuming the clear cover of the member to be 20mm. The effective cover of the member in the x direction will be 126mm and in the y direction will be 118mm.
for one end continuous and other discontinuous the effective length in the x and y direction is $L_x + d_x/2 = 3963mm$ and $L_y + d_y/2 = 3959mm$. In ends with both ends discontinuous the effective length in the x and y direction is $L_x + d_x = 4026mm$ and $L_y + d_y = 4018mm$.
The moment resistance capacity of the flexural concrete member as per IS 456:2000 is

$$M_{ur} = 0.36x_u f_{ck} b(d - 0.42x_u) \tag{2}$$

The relation between the tensile and compressive forces is given below

$$(f_y/1.15)A_{st} = 0.36bf_{ck}x_u \tag{3}$$

The moment capacity per width at the section must exceed 5.39KNm for the positive moment at midspan. And to ensure the system is under-reinforced the $x_u > x_{u,balanced}$. $x_{u,balanced}$ of the system is 60.375mm

$$M_{u,lim} > M_{ur} = 0.36x_u f_{ck}(d - 0.42x_u) = 0.36 \times 35_u \times (118 - 0.42x_u) > 5.39 \tag{4}$$

$$\begin{gathered} 60.375mm > x_u > 3.434mm \\ 999.81mm^2 > A_st > 56.87mm^2 \end{gathered} \tag{5}$$

The $A_{st}$ variable mentioned in eqn-5 is the area per a width of 1000mm. The minimum area corresponds to a spacing of 1200mm, as the spacing can't exceed 300mm spacing is to be provided, the $A_{st}$ will amount to $261mm^2$ The corresponding percentage steel is 0.208%, which is greater than the minimum reinforcement percentage of 0.12% suggested by IS 456.
Similarly, design for the negative moment is performed and summarized in Table-2.
Based on fig-4 of IS456:2000, the modification factor for over strength is 2. Thus the $l/d$ ratio is under defined limits.

### 2.2 Shear Check

The maximum shear load at a distance d from the edge is 10.125×3.8×3.8/4=37.5KN. For 0.2% steel the design shear strength of concrete is 0.33Mpa, and the design shear capacity is 154Kn. Thus the slab is safe under shear.

| Nature of Load | | Intensity of Load |
|---|---|---|
| Dead Load | | 4.75KN/m$^2$ |
| 1 | Self-weight | $25 \times 0.150 = 3.75KN/m^2$ |
| 2 | Floor Finish Load | 1KN/m$^2$ |
| Live Load | | 2KN/m$^2$ |

Table 1: Load Details of Selected Slab

| Description | Mu | Diameter of bars | Spacing | Mur | Ductility of sysetm |
|---|---|---|---|---|---|
| Positive moment at midspan | 5.39KN | 8mm | 300mm | 19.9KN | Under-Reinforced |
| Negative moment at continuous edge | 7.24KN | 8mm | 300mm | 19.7KN | Under-Reinforced |

Table 2: Summary of longitudinal reinforcement for selected slab

| S.No | Type of Load | Magnitude of Load(maximum) | Associated Load case |
|---|---|---|---|
| 1 | B.M(-ve) | 28.12KNm | 1.2DL+1.2LL+1.2ELx |
| 2 | B.M(+ve) | 6.76KNm | 0.9DL+1.2ELx |
| 3 | V | 22.25KN | 1.5DL+1.5LL |
| 4 | T | 1.66KNm | 1.2DL+1.2LL+1.2ELx |

Table 3: Critical load on beam for considered load combinations

### 2.3 Torsional reinforcement in discontinuous slab corners

Appropriate torsional reinforcement is provided at the corners to restrain uplift at corners. Cl D-2.1.1 of IS 456:2000 is followed.

### 2.4 Detailing

The detailing for anchorage length along with development checks and torsional reinforcement at corners is performed in the below drawing. The total length of the beam is 6100mm(c/c between columns).
The detailed structural drawing of the designed slab is presented on page 9.

## 3 Manual Design of Typical Beam

The selected beam for manual design is highlighted in Fig-3. Based on Staad pro analysis the following critical loads were determined. Based on the response envelope of all load combinations, the critical loads were calculated and reported in table-3 The maximum hogging moment occurs at the centre of the beam and the sagging moment arises at the supports. As from the analysis, the beam dimensions are fixed at $300 \times 450mm$, appropriate reinforcement is to be designed for this fixed section. The column dimension is assumed to be $400 \times 400mm$. The clear span of the beam is 5700mm. The effective length of the beam is 6100mm.
Assuming a clear cover of 30mm, stirrup dia of 10mm, and bar of 20mm, the effective depth would be 405mm. $L/d$ ratio of the beam is $6100/400 = 15.08 < 20$. The beam satisfies the deflection check.

### 3.1 Flexural design

The moment resistance capacity of the flexural concrete member as per IS 456:2000 is specified in equation-2 For the positive moment at midspan the moment capacity at the section must exceed 6.76KNm. And to ensure the system is under-reinforced the $x_u > x_{u,balanced}$. $x_{u,balanced}$ of the section

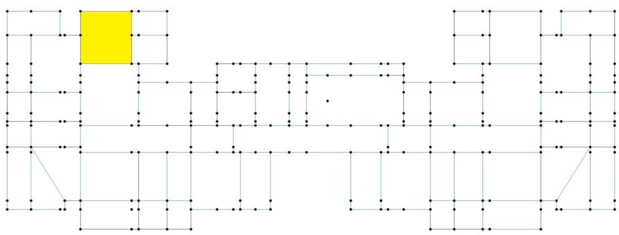


Figure 2: Layout of $3^{rd}$ floor plan, Detailed Design performed for the highlighted slab.

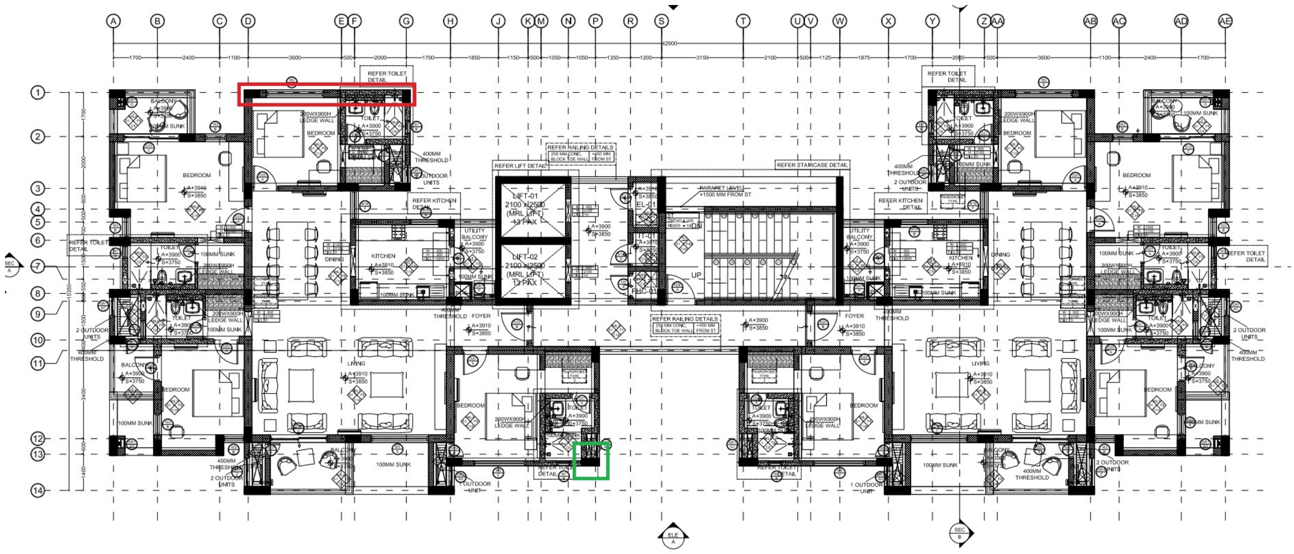


Figure 3: Layout of $1^{st}$ floor plan, Detailed design is performed for the highlighted beam and column(and footing)

can be calculated by considering the neutral axis depth when the strain value at the steel section reaches yield and concrete fails in compression simultaneously. ($\epsilon_{st} = f_y/1.15E_s = 0.0035(d - x_{N.A})/x_{N.A}$) Substituting $f_y$=500MPa, the depth of the balanced neutral axis is 249mm. Based on equations-2 and 3, the minimum depth of the neutral axis is to be achieved.
Based on Cl41.3 of IS 456:2000, the design value of moment and shear accounting for torsions is given by the equation-6

$$V_e = V_u + 1.6T_u/b; M_e = M_u + M_t \tag{6}$$

The design moment for hogging is 9.88KNm, for sagging is 31.25KNm and for shear is 22.26KN. For a hogging moment of

$$M_{u,lim} > M_{ur} = 0.36x_u f_{ck}(d - 0.42x_u) = 0..36 * 35 * x_u * (400 - 0.42x_u) > 9.88 \tag{7}$$

$$\begin{gathered} 249mm > x_u > 100.67mm \\ 2608mm^2 > A_st > 100.67mm^2 \end{gathered} \tag{8}$$

To resist crack propagation minimum steel of area 332.53mm$^2$ as per cl26.5.1 is to be provided. Thus 3 of 12mm bars can be used. The effective area is $339mm^2$¿$A_{st,min}$. $x_u$ of the provided section is 56.69mm, with an effective moment capacity of 46.06KNm By noticing that the minimum steel was the main constraint the same design can be extended to top steel for sagging. Thus 3 of 12mm dia steel bars would suffice for both hogging and sagging. The spacing between the bars well exceeds the minimum spacing for aggregates to flow through.
The percentage of tensile steel is 0.28% which is less than the maximum limit of 4%

### 3.2 Shear Design

The maximum shear experienced by the member for the considered load combination is 22.25 KN. Due to a lack of knowledge about the envelope for the critical section a distance d away from support, the shear at the critical section will be designed for maximum shear. The average shear stress experienced in the section is $22.25/404 \times 300$=0.183MPa. For a tensile steel per cent of 0.28%. The shear capacity of concrete is 0.37MPa, thus no additional shear reinforcements are required, and only minimum shear steel shall be provided. The minimum shear reinforcement shall be in accord with equation-9

$$A_{sv}/bs_v = \frac{0.4}{0.87f_y} \tag{9}$$

For Fe500 bars of 8mm dia, the maximum allowable spacing(cl 26.5.1) is 364mm as the spacing can't exceed 300mm, we shall provide 8mm two-legged stirrup bars at a spacing of 300mm throughout the member.

### Additional Comments

As shown earlier L/d¡20 and while considering modification factors, the value further decreases. And as $T_u << M_u$ no additional compressive reinforcement is provided as only modified bending failure(skew bending theory) is predicted.

## 3.3 Detailing

The development length of 12mm bars for a full design strength of 500/1.15Mpa in the concrete of grade M35 is 479mm(Cl 26.2.1.1). The required anchorage length as per Cl 26.2.3.3 is $L_0 + 1.3M_u/V_r \geq L_d$. Thus $L_0 \geq 479 - 1.3(46.06/22.25) \geq 0$..No additional anchorage is needed, but from the face of the member the bars must extend a distance of $L_d/3 = 160mm$, the bars shall be extended 100mm into the support and then bent 90° with an extension of 50mm. The effective anchorage length is 196mm≥180mm.
The detailed structural drawing of a similar beam conjoining the column is presented on page 10. [1]

# 4 Manual Design of Typical Column

The selected column is displayed in Fig-3. Based on the envelope of load responses the maximum responses are displayed in table-4. In analysis, dimensions are assumed as$400 \times 400mm$. Appropriate reinforcement is to be designed. The unsupported length of the column is 3.3m. To determine whether the column is braced/unbraced the stability factor is to be estimated. Lateral flexibility measures are shown in -eqn-**??**. The lateral flexibility would be critical in the traverse direction or along the shorter direction of the building. To simplify calculation all above floors are expected to carry $30KN/m^2$. The columns are assumed to be braced as the stability factor is within limits imposed by IS 456:2000.

$$\begin{gathered}\Delta_u/H_u = h_s^2[12E_{c,col} \times \Sigma(I_c/h_s0^{-1} + 12E_{c,beam} \times \Sigma(I_b/h_b)^{-1}] \\ (\Delta_u/H_u)_z = 3300^2[45 \times 29850 \times \Sigma(400^4/12/3300)^{-1} + 64 \times 29850 \times \Sigma(450^3 \times 300/12/3300)^{-1}] = 5.7 \times 10^{-8}.\end{gathered} \tag{10}$$

$$\begin{aligned}Q = \Sigma P_u/h_s(\Delta_u/H_u) &= 30 \times 15.35 \times 42.9m \times 3/3300 \times (5.7 \times 10^{-8} \\ &= 59300/3300 * 5.7 * 10^{-8} = 0.01 < 004[IS456:2000].\end{aligned} \tag{11}$$

The coefficients $\beta_1, \beta_2$ are estimated from eqn-12. From fig-26 of IS 456:2000, the effective length ratio is estimated at around 0.7. The effective length of the column is $3300 \times 0.7 = 2310mm$. The slenderness ratio is 2310/400=5.775, the member is a short column.

$$\begin{aligned}\beta_1 = \beta_2 &= \frac{\Sigma I_c/h_s}{\Sigma I_c/h_s + \Sigma 0.5 I_b/h_b} \\ &= \frac{2 \times (400^4/12/3300)}{2 \times (400^4/12/3300) + 0.5(\times(450^3/12/4500 + 450^3/12/4000))} = 0.545.\end{aligned} \tag{12}$$

Assuming an effective cover of 50mm, the clear cover is 30mm, and a tie dia of 10mm is assumed. $d'/D \approx 0.05$. Chart-35 of SP-16 is to be used.
The minimum eccentricity is estimated as $maxL/500 + d/300, 20$. The maximum eccentricity is 20mm. The moment due to this eccentricity exceeds the eccentricity of the load and thus the design moments would be $M_x = 74KNm, M_z = 26KNm$. From our designloads, $P_u/f_{ck}bd = 0.127, M_u/f_{ck}bd^2 \approx 1.15(M_x^2 + M_z^2)^{.5}/f_{ck}bd^2 = 0.04$. From graph reinforcement of $0.01_{ck} = 035\%$ would suffice. The

[1] The length and critical load of both beams are similar per architectural drawing and software analysis, the same design of beam suffices

| S.No | Type of Load | Magnitude of Load(maximum) | Associated Load case |
|---|---|---|---|
| 1 | Axial Force | 713KNm | 1.5DL+1.5LL |
| 2 | $M_x$ | 20.835KNm | 1.5DL+1.5LL |
| 3 | $M_z$ | 59KNm | 1.5DL+1.5LL |

Table 4: Critical load on the column for considered load combinations

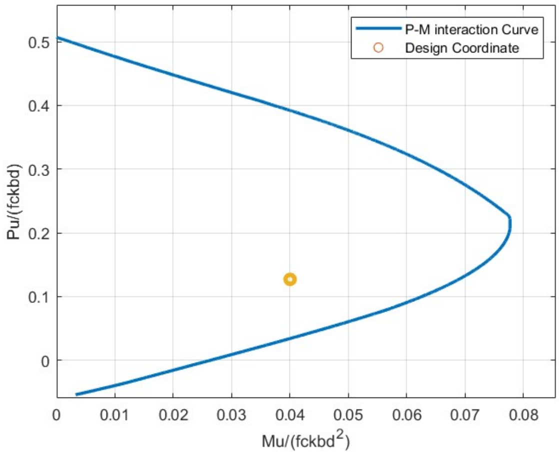



Figure 4: P-M interaction curve for designed column(Code submitted in assignment-6)

corresponding area of steel is $560mm^2$. The column can be split as 310mm steel at the top and bottom and two layers of 210mm steel at an intermediate distance. $P_{st}$ would then amount to 0.05%. $M_{ur} \approx 0.06 \times 400^3 \times 35 = 134KNm$

$$\alpha = f(713/(0.45 \times 35 \times 400 \times 400 + 0.75 \times 500 \times 785mm^2 = 0.25) \tag{13}$$

$$\begin{gathered} [M_{ux}/M_{ux1}]^{\alpha} + [M_{uy}/M_{uy1}]^{\alpha} \leq 1 \\ [74/134]^{1.083} + [26/134]^{1083} \leq 1 \end{gathered} \tag{14}$$

For the diameter of lateral ties, the diameter must not exceed ties of compression reinforcement, thus 6mm bars are assumed. The maximum spacing must be less than 300mm,400mm(=D),16*10. Thus the lateral ties of 6mm dia are to be provided at a spacing of 150mm.
The detailed structural drawing of the designed column is presented on page 10.

### 4.1 P-M Interaction Curve

The P-M interaction curve is obtained from the custom code submitted in assignment 6. The moment interaction curve for this geometry is displayed in fig-4

## 5 Manual Design of Typical Footing

The footing for the selected column is designed in this section Based on the envelope of loads the critical load responses are calculated and reported in fig-5.

| S.No | Type of Load | Magnitude of Load(maximum) | Associated Load case |
|---|---|---|---|
| 1 | Axial Force | 893KNm | 1.5DL+1.5LL |
| 2 | $M_x$ | 26.823KNm | 1.5DL+1.5LL |
| 3 | $M_z$ | 2.7KNm | 1.5DL+1.5LL |

Table 5: Critical load on selected footing for considered load combinations

Considering the weight of the backfill as 15%, the eccentricity of the moments is $26.823/(1.15 \times 893) = 26mm$,(The moment in the other direction is insignificant and is not considered). The maximum length must exceed the maximum eccentricity by 6 times to avoid tension in the soil base. Thus $L \geq 6 \times e = 160mm$
Assuming a soil bearing capacity of 200Kpa at 1.5m below the ground level, the stresses generated due to load must satisfy the equation-15.

$$893 * 1.15/(LB) + 26.82/(BL^2/6) \leq 200 \times 1.5 \tag{15}$$

Assuming a square footing(i.e. B=L), the relation reduces to that state in eqn-16.

$$1026L + 160.92 \leq 300L^3 \tag{16}$$

The solution to the equation-16 is $L > 1.92m$. Let us consider L=B=2m.
The factored soil pressure is then

$$\begin{aligned} q_{max} &= 1026/(4) + 160.92/(8) = 270KPa \\ q_{min} &= 1026/(4) - 160.92/(8) = 236KPa \end{aligned} \tag{17}$$

### One-way shear

The critical section is located d away from the face of the column(x=200mm). Assuming a conservative footing depth of 500mm, the $q_u$ load would be $q_u = 270 - 20.115 \times (1000 - 200 - d)/2000 = 0.265MPa$. The shear at the region in $0.265 \times 2000 \times (800 - d) = 424000 - 530d$ For an M35 concrete with 0.25% steel(assumed), $\tau_c = 0.37MPa$, THe shear strength of concrete is $0.37 \times 2000 = 740d$ For the capacity to exceed load, $d > 3389mm$

### Two-way shear

For two-way shear, the critical section is present at a distance of d/2 from the column, $q_u = 256.5KPa$ The shear force at the critical region is $V_{u,two-way} = 0.2565 \times (2000 \times 2000 - (400 + d)^2)$. For a d of 400mm, $V_{u,two-way} = 860KN$.
For two-way shear the limiting the shear capacity of concrete is $\tau_{cz} = k_s 0.25\sqrt{f_c k} = 1.47Mpa$. $V_{c,two-way} = 1.47 \times (1600 + 4d) \times d$. For d=400mm, $V_{c,two-way}$=1800KN¿$V_{u,two-way} = 860KN$. Thus the effective depth of footing can be 400mm. Assuming a cover of 40mm, The total depth of footing can be set at $D = 450mm$.

### 5.1 Reinforcement Design

The project of the footing is 800mm with a width of 2000mm. A uniform load of $256.5 + 20.21/2)$ is assumed in the projection. The design moment would be $M_u = 0.266 \times 2000 \times 800^2/2 + .011 \times 2000 \times 800^2 \times 2/3 = 179KNm$, the required percentage of steel would amount to 0.13%. For the design of one-way shear, the percentage of steel assumed is 0.25%, so it shall be provided. Area of steel required$\approx 2100mm^2$. Considering 16mm diameter bars, the required bars would amount to 11. Thus the spacing would be 187¡300mm. Thus 11 of 16mm dia bars(both sides) would suffice longitudinal flexural reinforcement on both sides.
Development length for 16mm dia bar=752m≥800mm. Regardless bars are bent 90° at the end with an extension of 64mm.

## 6 Software Aided Design of Members

Using the StaadPro software design for all members was performed. The complete design is included in the submitted Staadpro file. The foundation design performed through Staad Foundation Advanced is also attached to the submission
The detailed structural drawing of the designed footing is presented on page 11.

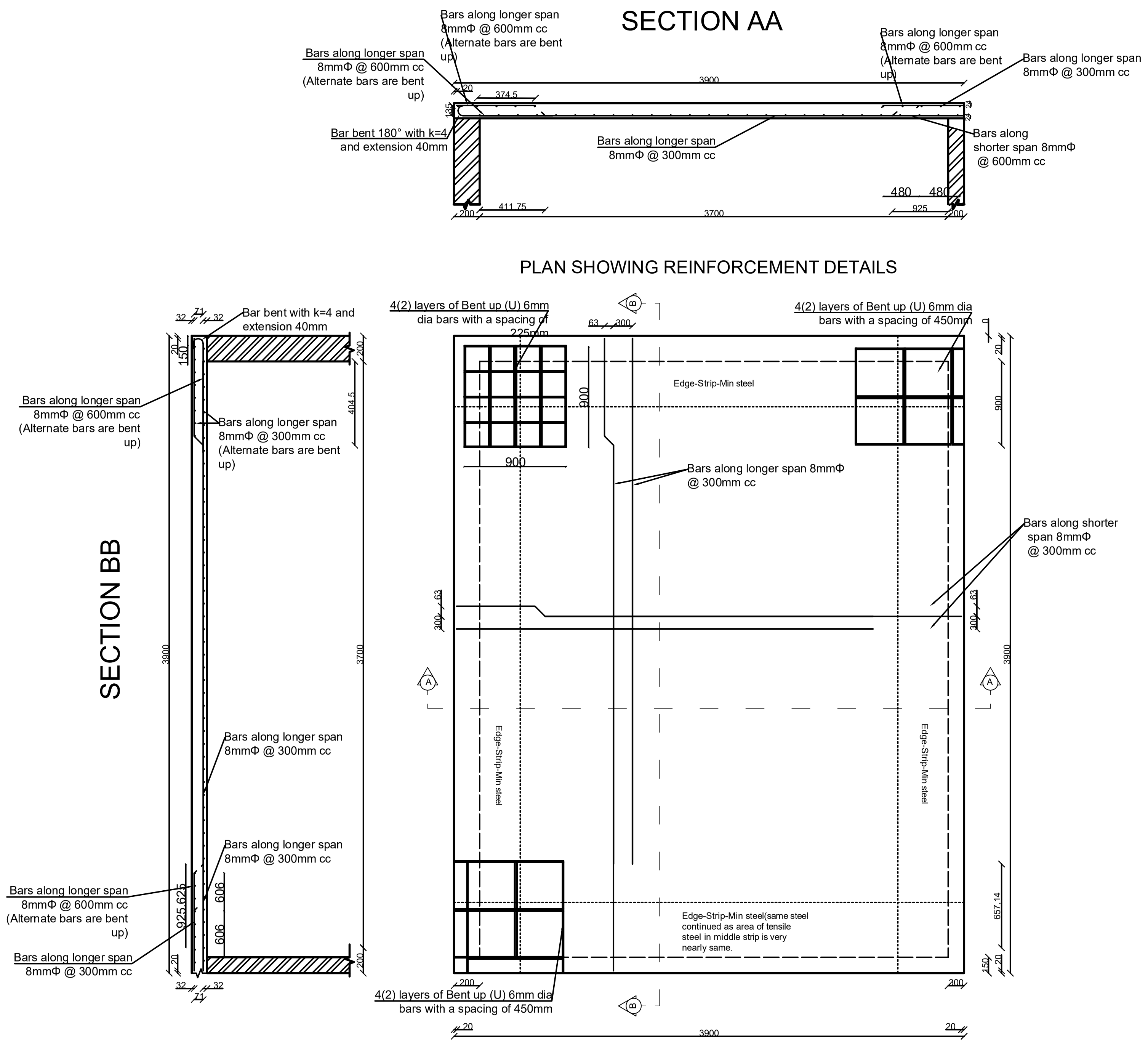
## 7 Detailed Drawings of Manually Designed Members

The detailed drawings for slabs, beams, columns and footings are presented in the below pages[2].

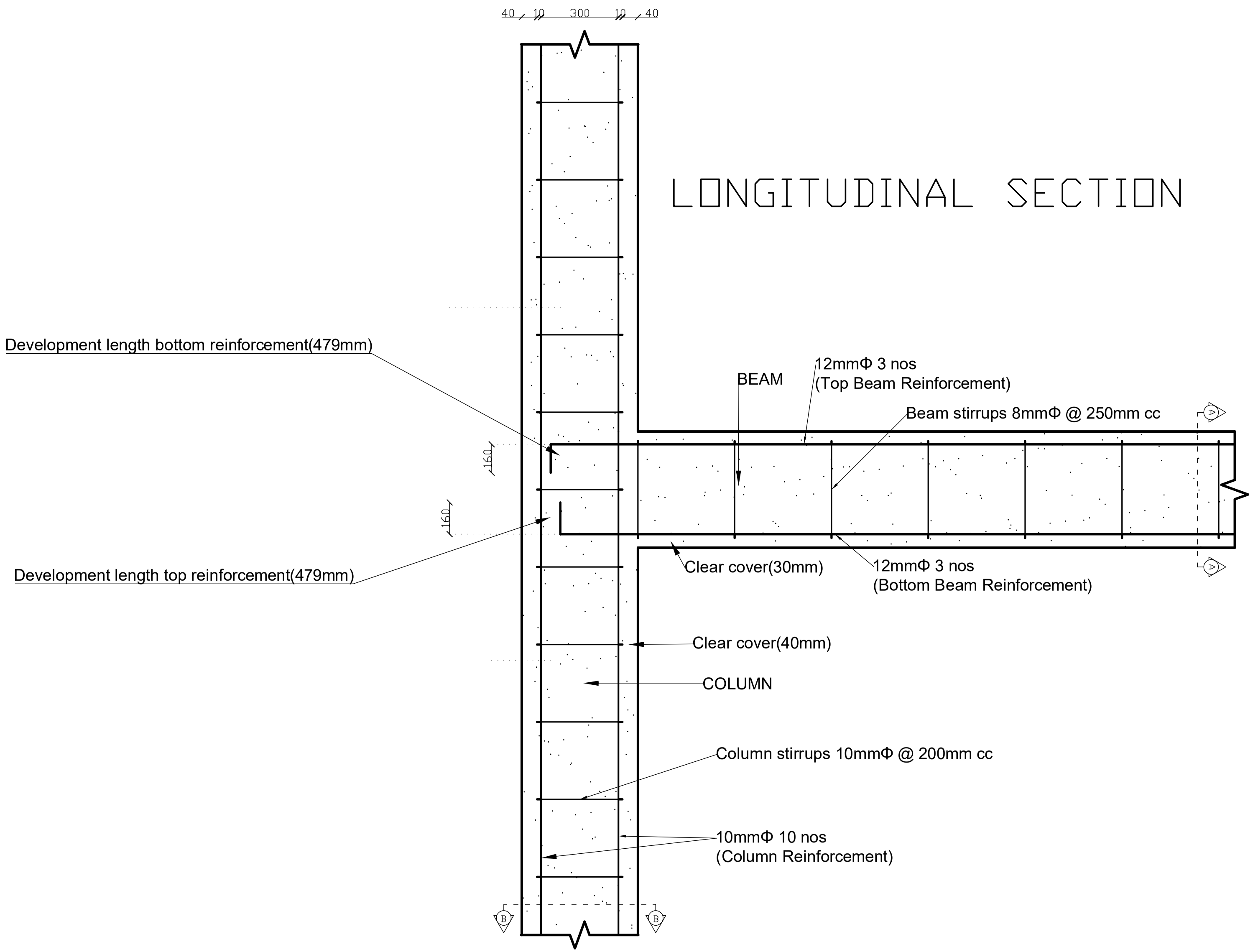
## 8 Conclusion

In this report, the manual design of a typical beam, column, footing and slab is presented along with respective detailed drawings.

[2] For the beam, instead of drawing for the selected beam, the drawing for the beam joining the column is shown, as the beam dimensions are nearly the same and have nearly the same critical loads based on software analysis.


SECTION AA
Bars along longer span 8mmΦ @ 600mm cc (Alternate bars are bent up)
Bars along longer span 8mmΦ @ 600mm cc (Alternate bars are bent up)
Bars along longer span 8mmΦ @ 600mm cc (Alternate bars are bent up)
Bars along longer span 8mmΦ @ 300mm cc
Bar bent 180° with k=4 and extension 40mm
Bars along longer span 8mmΦ @ 300mm cc
Bars along shorter span 8mmΦ @ 600mm cc
3900
374.5
411.75
3700
925
480
480
200
200
PLAN SHOWING REINFORCEMENT DETAILS
4(2) layers of Bent up (U) 6mm dia bars with a spacing of 225mm
4(2) layers of Bent up (U) 6mm dia bars with a spacing of 450mm
4(2) layers of Bent up (U) 6mm dia bars with a spacing of 450mm
Edge-Strip-Min steel
Edge-Strip-Min steel
Edge-Strip-Min steel
Edge-Strip-Min steel(same steel continued as area of tensile steel in middle strip is very nearly same.
Bars along longer span 8mmΦ @ 300mm cc
Bars along shorter span 8mmΦ @ 300mm cc
900
900
900
657.14
3900
3900
SECTION BB
Bar bent with k=4 and extension 40mm
Bars along longer span 8mmΦ @ 600mm cc (Alternate bars are bent up)
Bars along longer span 8mmΦ @ 300mm cc (Alternate bars are bent up)
Bars along longer span 8mmΦ @ 300mm cc
Bars along longer span 8mmΦ @ 300mm cc
Bars along longer span 8mmΦ @ 600mm cc (Alternate bars are bent up)
Bars along longer span 8mmΦ @ 300mm cc
150
404.5
925.625
606
606
3900
3700

# LONGITUDINAL SECTION

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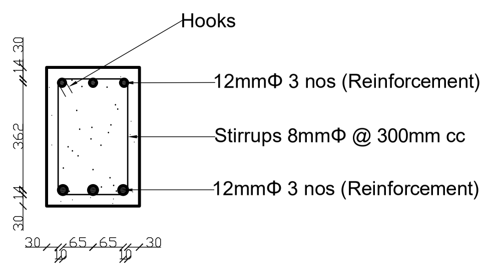
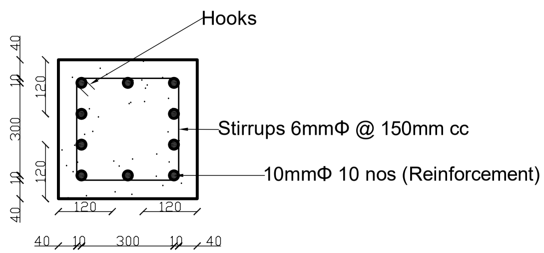
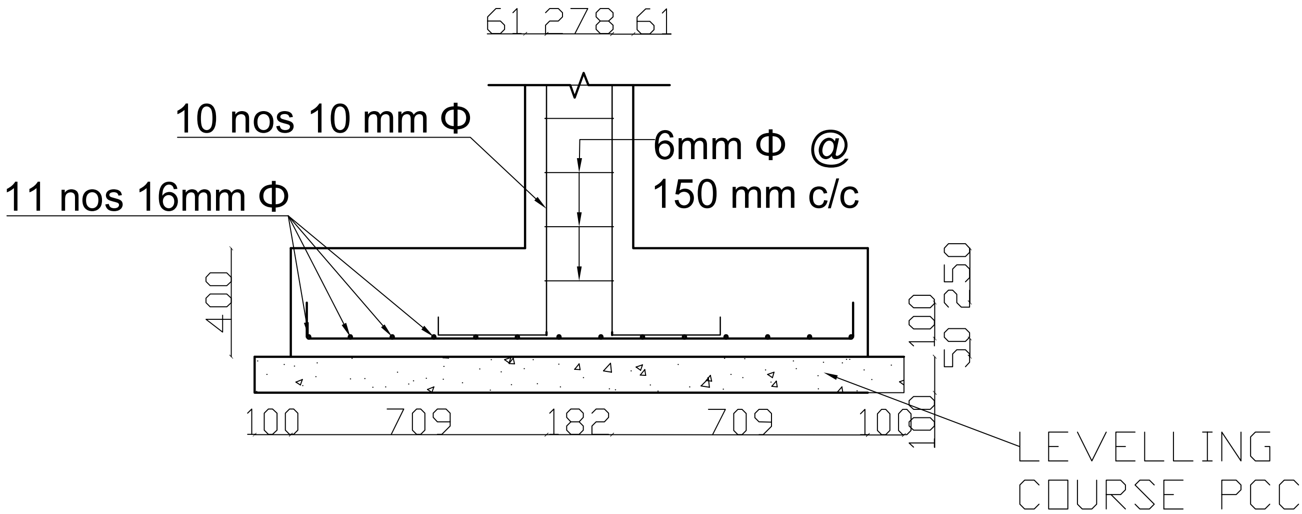



# SECTION BB



# SECTION AA



61 278 61
10 nos 10 mm Φ
6mm Φ @
150 mm c/c
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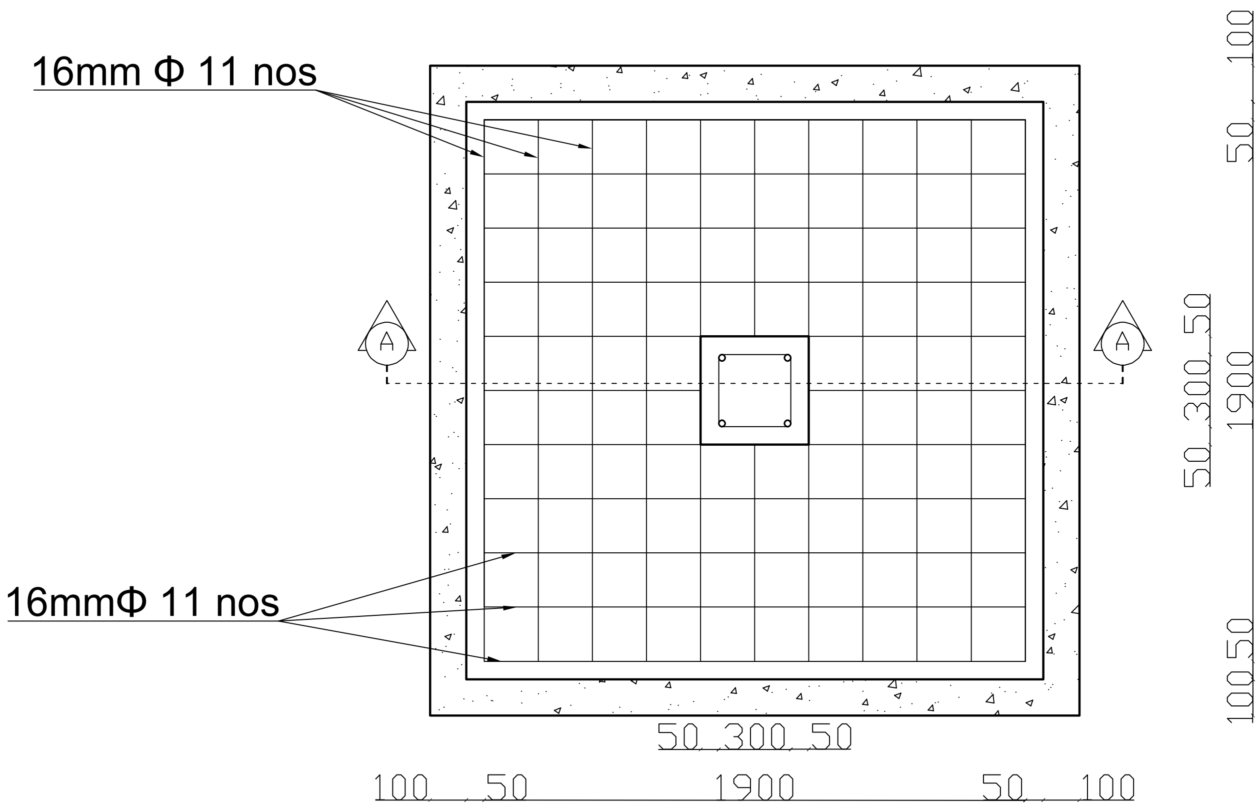
LEVELLING
COURSE PCC

16mm Φ 11 nos
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A
16mmΦ 11 nos
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50
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