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INGENIEUR EN CONSTRUCTION & CONSULTANT INDEPENDANT
DIRECTEUR TECHNIQUE DU BUREAU D'ETUDE BETRAGEC
EXPERT IMMOBILIER AGREE PAR L'ORDRE DES EVALUATEURS
DES BIENS IMMOBILIERS AU RWANDA (IRPV)
CERTIFICATE N° RC/IRPV/063/2011, RÉF. A/009/IRPV/2011
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PROJECT : REINFORCED CONCRETE STRUCTURAL DESIGN OF TWO STORY RESIDENTIAL BUILDING LOCATED AT RUBAVU DISTRICT; NENGO CELL, GISENYI SECTOR, PLOT NUMBER : 574

PROJECT OWNER : Mr EUGENE NKUNDABERA

DESIGN CODE : BS 8110 (Practice for structural use of concrete 2013)

October 2017

I. NOTATION AND ABBREVIATION

BS : British Standard

A_s : Cross sectional area of tensile reinforcement

As' : Cross sectional area of compressive reinforcement

A_{sv} : Cross sectional area of shear reinforcement in the form of links

A_{cr} : Distance from surface of crack to print if zero strain (crack width)

b : Width of any cross section

b_w : Breadth of section width of web

d : Effective dept of section

f_{cu} : Characteristic concrete cube strength

f_s :Service strength of steel

f_y : Characteristic strength of reinforcement

G_k : Characteristic dead load (Design permanent load)

Q_k : Characteristic live load (Design live load)

Ht : Overall dept of section in the plane of bending

Ho : Effective depth of the beam

l_e : Effective height of column

M : Bending moment

M_u : Ultimate moment of resistance

N : Axial load

N : Total distributed load on the slab panel

n : Ultimate design load

n : number of legs (branch) of one stirrup

S_v : Spacing links along member

V : Ultimate shear force

V : Design shear stress

v_c : Design concrete shear stress

Z : Lever arm

γ_m : Partial safety factor for strength

w	: Design load (combination of live and dead loads)
ℓ	: Length of beam span
BAEL	: Béton Armé aux états limites
γ_f	: Partial safety factor for load
	- Deal load : $\gamma_f = 1.40$
	- Imposed (live load) : $\gamma_f = 1.60$
Ø	: Bar diameter
S =	: Xu Position of neutral axis
So	: Clear span
S	: distance center to center between stirrups
P_s	: Soil bearing capacity
e	: eccentricity
Qf	: Punching shear force in foundation
Nf	: load transmitted by the column to footing of foundation
Δq	: balanced soil pressure
Ab	: Average lateral area of the punching pyramid
Um	: Average perimeter of punching pyramid
NC	: Characteristic load transmitted by the column to the foundation
ξR	: 0.559
ρ_{rc}	: Specific weight of reinforced concrete
ρ_{cp}	: Specific weight of cement plaster
ρ_{mw}	: Specific weight of masonry wall
TMT	: three moments theorem
q_{sw}	: shear force carried by stirrups
φbf	: coefficient for the ordinary concrete
HA : HR	: Hot rolled high yield bar
XL	: Length centroidal distance
XR	: Right centroidal distance
R	: Mild steel

D : Overall depth

b_f : Width of flange

h_f :Depth of flange

$A_{s\,(prov)}$: Area of steel provided

$A_{s\,(reqd)}$: Area of steel required

Asw : Area (cross section) of one leg of stirrup

C/c : Center-to-center

Rb : concrete compressive design strength (= 1.40 KN/cm^2)

Rbt : concrete tensile design strength (= 0.09KN/cm^2)

Rs : Steel design strength (= 40 KN/cm^2)

RSC : Design steel compressive force

Nb : Rb* Abc : Resultant compressive force carried by concrete

NS' : RSC*As' : Resultant compressive force carried by reinforcement

NS : RSC*AS : Tensile force carried by reinforcement

Abt : concrete tensile area (to be neglected)

Abc : Concrete compressive area

Ab : cross section area of the column

Xu : location of neutral axis

ho : Effective depth of the cross section : 0.8s : compressive concrete depth :

Qsinc : Total vertical component of the shear force carried by all inclined bars at the distance Co = Shear force carried by bent up bars

Co : Projection of stirrups

Qsw : shear force carried by stirrups = Σ Rsw * Asw

Rsw : 0.8 Rs: Design strength of the stirrups and the inclined bars

Qb : shear force carried by concrete in the compression area

Qmax : QD : Maximum shear force in the beam

λ : $\frac{lo}{a}$: Slenderness ratio of column

φ : coefficient taking into account the slenderness ratio of column and the construction inaccuracies

βsx : Short span coefficient in slab design

F/c : Footing under column

FBM : Free Bending Moment

FEM : Fixed End Moment

II. MATERIAL STRENGTHS

1. (Cube strength of concrete (f_{cu}) = 25N/mm^2
2. Density of concrete ($\gamma_{concrete}$) = 24KN/m^3
3. Characteristic strength of reinforcement (f_y) = 250N/mm^2 (Mild steel)
4. Characteristic strength of reinforcement (f_y) =460N/mm^2 (High yield steel)

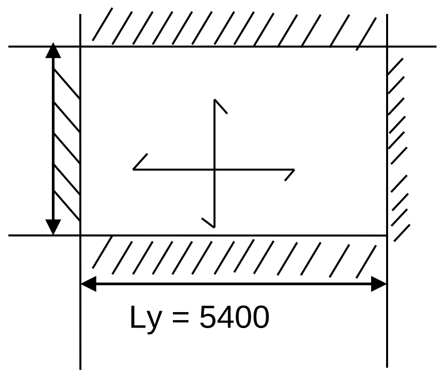
III. EXPOSURE CONDITIONS (According to BS code of Practice for structural use of concrete 2013)

1. Fire resistance of 1.5hrs for all members
2. Members in contact with soil : 50mm cover for very severe conditions
3. Members not in contact with soil : 30mm cover for very severe conditions
4. 25mm cover for staircase members.
5. 30mm cover for the beam
6. 50mm cover for the footing of foundation
7. 25mm cover for slab

IV. REINFORCED CONCRETE DESIGN

Ref	Calculation	Output
BS8110 2013	**4.1 DESIGN OF SLAB** ***Durability and fire resistance*** Nominal cover for very moderate conditions of Exposure = 25mm Nominal cover for 1.5 hours fire resistance =20 mm Since 25>20, provide nominal cover 25mm	Provide nominal cover = 25mm
	<u>Preliminary sizing of slab</u> $\frac{lx}{40} \leq ho \leq \frac{lx}{25} = \frac{530}{40} \leq h \leq \frac{530}{25}$ $13.25 \leq h \leq 21.20$ Taken h = 15cm Effective depth in all direction of the slab ho = 15cm – 2.5cm = 12.5cm	h=15cm ho=12.5 cm

REINFORCED CONCRETE DESIGN

Ref	Calculation	Output
	➢ **Sketch** **Lx=**5300  Ly = 5400mm Lx= 5300 mm λ= Ly/Lx= 5400/5300= 1.02< 2 Hence slab is designed as two ways span with four edges continuous. ➢ **Loading** -Self weight of slab 1.40*0.15*1*1*24= 5.04KN/m^2 -Finishes =1.40*1.5=2.1KN/m^2 Total dead load =7.14KN/m^2	The panel choosen is the slab with the largest side in order to obtain the greatest thickness of the slab (h=15cm) Total dead load = 7.14KN/m^2

REINFORCED CONCRETE DESIGN

Ref	Calculation	Output
BC Standard 2013	Design live load for residential house = 1.60*1.50KN/m^2 = 2.40kN/m^2 Design load (n) = 7.14KN/m^2 + 2.40kN/m^2=9.54KN/m^2 For a 1m width, n=9.54 KN/m^2 (n=Total distributed load on the slab panel) <u>Bending moment in slab</u> (See on pages 25) Table 6.6 $\lambda = \frac{Ly}{Lx} = \frac{540}{530} = 1.02 \approx 1.10$ ➢ **Short span** **Positive moment at mid span** Msx1 = βsx$_1$*n*lx^2, βsx1=0.024 Msx$_1$= 0.024*9.54*5.30*5.30 =6.43KNm **Negative moment at continuous edge** Msx2=βsx2*n*Lx^2, βsx2 = 0.031 = 0.031*9.54*5.30*5.30=8.31KNm ➢ ***Long span*** - ***Positive moment*** Mid span moment Msx1= βsy1*n*ly^2, βsy1= 0.024 = 0.024*9.54*5.40*5.40=6.68KNm (bending moment with deflection) - **Negative moment at** Continuous edge moment Msx2= βsy2*n*ly^2, βsy2=0.032 = 0.032*9.54*5.40*5.40 = 8.90 KNm ***<u>Reinforcement Analysis</u>*** ***Effective depth = ho = 15cm -2,5 cm = 12.50 cm*** ***a.*** Required steel at the top $\propto m = \frac{Mmax}{Rb* b * ho^2} = \frac{8.90KNm \times 100}{1.40*100*(12.50)^2} = 0.041$ $\propto m$ = 0.0.41 ⟹ ξ = 0.04; ⟹ = n = 0.980 (see table of coefficients relative to the design of members subjected to bending moment page 65)	Qk=2.40KN/m^2 n=9.54KN/m^2 Negative Mmax = 8.90 KN.m Positive Mmax = 6.68 KN.m

	$\bar{A}s = \frac{Mmax}{n * Rs * ho} = \frac{8.90*100}{0.890*40*12.5} = 0.041$ $\bar{A}s$ = 1.816 cm^2 = 3 ø 10=2.36 cm^2 (not sufficient) Taken 5 ø14/m e.g: ø14/20cm (5 bars min/m in slab), because in general the minimum bars required per meter in the slab is taken as 5Ø12	

REINFORCED CONCRETE DESIGN

Ref	Calculation	Output

b) **Required Steel at the bottom**

$$\propto m = \frac{Mmax}{RS * b * ho^2} = \frac{6.68 \times 100}{1.40 * 100 * (12.5)^2} = 0.031$$

$\propto m$ = 0.031 ξ = 0.03; n= 0.985 Singly reinforced section

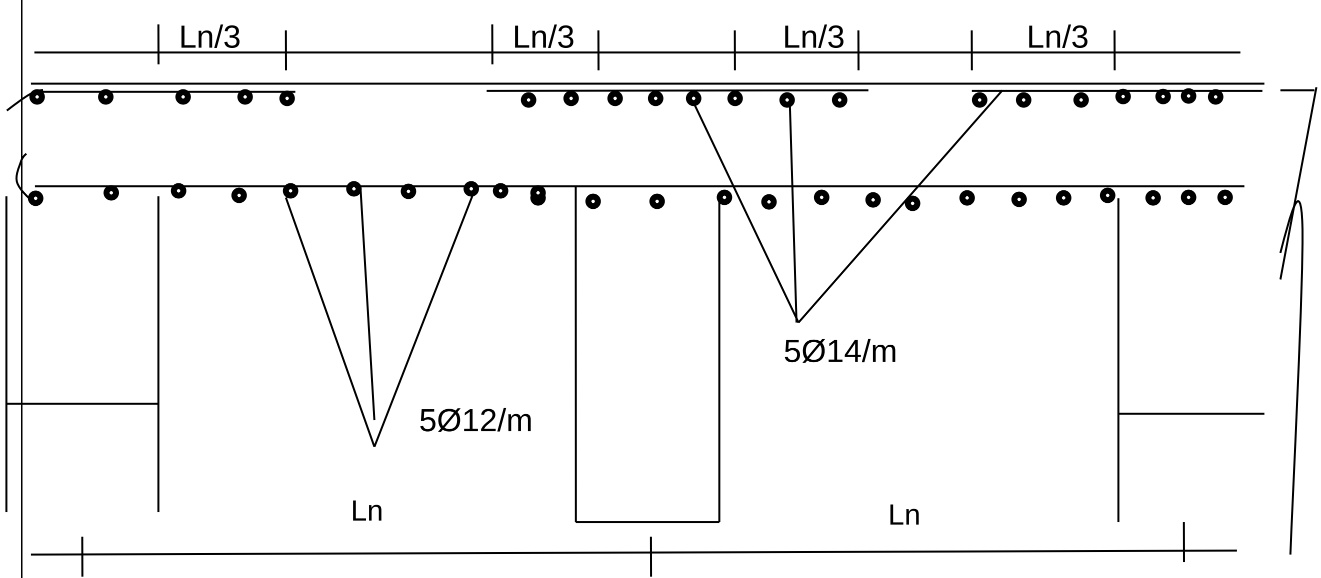
$$As^+ = \frac{Mmax}{n*RS*ho} = \frac{6.68 * 100}{0.985*40*12.5} = 1.356 \ cm^2$$

As^+= 1.356 cm^2 = 135.6 mm^2 = 2ø10/m = 1.57 cm^2 (not sufficient)

Taken 5ø12/m provide ø12/20cm (5 bars min / m in slab), because in general, the minimum bars required per meter on the slab is taken as 5ø12.

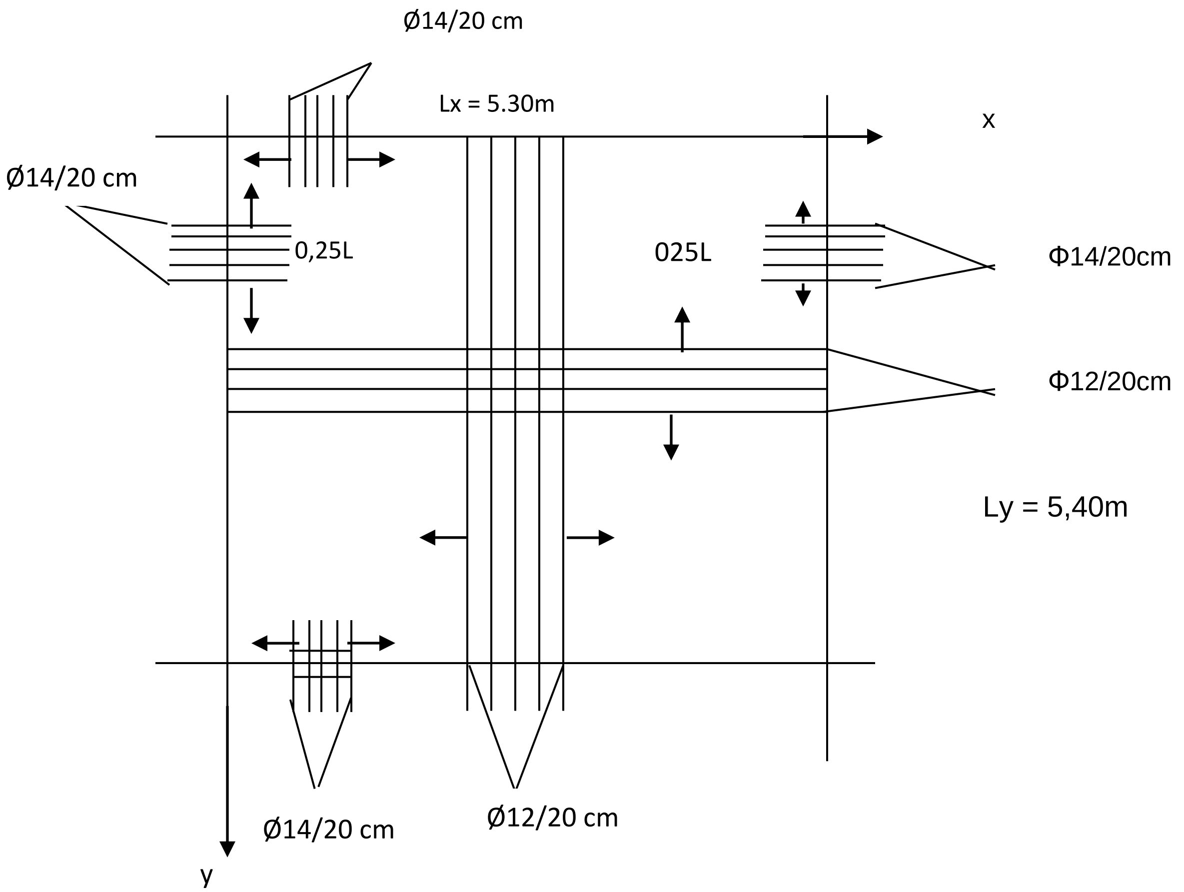
<u>ARRANGEMENT OF STEEL REINFORCEMENT IN THE SLAB</u>

a) **<u>Transverse cross section</u>**



Ref	Calculation	Output

B. Plan view cross section

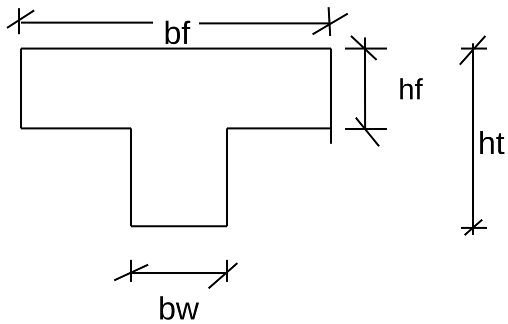


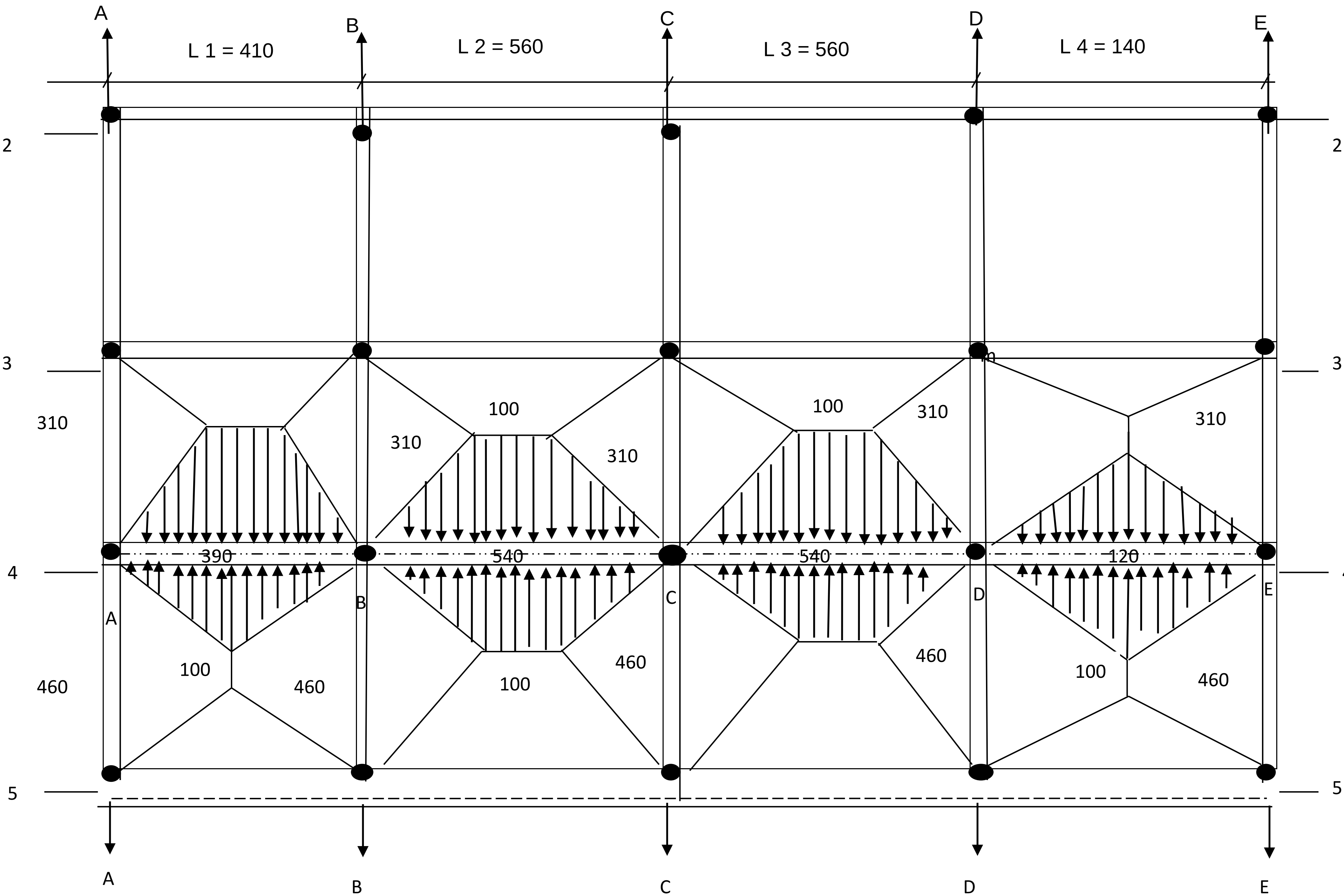
REINFORCED CONCRETE DESIGN

Ref	**4.2. DESIGN OF BEAM**	**Output**
	- The total height (ht) of the beam has to be in the range below : $\frac{\ell y}{15} \leq ht \leq \frac{\ell y}{8} = \frac{540}{15} \leq ht \leq \frac{540}{8} = 36 \leq ht \leq 68$ Taken ht = 50 cm - The Breadth of the section (bw) of the beam has to be in the range below : $0.50 < \frac{b}{ht} < 1 = 0.50 = \frac{b}{50} = b = 25$ cm Taken : b = 30 cm - The flange (bf') of the beam has to be the lesser of a) $\frac{\ell y}{3} = \frac{540}{3} = 180$cm b) $\frac{\ell x}{2} = \frac{530}{2} = 256$ cm c) 12h+b = (12 * 15) + 30 = 210 cm d) Taken bf' = 180 cm	ht : 50 cm bw = 30 cm bf' = 180 cm

Note: The design consider the most unfavorable case of a most loaded beam. The final dimensions thus obtained will be applied to the set of the remaining beams, in order to reduce the volume of tedious calculations.

REINFORCED CONCRETE DESIGN

Ref	Calculation	Output
BSS8110	**4.2.1. <u>Dimensions of the beam</u>** **(T. section)** **Sketch**  	$bf' = 180cm$ $bw = 30cm$ $hf = 15cm$ $ht = 50cm$


A
B
C
D
E
L 1 = 410
L 2 = 560
L 3 = 560
L 4 = 140
2
3
4
5
310
100
390
540
120
460
A
B
C
D
E

4.2.2. <u>Calculation of Areas of influence on the beam 4 – 4</u>

A1a = $(\frac{390+100)\ x155}{2}) = 3.80\ m^2$

A1b = $(\frac{390\ x\ 180}{2}) = 3.51\ m^2$

A2a = $(\frac{540\ +\ 100}{2})\ x\ 155 = 4.96\ m^2$

A2b = $(\frac{540\ +100)\ x\ 230}{2}) = 7.36\ m^2$

A3a = $(\frac{540\ +100)\ x\ 155}{2}) = 4.96\ m^2$

A3b = $(\frac{540\ +100)\ x\ 2.30}{2}) = 7.36\ m^2$

A4a = $(\frac{120\ \ +155)}{2}) = 0.93\ m^2$

A4b = $(\frac{120\ x\ 1.80)}{2}) = 1.08\ m^2$

Summary

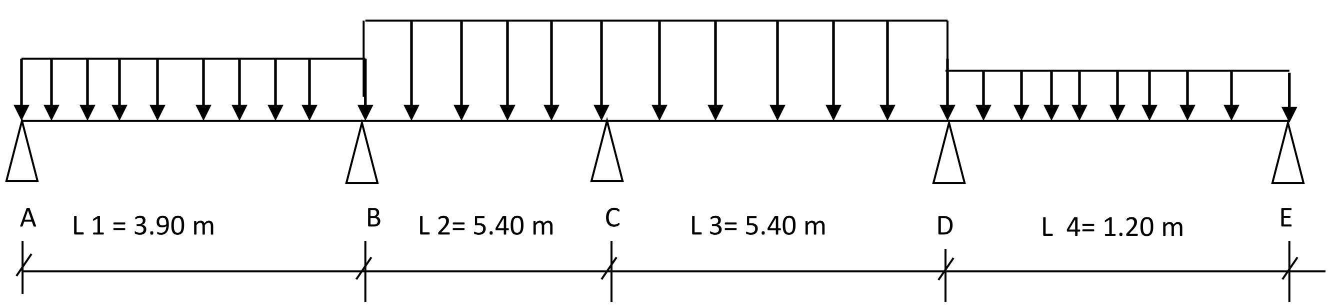
A1 = 3.80 m^2 + 3.51 m^2 = 7.31 m^2

A2 = 4.96m^2 + 7. 36 m^2 = 12.32m^2

A3 = 4.96 m^2 + 7. 36m2 = 12.32 m^2

A4 = 0.93m^2 + 1.08 m^2 = 2.01 m^2

4.2.3. static calculation chart



4.2.4. Calculation of dead load on the beam 4 – 4 axis

a) <u>Span AB</u>

- Self weight of slab span = 7.31m^2 * 0.125m * 24KN/m^3 = 15.45KN
- Finishes = 7 .31m^2 *0.50KN/m^2 = 3.66 KN

  Where 0.50KN/m^2 is permanent load for finishes (guide de calcul page 25)
- Cement plaster on the beam span = 0.03m*1.00m*3.90m*20KN/m^3 =2.34KN, where 20KN/m^3 is the specific weight for the cement plaster and 0.03m is the cover of cement on beam.
- Self weight of beam span AB = (0.50m– 0.125m)*3.90m*0.30m*24KN/m^3

  = 0.375m*3. 90m*0.30m*24KN/m^3

  = 10.53 KN
- Maconnery wall = 0.20m*2.65m*3.90m*18KN/m^3=37.21KN

  Where 2.65m is effective height of the wall

  And 18KN/m^3 is specific weight for the maconnery wall
- Total dead load on span AB = 15.45KN + 3.66 KN + 2.34 KN +10.53KN+37.21KN = 69.19 KN

b) <u>Dead load on Span BC</u>

- Self weight of slab span = 12.32m^2 * 0.125m*24KN/m^3 = 36.96 KN
- Finishes: 12.32m^2 * 0.5KN/m^2 = 6.16 KN
- Cement plaster on the beam span = 0.03m*1.00m*5.40m*0.30m*20KN/m^3 = 0.97KN
- Self weight of beam span BC = (0.50mm – 0.125m) * 5.40m * 0.30 * 24KN/m^3

  = 0.375m*5.40m*0.30m*24KN/m^3 =14.58 KN
- Maconnery wall = 0.20m*2.65m*5.40m*18KN/m^3 = 51.52KN

Total dead load on span BC = 36.96 KN + 6.16 KN + 0.97KN + 14.58 KN + 51.52KN

= 110.19 KN

c) **<u>Dead load on Span CD</u>** (same calculation like on span BC)

- Total dead load on span CD = 110.19KN

d) <u>Dead load on Span DE</u>

- Self weight of slab on span DE = 2.01m^2 * 0.125m^2 * 24KN/m^2 = 6.03KN

- Finishes = 2.01m^2 * 0.50KN/m^2 = 1.01KN

- Cement plaster on the beam span = 0.03m^2*1.00m*1.20m*0.30m*20KN/m^2=0.22KN

- Self weight of beam span DE = 0.375m*1.20m*0.30m*24KN/m^3=3.24KN

- Maconnery wall on the beam span = 0.20m*2.65m*1.20m*18KN/m^2=11.45KN

Total dead load on span CD = 6.03 KN + 1.01 KN + 0.22 KN + 3.24KN + 11.45KN

= 21.95 KN

4.2.5. Calculations of dead load / m on the beam 4 - 4

- Span AB = 69.19 KN/ 3.90 m = 17.74 KN/m
- Span BC = 110.19 KN/ 5.40 m = 20.41 KN/m
- Span CD = 110.19 KN/ 5.40 m= 20.41 KN/m
- Span DE = 21.95 KN/ 1.20 m = 18.29 KN/m

Calculations of live load on the beam type 4 – 4

Because of the purpose of the building, we assume that the live load is taken as 1.50 KN/m^2

4.2.6. Calculations of combination of load (Design load)

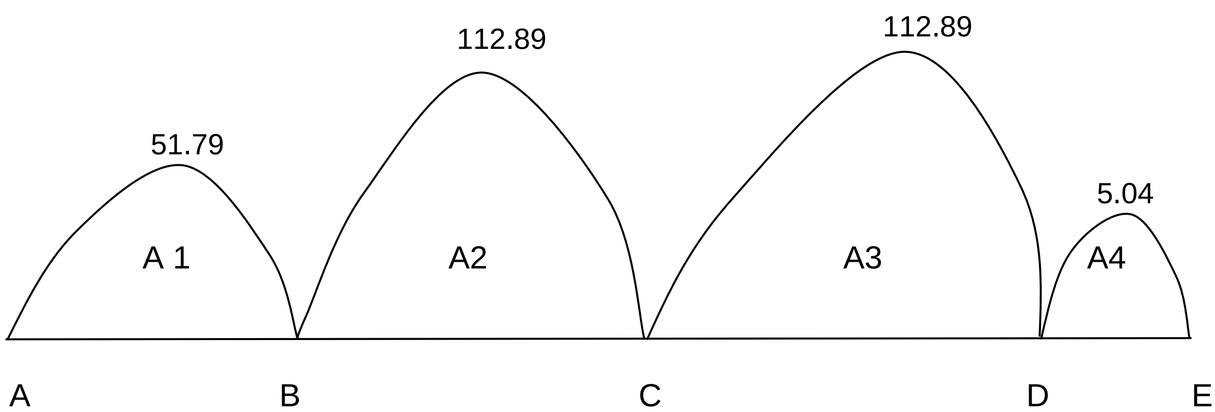
- ✓ Span AB = (1.40 * 17.74) + (1.60 x 1.50) = 27.24 KN /m
- ✓ Span BC = (1.40 * 20.41) + (1.60 x 1.50) = 30.97 KN /m
- ✓ Span CD = (1.40 * 20.41) + (1.60 x 1.50) = 30.97 KN /m
- ✓ Span DE = (1.40 * 18.29) + (1.60 x 1.50) = 28.01 KN /m

4.2.7. Calculation of bending moments and shear forces using CLAPEYRON's three moments theorem method (TMT method)

1. Calculation of free bending moment at mid spam

- Span AB = $\frac{w\ell^2}{8} = \frac{27.24KN * 3.90 * 3.90}{8}$ = 51.79KN.m
- Span BC = $\frac{w\ell^2}{8} = \frac{30.97KN * 5.40 * 5.40}{8}$ = 112.89KN.m
- Span CD = $\frac{w\ell^2}{8} = \frac{30.97KN * 5.40 * 5.40}{8}$ = 112.89KN.m
- Span CD = $\frac{w\ell^2}{8} = \frac{28.01KN * 1.20 * 1.20}{8}$ = 5.04KN.m

2. Free bending moment diagram



3. calculation of areas of the free bending moment

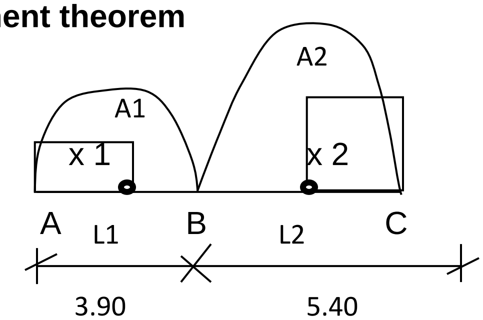
- Span AB = $\frac{2}{3}$ x 3.90 * 51.79 = 134.65m^2 = A1
- Span BC = $\frac{2}{3}$ x 5.40 * 112.89 = 406.40m^2 = A2
- Span CD = $\frac{2}{3}$ x 5.40 *112.89 = 406.40m^2 = A3
- Span DE = $\frac{2}{3}$ x 1.20 * 5.04 = 4.03m^2 = A4

4. Calculation of centroidal distance of the fixed bending moment diagram (x l and xr)

- ❖ Span AB = X L = X R = 3.90/2 = 1.95 m
- ❖ Span BC = X L = X R = 5.40/2 = 2.70 m
- ❖ Span CD = X L = X R = 5.40/2 = 2.70 m
- ❖ Span DE = X L = X R = 1.20/2 = 0.60 m

5. Applying CLAPEYRON'S three moment theorem

- ❖ Spans AB & BC (beam A – B - C)



Three moment equation is:

$$(MA * L1) + 2\ MB\ (L1 + L2) + (MC * L2) + \frac{6 * A1 * x1}{L\ 1} + \frac{6 * A2 * x2}{L2} = 0 \quad (1)$$

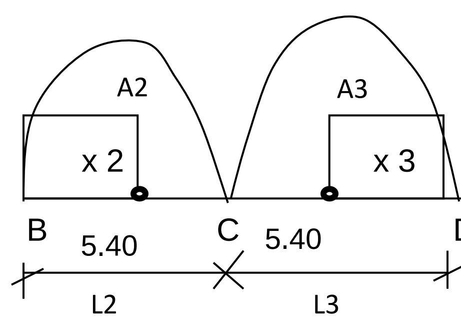
Considering that MA = 0

$\Rightarrow$ $0 + 2\ MB\ (3.90 + 5.40) + (MC * 540) + \frac{6 \times 134.65 \times 1.95}{3.90} + \frac{6 \times 406.40 \times 2.7}{5.40} = 0$

$\Rightarrow$ 18.6 MB + 5.40 MC + 403.95 + 1219.20 = 0

$\Rightarrow$ 18.6 MB + 5.40MC = - 1623.15 (1)

- ❖ Spans BC & CD (beam B – C-D)



Three moment equation is:

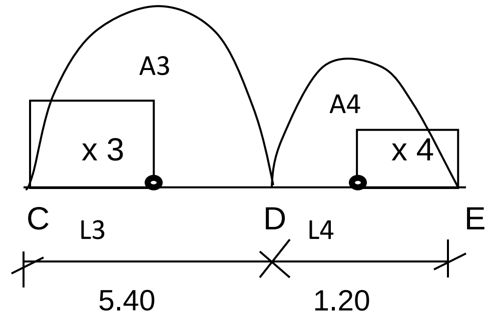
$$(MB \times L2) + 2\ MC\ (L2 + L3) + (MD \times L3) + \frac{6 * A2 * x\ 2}{L\ 2} + \frac{6 \times A3 * x3}{L3} = 0 \quad (2)$$

$\Rightarrow$ $5.40\ MB + 2MC(5.40 + 5.40) + (MD \times 5.40) + \frac{6 \times 406.40 \times 2.70}{5.40} + \frac{6 * 406.40 * 2.70}{5.40} = 0$

$\Rightarrow$ 5.40 MB + 21.60MC + 5.40MD = - 1219.2 – 1219.20

$\Rightarrow$ 5.406 MB + 21.60 MC + 5.40 MD = -2438.40 (2)

- ❖ Spans CD & DE (beam C-D.E)



Three moment equation is:

$$(MC \times L3) + 2\,MD\,(L3 + L4) + (ME \times L4) + \frac{6 * A3 * x3}{L3} + \frac{6 * A4 * x4}{L4} = 0 \quad (3)$$

Considering that $ME = 0$

$$\Rightarrow 5.40MC + 13.20\,MD + 0 + \frac{6*406.40*2.70}{5.40} + \frac{6*4.03*0.60}{1.20} = 0 \quad (3)$$

$$\Rightarrow 5.40MC + 13.20\,MD + 0 + 1219.20 + 12.09 = 0$$

$$\Rightarrow 5.40\,MC + 13.20\,MD = -1231.29 \quad (3)$$

<u>Solving the system of three equations with three unknowns below</u>

	End fixed moments are:
$18.6\,MB + 5.40MC = -1623.15$ (1)	$MA = 0$
$5.40\,MB + 21.60MC + 5.40\,MD = -2438.40$ (2)	$MB = -63.43$ KNm
$5.40\,MC + 13.20\,MD = -1231.29$ (3)	$MC = -82.11$ KN.m
	$MD = -59.69$ KN
	$ME = 0$

In solving the system, we use the unknown elimination method, thus:

$$\left\{\begin{array}{l} 18.60MB+5.40MC=-1623.15 \quad (1) \\ 5.40MB+21.60MC+5.40MD=2438.40 \quad (2) \\ \underline{\qquad\qquad +5.40MC+13.20MD=-1231.29} \quad (3) \end{array}\right\}$$

$(18.60MB+5.40MC=-1623.15)*1$

<u>$(5.40MB+21.60MC+5.40MD=-2438.40)*-3.445$</u>

Let eliminate MB

⇨ +18.60MB + 5.40MC= -1623.15

-18.60MB - 74.412MC - 18.603MD = +8400.288

⇨ 0MB - 69.012MC – 18.603MD = +6777.138 (2)

+5.40MC+13.20MD = 1231.29 (3)

Let eliminate MC

(-69.012MC-18.603MD=+6777.138)*1

(+5.40MC+13.20MD=-1231.29)*12.78

⇨ -69.012MC-18.603MD=+6777.138

+69.012MC+168.696MD=-15735.8862(3)

⇨ 0MC + 150.093MD=-8958.7481

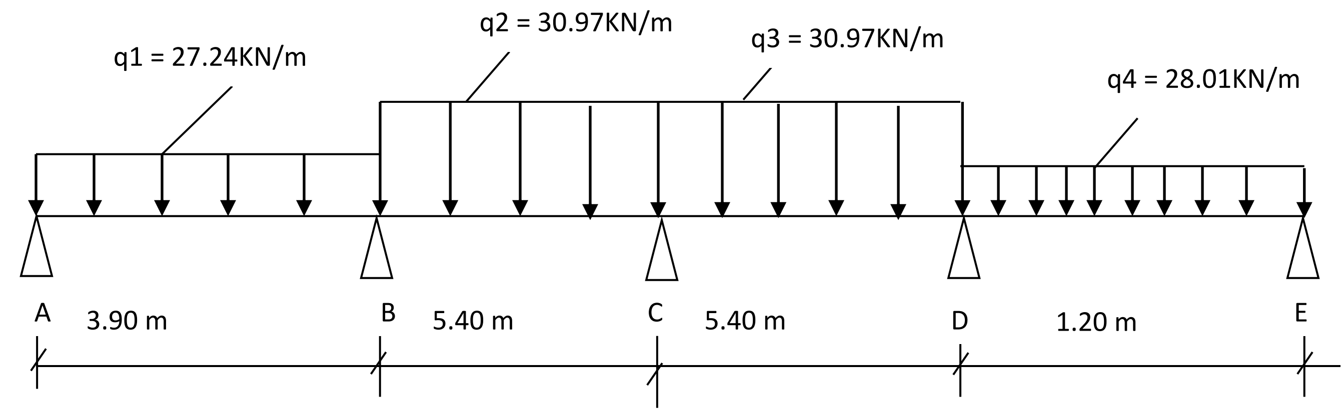
⇨ MD= $\frac{-8958.7481}{+150.093}$

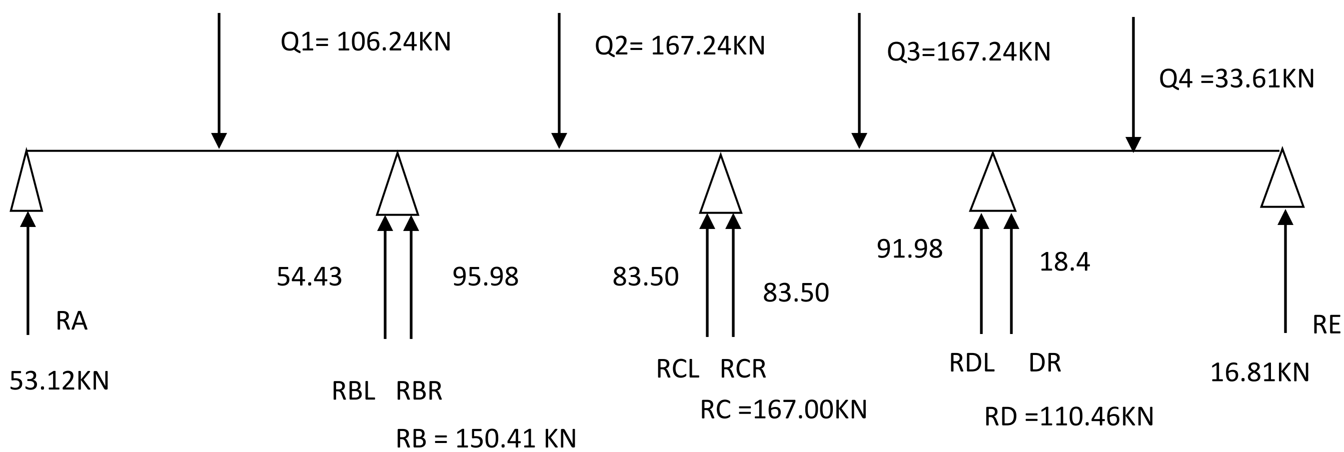
⇨ **MD = -59.69KN.m**

- According to equation nº3, we calculate MC value
  - +5.40MC + 13.20MD=-1231.29 (3)
  - +5.40MC+(13.20*-59.69)=-1231.29(3)
  - +5.40MC-787.908=-1231.29
  - +5.40MC=-1231.29+787.908
  - +5.40MC=-443.382
  - MC=-$\frac{443.382}{5.40}$
  - **MC=-82.11KN.m**
- According to equation nº1, we calculate MB value
  - 18.60MB+5.40MC=-1623.15 (1)
  - 18.60MB+(5.40*-82.11)=-1623.15 (1)
  - 18.60MB+(5.40*-82.11)=1623.15(1)
  - 18.60MB-443.394=-1623.15
  - 18.60MB=-1623.15+443.394
  - 18.60MB=-1179.756
  - MB = -$\frac{1179.756}{18.60}$ = - 63.43KN.m
  - MB = -63.43KN.m

6. Calculation of support reactions

According to the standard of BAEL 91 modified 99 by professor engineer Jean Pierre Mougin, on page 108, we can to calculate the support reactions as follows:





RA = 106.24 KN / 2 = 53.12 KN

RB = (RBL + RBR) + 0.1 (RBL + RBR) $= \left(\frac{106.24}{2} + \frac{167.24}{2}\right) + 0.1 \left(\frac{106.24}{2} + \frac{167.24}{2}\right)$

= (53.12 + 83.62) + 0.1 * (53.12 + 83.62)

= 136.74 + 13.67

RB = 150.41 KN

RC = RCL + RCR = $\frac{167.24}{2} + \frac{167.24}{2}$ = 83.50 + 83.50 = 167.00 KN

RD = (RDL + RDR) + 0.1 * (RDL + RDR)= $\left(\frac{167.24}{2} + \frac{33.61}{2}\right) + 0.10 \left(\frac{167.24}{2} + \frac{33.61}{2}\right)$

RD = (83.62 +16.81) * 0.10 (83.61 + 16.81)

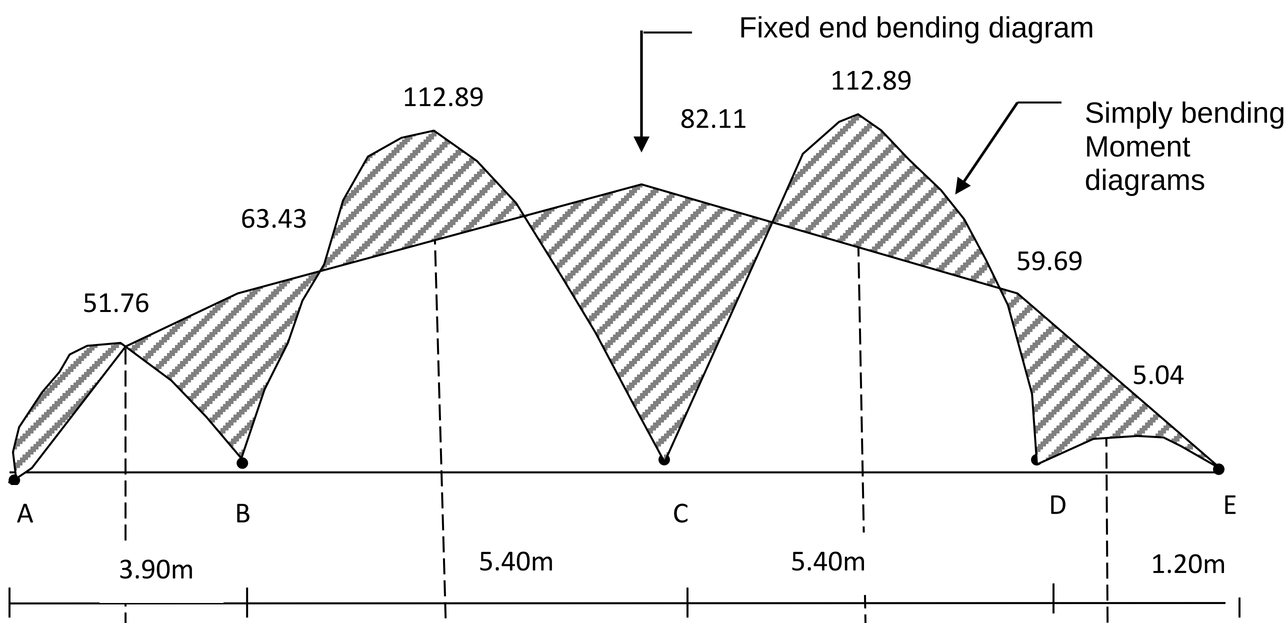
= 100.42 + 10.04

RD = 110.46 KN

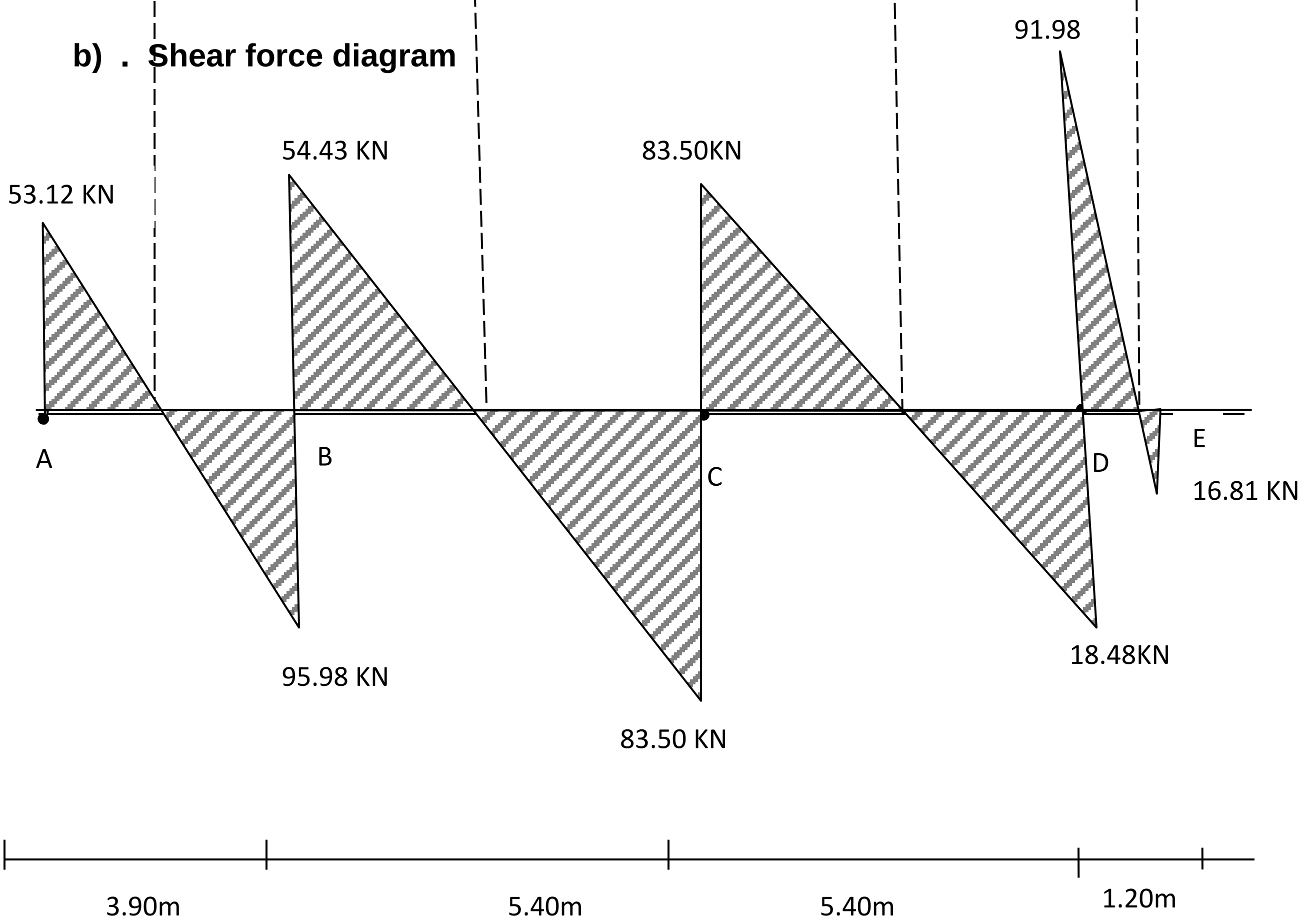
RE = $\frac{33.61}{2}$ = 16.81 KN

7. Bending moments and shear forces diagram

a) Combined bending moment diagrams



b) . Shear force diagram



Summary

Maximum hogging moment: 82.11 KN.m (Mmax)$^-$

Maximum sagging moment: 112.89 KN (Mmax)$^+$

Maximum shear force: 167.00 KN (Vmax)

4.2.4. Required steel reinforcement in the beam type along 4 – 4 axis

❖ Effective depth (ho) = 50 cm – 3.0 cm = 47cm

a) Required steel at the top or support reinforcement

$$\propto m = \frac{\overset{-}{Mmax}}{Rb * b * ho^2} = \frac{82.11 \times 100}{1.40 * 30 * 47 * 47} = 0.089$$

$\propto m$ = 0.089 $\Longrightarrow$ ξ = 0.09 and n = 0.955

ξ = 0.9 < ξ R = 0.559 $\Longrightarrow$ The T section is singly reinforced

Thus x = ξ x ho = 0.09 x 47 < (35 cm = $ht - hf$) = 0.50-0.15)

Where hf is the thickness of the flange of the T section? For that the compression area is reinforced, we have:

$$As^- = \frac{\overset{-}{Mmax}}{n * RS * ho} = \frac{82.11 * 100}{0.955 * 40 * 47} = 4.57 \text{ cm}^3$$

Taken = 3 Ø 16 = 6.03 cm^2

b) Required steel at the bottom or mid span reinforcement

$$\propto m = \frac{\overset{+}{Mmax}}{Rb * bf * ho^2} \quad \frac{112.89 \times 100}{1.40 * 180 * 47 * 47} = 2.028$$

$\propto m$ = 0.028 $\Longrightarrow$ ξ = 0.36 and n = 0.820

ξ = 0.36 < ξ R = 0.559 $\Longrightarrow$ The T section is singly reinforced

$$As = \frac{112.89 \times 100}{0.820 * 40 * 47} = 7.32 \text{ cm}^2$$

Taken 4 Ø 16 = 8.04 cm 2

c) Design of stirrups or shear reinforcement

Vmax (Maximum shear force) = 167.00 KN

qsw = shear force carried by stirrups

$$qsw = \frac{(Vmax)^2}{4\varphi bf * Rbt * bw * ho2}$$

Where φbf = 1.50

Rbt = 0.09 KN/cm^2

$$qsw = \frac{(167.00)^2}{4 * 1.50 * 0.09 * 30 * 46.5 * 46.5} = \frac{27.889}{35785.80} = 0.779KN/cm^2$$

- ❖ Let us use stirrups of Ø 8 ⟹ Asw = 50.3 mm^2 = 0.503 cm^2

⟹ Rsw = 0.8 * Rs = 0.8 * 40 KN / cm^2 = 3.20 KN / cm^2

- ❖ **Distance between stirrups (S)**

$$S = \frac{Rsw * Asw * n}{qsw}$$, where n = number of legs for stirrup

$$S = \frac{0.8 * 40KN/cm2 * 0.503\ cm^2 * 2}{0.779KN/Cm^2} = 41.32cm$$

<u>Note:</u> The distance between stirrups must be lesser than the three following values

1) $$Smax = \frac{0.75\ \varphi bf * Rbt * b * ho^2}{Vmax}$$

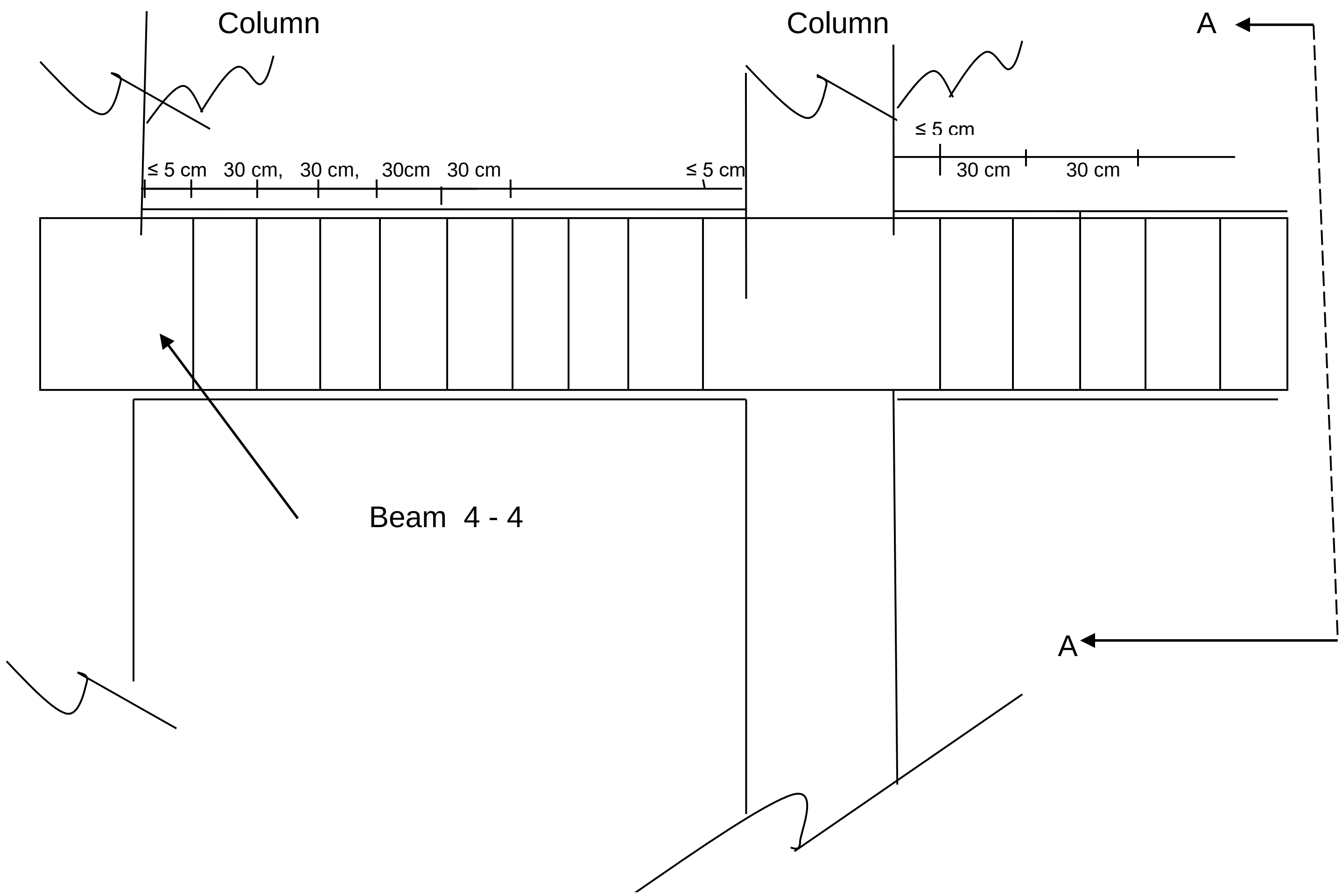
$$= \frac{0.75 * 1.50 * 0.09 * 30 * (47)^2}{Vmax} = \frac{6709.8375}{167.00} = 40.18cm$$

2) The width of the beam web = bw =30 cm
3) 30 cm

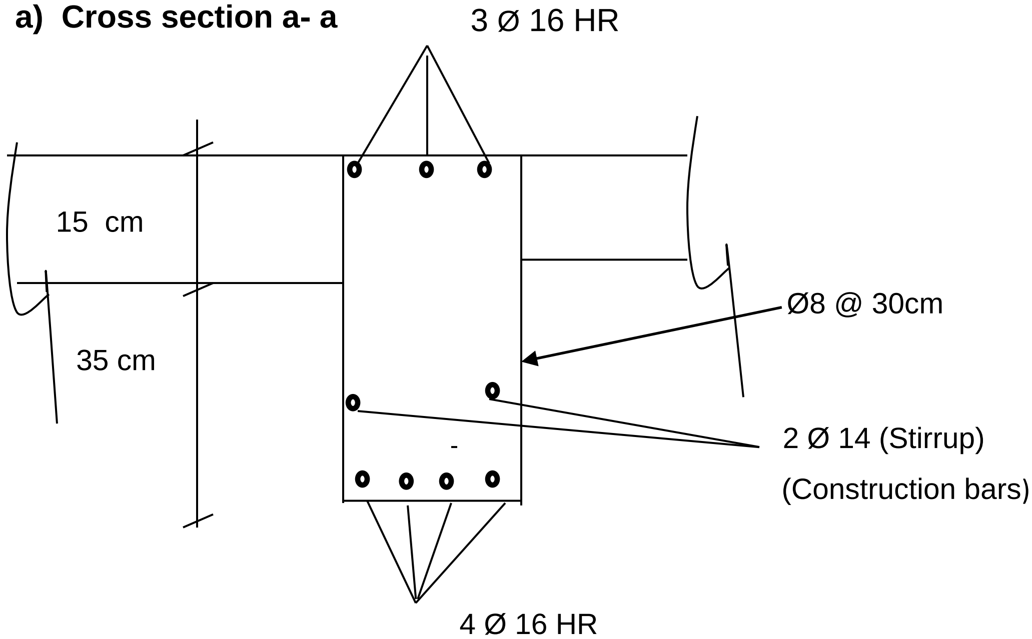
Thus S = Min $\left\{ 40.18cm; 30 cm; 30 cm \right\}$ = 30 cm Taken: Ø 8 @30 cm

4.2.5. Arrangement of steel reinforcement in the beam

a) Longitudinal section



a) Cross section a- a



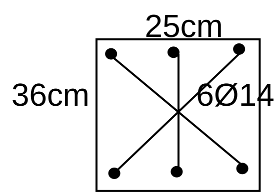
4.3. DESIGN OF TIE BEAM

Without calculation, the theory assumes that the cross section of the tie beam or plinth beam, have to be in the following range.

a) $h = \frac{L}{15} \sim \frac{L}{10}$,where L=greatest span beam
b) $b \geq 20cm$, where b=width of the tie beam
c) $As = 1\%$ of cross section of the tie beam. Therefore cross section characteristics are:
    a) $h = \frac{540}{15} \sim \frac{540}{10} \gg h = 36cm \sim 54cm$
    b) $b \geq 20cm, where\ b = width\ of\ the\ tie\ beam$
    c) As=1%*25*36cm=9.00cm^2 (minimum value)
    As=9.00cm^2=6Ø14=9.24cm^2

    Provide 6Ø14=9.24cm^2

SKETCH



4.4. DESIGN OF THE LINTEL BEAM

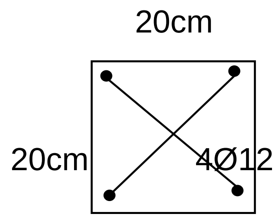
Without also calculation, the theory assumes that the cross section of the lintel beam have to be in the following range.

a) h ≥ 20cm
b) b = thickness of the wall maconnary
c) As = the cross section of the lintel beam

Therefore, cross section of the lintel are:

a) h=20cm
b) b=20cm
c) As=1%*20cm*20cm=4cm^2

SKETCH



REINFORCED CONCRETE DESING

Ref	Calculation	Output
BS 8110	**4.3. COLUMN DESIGN ANALYSIS C2** Clear height of ground floor column = 300 cm **End conditions** **Condition at top** End of column is connected monolithically to beams on either side and are at least as deep at the overall **Condition at bottom** End of column is connected monolithically to beams or to footing on either side and are at least as deep as the overall Dimension of the column. In general minimum cross section is assumed to be Ab=25cm*25cm a= 250 mm b = 250 mm β = 0.7 (braced column) H = Total height of column Effective height of column = $\ell o = \beta$ x H ℓo = 0.70 X 3.0m = 210 cm λ (slenderness ratio) : $\frac{\ell o}{a} = \frac{210\ cm}{25\ cm} = 8.40$ ⇨ φ = 0.91 Hence column is to be designed as short braced axially loaded column (short column)	

4.3.1. <u>Loads on the column c2</u>

a) Column loading area = $\left(\frac{3.10}{2} * \frac{5.40}{2}\right) * 2 = 8.37\ m^2$

$+ \left(\frac{5.40}{2} * \frac{4.60}{2}\right) * 2 = 12.42\ m^2$

Column loading area = $8.37m^2 + 12.42m2 = 20.79m^2$

b) Slab (permanent load) = $7.14\ KN/m^2 * 20.79m^2 = 148.44$ KN

c) Live load from the slab = $1.50\ KN / m^2 * 20.79\ m^2 = 31.19$ KN

d) Load from beam (0.30 * 0.50 * 1.40 * 1 * 24) + (1.40 * 0.03 * 0.70 * 20)

= (0.59 + 5.40) = 5.99KN/m * 9.25 m = 55.41 KN

e) Wall and plaster $\left\{ \begin{array}{l} (1.40 * 0.20 * 3.00 * 1 * 18) + (1.40 * 0.03 * 3.00 * 20 * 2) \\ = (15.12\ KN / m + 5.04\ KN /m) * 9.25m = 186.48\ KN \end{array} \right\}$

f) On floor of column = 1.40 * 0.25 * 0.25 * 3.00 * 24 = 6.30 KN

g) Load from the light roof ≅ $\frac{\text{Permanent load from slab}}{2}$ =

$$= \frac{148.44\ KN}{2} = 74.22\ KN$$

4.3.2. Ground floor part of the column

$$N1 = \left[(148.44KN + 31.19KN + 55.41KN) * 1 + (186.48*2) \right] + (6.30\ KN * 2) + 74.22\ KN$$

N1 = 607.91 KN + 12.60KN+74.22KN = 694.81KN

4.3.3. Required steel reinforcement

$$AS = \frac{\frac{N}{\varphi} - RB * Ab}{RS} = \frac{\frac{694.81}{0.91} - 1.40 * 625}{40}$$

$$AS = \frac{694.81 - 875}{40} = -4.50cm^2$$

Negative sign indicate that compression steel reinforcement is not required because AS < 0

Therefore the theory assumes that the minimum percentage of steel reinforcement must be evaluated as follows

Asmin = 0.004 Ab

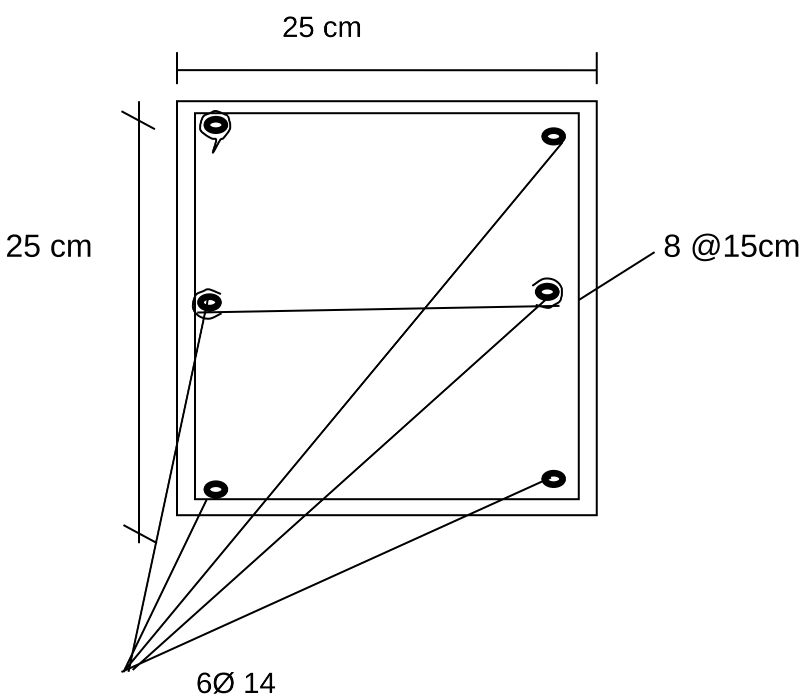
Asmin = 0.004 * 25 * 25 = 2.5 cm^2

Taken = 4 Ø 10 = 3.14 cm^2

But, because of the minimum diameter of bar in the column is assumed as Ø 12, we must use 6 Ø14

Thus we arrange the same steel up to the top floor

- Smin = distance between stirrups = 1 / 4 *12 mm = 3mm
- Smax = 12 * Ø max = 12 * 12 = 144 mm
- Taken 150mm = 15 cm

Ref	Calculation	Output
	Taken S = 15 cm Thus we use Ø 8 @ 15 cm **CROSS SECTION OF THE COLUMN C2** 	

Ref	Calculation	Output
	4.4. DESIGN OF PAD FOUNDATION **4.4.1. Soil bearing capacity** PS = 200 KN /m^2	

4.4.2. Characteristic load transmitted to the foundation

$$NC = \left\{ \left[\frac{148.44}{1.40} + \frac{31.19}{1.60} + \frac{55.41}{1.40} \right] * 1 + \frac{186.44}{1.40} \right\} *2 + \frac{6.30 \times 2}{1.40} + \frac{74.22}{1.40}$$

NC = (106.03 + 19.49 +39.57) * 1 + 266.34 + 9.00 + 53.01

NC = 165.09 + 266.34 + 9.00 + 53.01

NC =493.44KN

4.4.3. Weight of the foundation

$$\frac{NC}{10} = \frac{493.44\ KN}{10} = 49.34KN$$

4.4.4. Foundation base dimensions

$$Af = \text{Area of footing} = \frac{NC + NC/10}{PS} = \frac{493.44 + 51.31}{200}$$

$$= \frac{542.78KN}{200KN/m^2}$$

Af = 2.71m^2

af x bf = $\sqrt{2.71cm^2}$ = 1.65m ≈ 1.70 m

⟹ af = bf = 1.70 m   Af = 170 cm * 170 cm = 2.89m^2

4.4.5. Checking of the punching shear

❖ *Condition of no pu*nching shear:

Qf = Nf - Δq ≤ Rbt * Ab

Where : Qf : Punching shear force

N1 = Nf = load transmitted by the column to the foundation

Δq = Balanced soil pressure

Ab = Average lateral area of the punching pyramid

Um : Average perimeter of the punching pyramid

Rbt = Concrete tensile design strength = ($0{,}09$ KN / cm^2)

P = pressure = $\frac{\text{Force}}{\text{Area}} = \frac{F}{A} = \frac{N1}{Af} = \frac{694.81KN}{28900cm^2} =$

P = 0.024KN/ cm^2

af = bf = sides of footing

ac = bc = dimensions of cross section of column

ho = Effective depth of footing

- Let us take hf = 55 cm ⇨ $ho = hf$ - 5 cm

ho = 55 cm – 5 cm = 50 cm

$Um = 2\,(ac + bc + 2\,ho) = 2\,(25 + 25 + 2 * 50)$

Um = 300 cm

$Ab = Um * ho =$ 300 cm * 50 cm = 15000 cm^2

$\Delta q = P(\,ac + 2\,ho\,)(\,bc + 2\,ho)$

$\Delta q = 0.024\,KN/cm^2\,(\,25 + 2 * 50)\,(\,25 + 2 * 50)$

$\Delta q = 0.024 * 125 * 125$

$\Delta q = 375.00KN$

Thus : $Qf = Nf - \Delta q \le Rbt * Ab$

$Qf = 694.81KN - 375.00\,KN \le 0.9 * 15000$

$Qf = 319.81\,KN < 1350\,KN\ OK$

The condition is satisfaction ; thus No punching shear

4.4.6. Required steel reinforcement for the foundation

$$Maf = Mbf = \left(\frac{P * af}{2}\right) \left(\frac{bf - bc}{2}\right)^2$$

Where: Maf : Bending moment about side af of the Footing

Mbf : Bending moment about side bf of the footing

Thus : $Maf = Mbf = \left[\frac{0.024 * 170}{2}\right]\left[\frac{170 - 25}{2}\right]^2$

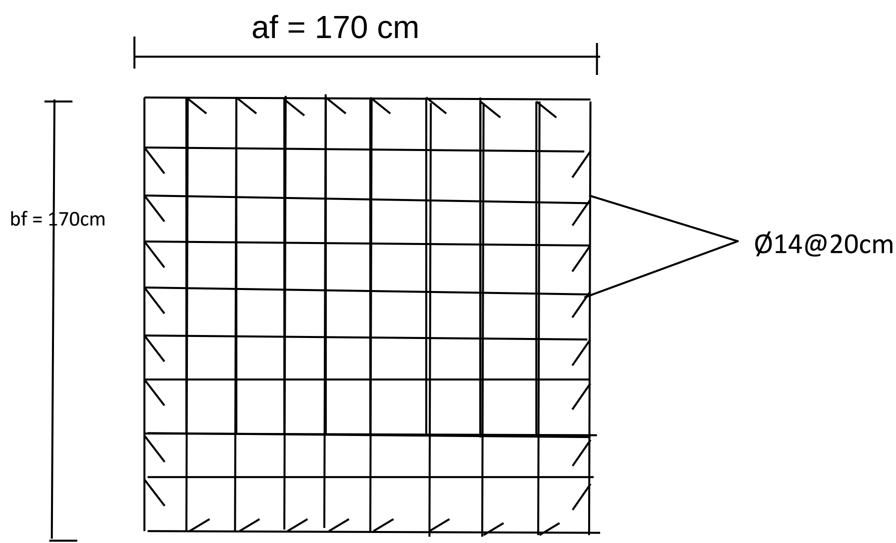
$= 2.04 * 5256.25 = 10722.75 KN.cm$

As = $\frac{Mmax}{0.9 * Rs * ho} = \frac{10722.75\ KN.cm}{0.90 * 40 * 5o}$ = 5.96cm^2

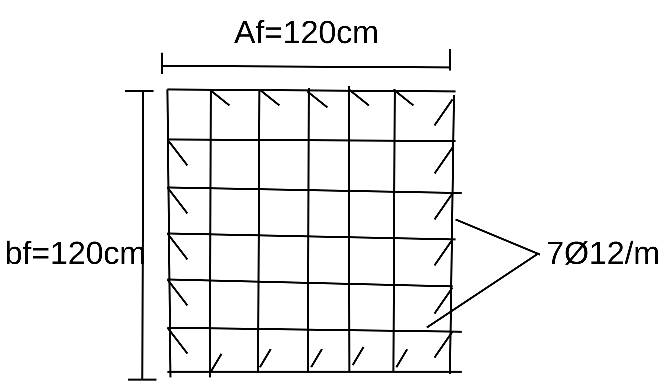
As = 5.96cm^2

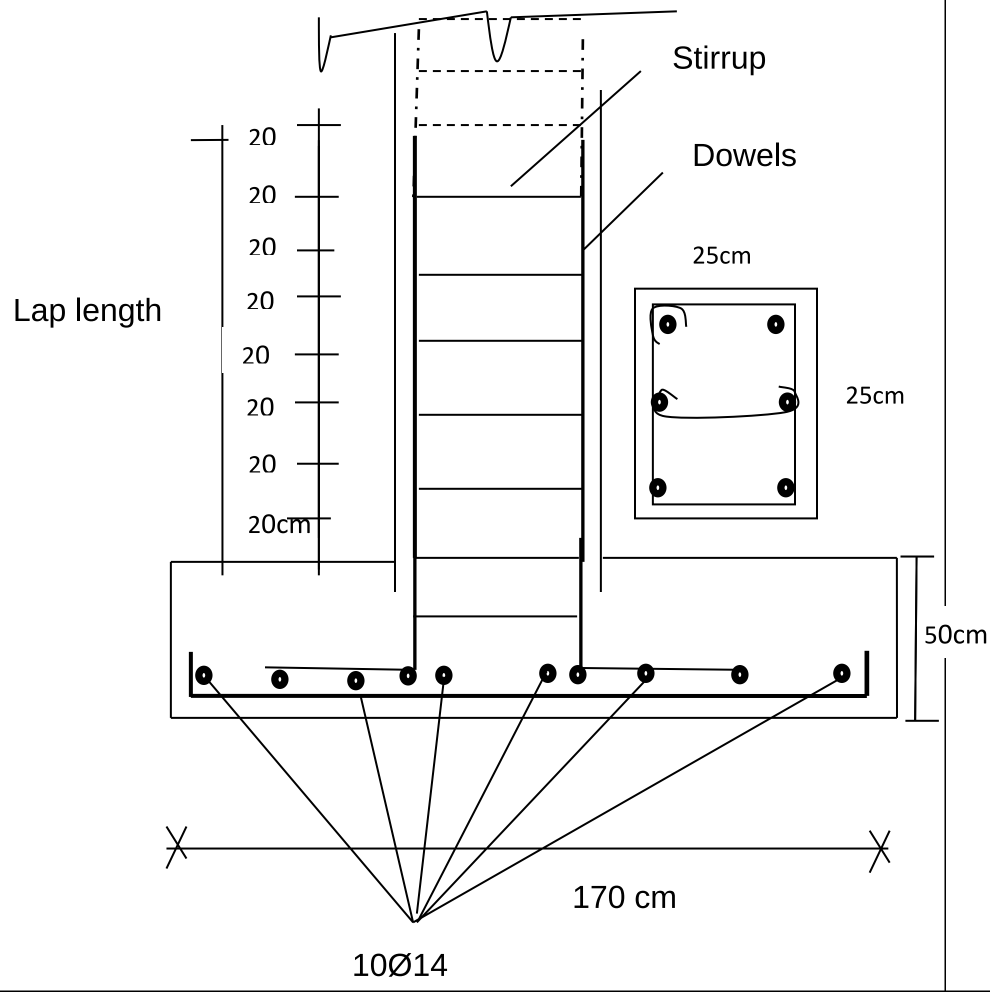
Taken : 10 Ø 14/m provide Ø 14 @ 20cm

- **<u>Cross section of the most loaded footing</u>**



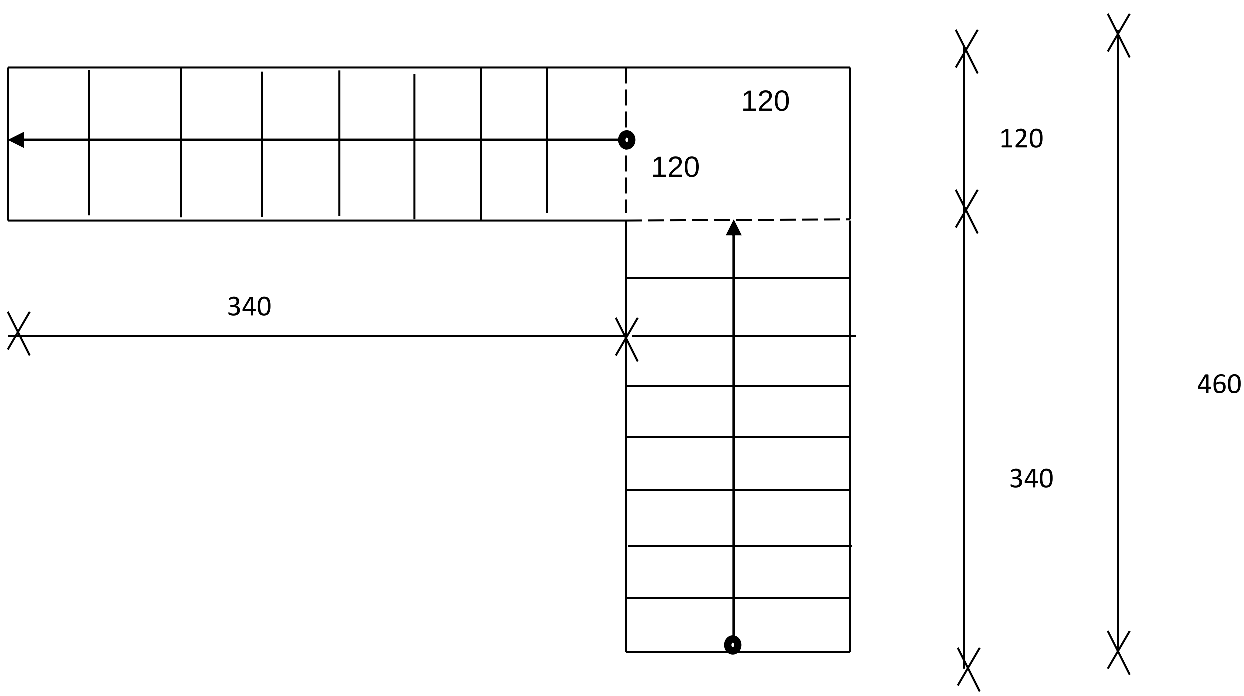
- **Cross section of the medium loaded footing**



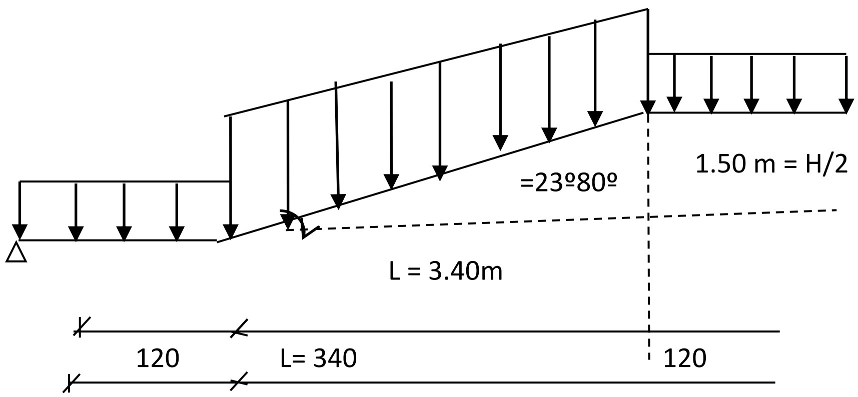
	4.4.7. Steel Reinforcement Arrangement 	

Ref	Calculation	Output
BS8110	**4.5.DESIGN OF STAIR CASE** ***Durability and fire resistance*** Nominal cover for very moderate condition of exposure = 25mm Nominal cover for 1.5 hours fire resistance =20mm Since 25>20,provide nominal cover =25mm Therefore durability and fire resistance are satisfactory ***Preliminary sizing of staircase members*** Height from ground floor slab to first floor slab=3000mm Height from ground floor landing=3000/2=1500mm	Provide nominal Cover=25mm R=175mm G=300mm

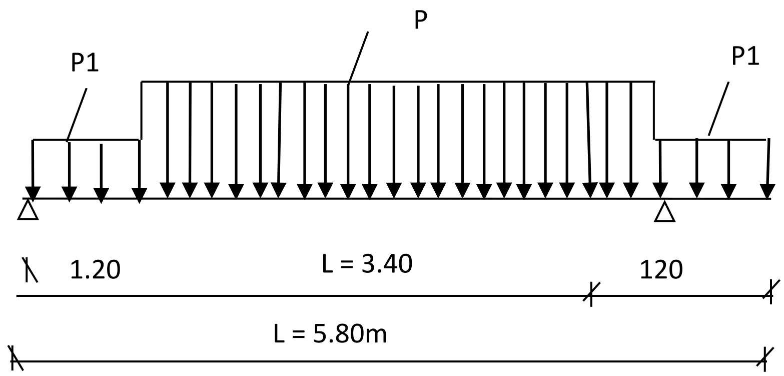
a) Plan view



Vertical cross section



c) Horizontal equivalent slab



4.5.1 Calculation of load P

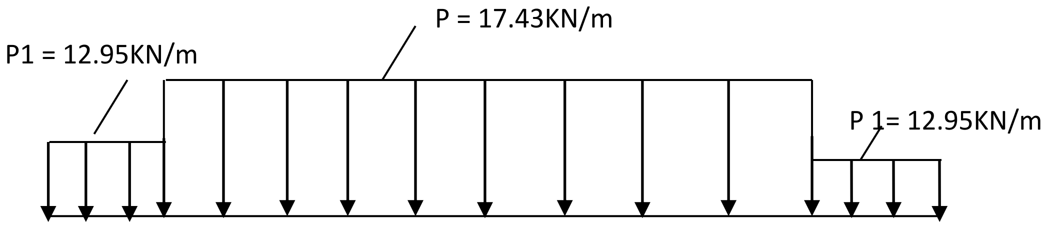
- $Tg\propto = \frac{H/2}{L} = \frac{150}{340} = 0.44117647 \quad \propto = 23^{o}\ 80$
- Thickness of horizontal equivalent slab

$$h = \frac{d\ell}{\cos\propto} + \frac{2H1}{3} = \frac{18}{0.914959667} + \frac{2 * 17.5}{3} = 31.34\ cm = 0.3134m$$

Self weight = 1.40 * 0.3134 x 1 * 24 = 10.53 KN/m

- Finishes = 1.40 * 1.50 = 2.10 KN /m
- Live load = 1.60 * 3KN/m^2*1m = 4.80KN/m

❖ Calculation of load P = 10.53 + 2.10 + 4.80 = 17.43 KN/m



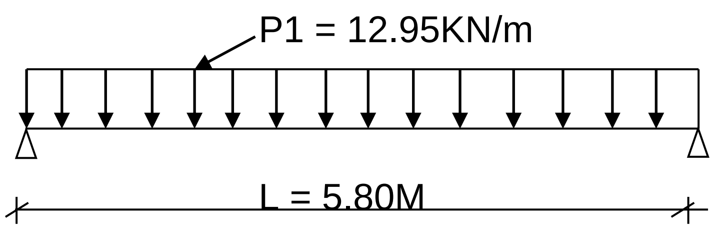
4.5.2. Calculation of load P1

5. Self weight = 1.40 * 0.18 * 1 * 1 * 24 = 6.05 KN/m
6. Finishes = 1.40 * 1.50 = 2.1 KN/m
7. Live load = 1.60 * 3.00 = 4.80KN/m

   Total load P1 = 6.05KN/m + 2.10KN/m + 4.80KN/m = 12.95KN/m

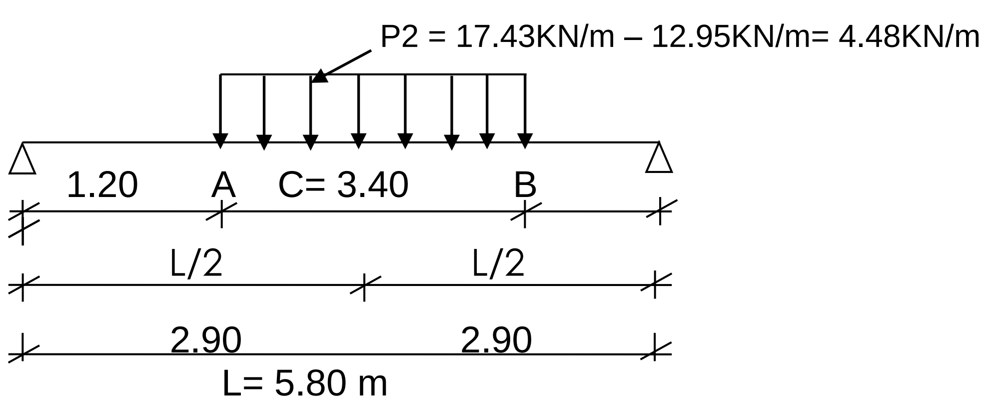
4.5.3 Calculation of Maximum Bending moment for beam P1 as simply supported

We have to consider two cases



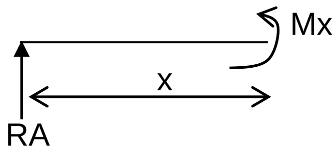
$$\text{Mmax 1} = \frac{P1L^2}{8} = \frac{12.95 * 5.80 * 5.80}{8} = 54.45\text{KN.m}$$

4.5.4. Calculation of Maximum Bending moment for beam without considering landing



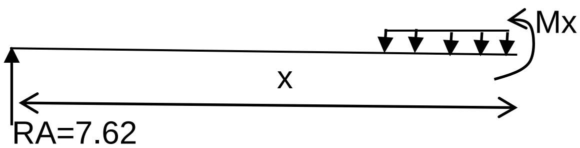
Calculation of Mmax²

Support reaction RA=RB=$\frac{P2*C}{2}=\frac{4.48*3.40}{2}=7.62KN$



- Mx-7.62x=0

  Mx=7.62x in range (0≤x≤1.20)



- Mx-RA*x+P2$\frac{(x-1.20)^2}{2}$
- Mx=RA*x-P2$\frac{(x-1.20)^2}{2}$ in the range (0≤x≤4.60)
- Mx=7.62-4.48 $\frac{(x-1.20)^2}{2}$

we know that $\frac{dMx}{dx}$ = Tx (shear force)

- Tx=$\left[7.62x-4.48\frac{(x-1.20)^2}{2}\right]'$ =7.62-4.48(x-1.20)
- Tx=7.62-4.48x+5.38=0
- Tx=13.00-4.48x=0
- x=13
- -4.48x=-13.00
- x=$\frac{-13.00}{-4.48}=2.90m$

b) Mmax2=RA*x-P2$\frac{(x-1.20)^2}{2}=7.62*2.90-4.48*\frac{(2.90-1.20)^2}{2}$

Mmax2=22.10-4.48*1.45

Mmax2=22.10-6.50=15.60KN.m

4.5.5. Calaculation of total Mmax

Mmax=Mmax1 + Mmax2

Mmax=54.45KN/m+15.60KN.m

Total Mmax=70.05KN.m

4.5.6. calculation of steel reinforcement in the stairs

Ho = dl -2.5cm = 31.34 – 2.5cm = 28.84cm

$$\propto m = \frac{\text{Total Mmax}}{Rb * b * h^2o} = \frac{70.05 \text{ x}100}{1.40 * 100 * 28.84 * 28 * 84} = 0.060$$

From the table of coefficients related to the design of members subjected to bending moment

$\propto m = 0.060 \Rightarrow n = 0.970$

<u>Main steel reinforcement</u>

$$As_M = \frac{\text{Total Mmax}}{n * h_o * Rs} = \frac{70.05 * 100}{0.970 * 28.84 * 40} = 6.26cm^2/m$$

Taken 5Ø 14/m = 7.70cm^2/m

Provide 1 Φ14@20cm as main steel reinforcement

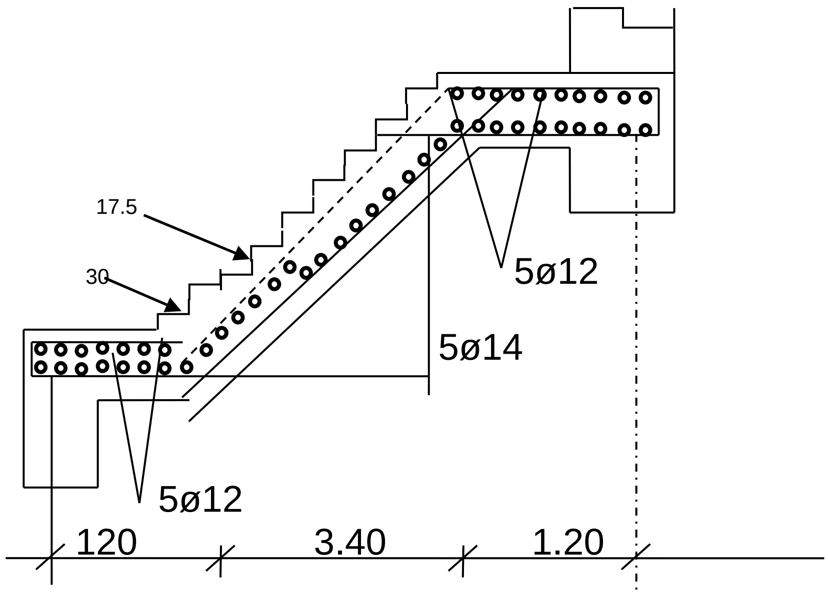
Distribution steel reinforcement

$As_D = As_M * ⅕ = 7.70cm^2 ÷ ⅕ = 1.54cm^2$

Taken 5Φ12 = 5.65cm^2

Provide 1Φ12@20cm as Distribution steel reinforcement

4.5.6. Steel reinforcement arrangement in the stairs



Design by : **Eng. Augustin NDIMUTO**

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