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INTRODUCTION:

Plain Concrete is made of by mixing cement, fine aggregate, coarse aggregate, water and frequency admixture. When reinforcing steel is placed in the forms & wet concrete mix is placed at around it, the solidified mass becomes reinforced reinforce concrete. The strength of concrete depends on many factors, notably the proportion of the ingredients & the conditions of temperature and moisture under which it is placed and cured. Contained in subsequent sections are brief discussions of the materials in and the properties of plain concrete. The treatment is intended to be only introductory; an interested reader should consult standard references devoted entirely to the subject of plain concrete.
Reinforced concrete is a logical union of two materials; plain concrete which possesses high compressive strength but little tensile strength, and steel bars embedded in the concrete which can provide the needed strength in tension.
For most effective reinforcing action, it is essential that steel and concrete deform together, i.e. Us we are see that there is a sufficiently strong bond between the two materials to ensure that no relative movement of the steel bars and the surrounding concrete occur.
This bond is providing by the relatively large chemical adhesion which develops at the steel. Concrete interface, by the natural roughness of the mill scale of boot-rolled reinforcing bars, and by the closely spaced rib shaped in order to provide a high degree of inter locking of the two materials.
The project is "Structural Design for Secondary School" with eighteen classrooms in two stories.
The building is consists of four departments which are:

1. Management Department
2. Laboratory Department
3. Classroom Department
4. Healthy Department

Construct rally, the building is divided in to four joint sections, each story it builds with concrete blocks and reinforced concrete.
"Bearing wall" is the case so strip or wall footing is required for this, but there is some column in the structure which has to column footing.
There is located place in the master plan for the activity hall in future.

NOTATION:

As: Area of tension reinforcement, (in^2)

As′: Area of compression reinforcement, (in^2)

b: Width of compression face of member, (in^2)

d: Distance from extreme compression fiber to centric of tension reinforcement, (in^2)

E c: Modulus of elasticity of concrete, (Ksi)

E s: Modulus of elasticity of steel, (Ksi)

F c′: Specified compressive strength of concrete, (psi)

F y: Specified strength of non– pre stress reinforcement, (psi)

L: Clear span for positive moment or shear and the average adjacent clear spans for negative moment.

N: Modulus ratio = Es/Es.

W: Design load per unit length of bean or per unit area of slab.

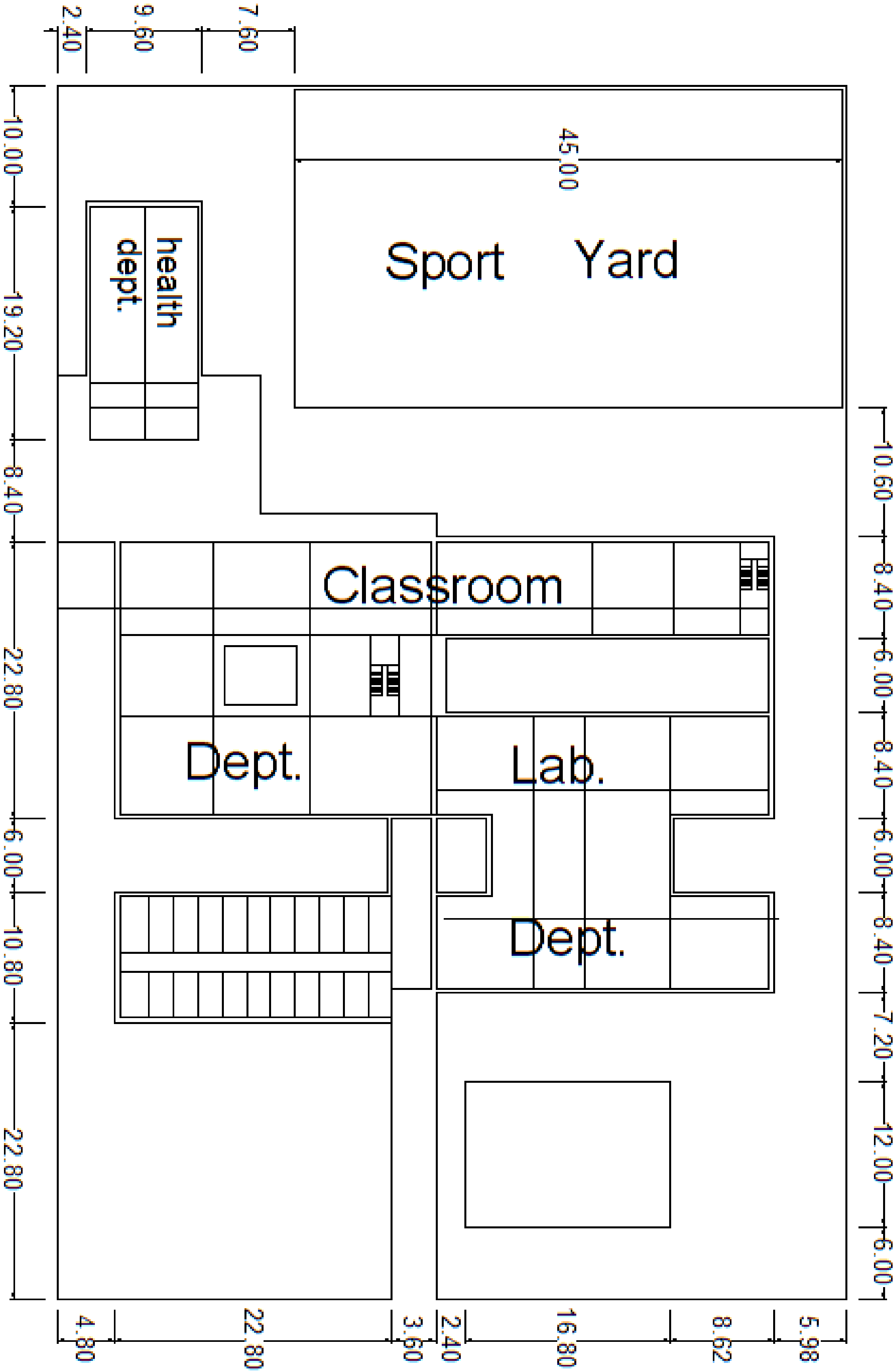
V c: Nominal permissible shear stress carried by concrete.

β_1: A factor defined in section 10.2.T ACI-Code.

ρ: As/b*d: ratio of non- pre stressed tension reinforcement to the effective area Of the section

ρ′: As′/b*d.

ρ b: Reinforcement ratio producing balanced Condition.


Sport Yard
45.00
health dept.
Classroom
Dept.
Lab.
Dept.
2.40
9.60
7.60
10.00
19.20
8.40
22.80
6.00
10.80
22.80
10.60
8.40
6.00
8.40
6.00
8.40
7.20
12.00
6.00
4.80
22.80
3.60
2.40
16.80
8.62
5.98

Chapter One

Management Department

F c′ = 3000 psi , F y= 6000 psi

For finding the thickness of the slab we chose the larger span of the building. The larger span is in (laboratory department) which is (27` * 19.7`).

Clear span = 26.3` * 19`

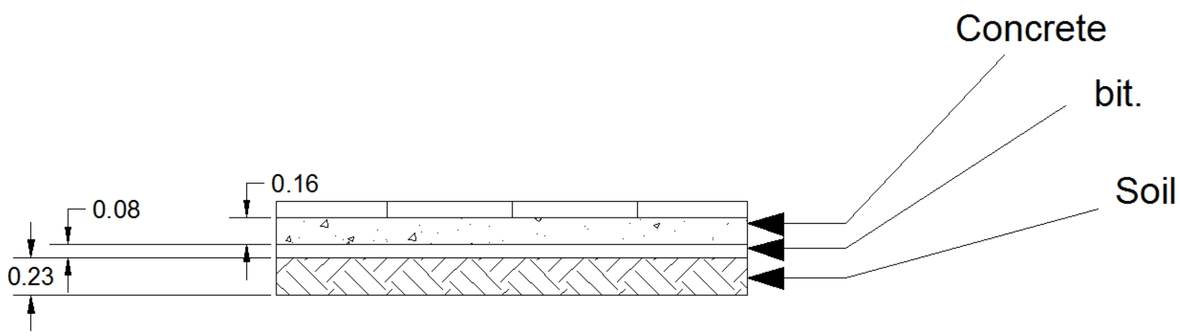
h minimum = $Perimeter/\ 180 = \frac{1}{180} * [2(26.3 + 19) * 12] = 6''$

- Use thickness of slab = 6″

Management Department

L.L = 40psf

D.L for 1ft2 = (1*1*$\frac{6}{12}$)*150 = 75psf



γ Soil =100 lb/ft^3

γ Bit = 84 lb/ft^3

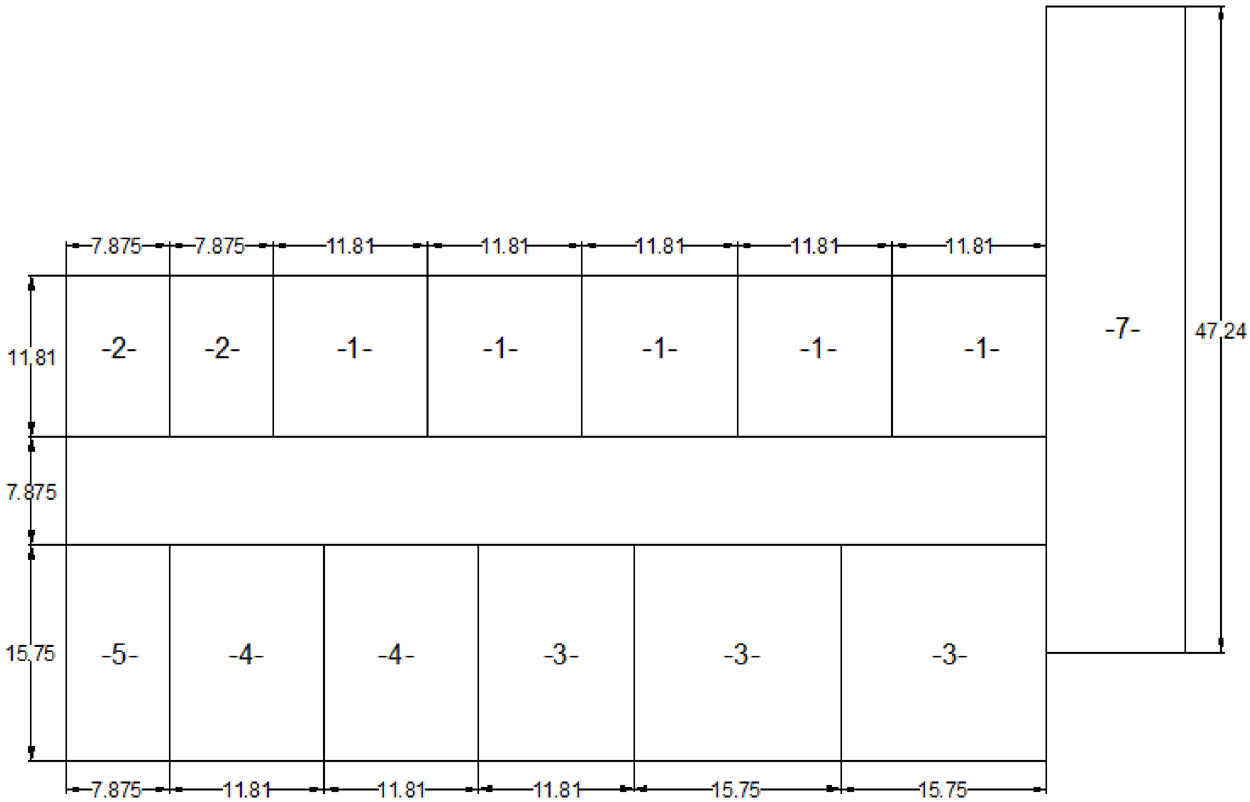
D.L = (0.16*150) + (0.08*84) + (0.23*100) ≈ 55 psf.

- Total D.L =1.4 * 130 = 182 psf.

Factored D.L = 1.4 * 130 = 182 psf.
Factored L.L = 1.7 * 40 = 68 psf.

Wu = Factored D.L + Factored L.L

= 182 + 68 = 250psf.



$m = \frac{la}{lb}$

Ma = (Ca)*(w)*$(la)^2$

Mb = (C b)*(w)*$(lb)^2$

-ve disco. moment = $\frac{+ve}{3}$

-Ca & -Cb ⟶ table 8.3

+Ca* D.L. & + Cb*D.L. ⟶ table 8.4

+ Ca*L.L. & + Cb*L.L. ⟶ table 8.5

+ μ = (+ μ D.L) + (+ μ L.L)

Slabs	Clear Span	m	direction	Case	-ve c	+ve c d.l.	+ve c l.l.	-ve M con. k.ft	-ve M disco. k.ft	+ve M k.ft
S 1 (l_a, l_b)	11.10	1.000	a	8	0.035	0.02	0.028	1.016	0.23	0.68
	11.10		b		0.061	0.023	0.03	1.88	0.26	0.77
S 2 (l_a, l_b)	11.1	0.750	a	4	0.076	0.043	0.052	2.34	0.56	1.4
	15.1		b		0.024	0.013	0.016	1.37	0.26	0.8
S 3 (l_a, l_b)	15.1	1.000	a	8	0.033	0.02	0.028	1.88	0.47	1.26
	15.1		b		0.061	0.023	0.03	3.48		1.4
S 4 (l_a, l_b)	11.1	0.75	a	9	0.078	0.031	0.046	2.4	0.16	1.08
	15.1		b		0.014	0.01	0.013	0.80		0.50
S 5 (l_a, l_b)	7.21	0.6	a	4	0.094	0.059	0.077	1.2	0.28	0.83
	15.1		b		0.006	0.004	0.005	0.24	0.08	0.24
S 6 (l_a, l_b) One-way slab	7.21				+ve M = $w L^2 / 24$			1.3		0.65
	74.1				-ve M = $w L^2 / 12$					
S 7 (l_a, l_b) One-way slab	11.1							2.90		1.45
	46.56									

It is clear that the maximum calculated moment from the table is 3.48 k. ft in slab (S_3). So we design the slabs for that maximum moment to finding "As, Area of steel". If it adequate for that maximum moment then it's O.k. and safe for the other slabs.

For $S_{3:}$

-ve Ma = 1.88 k. ft = 22560 lb. in
-ve Mb = 3.48 k. ft = 41760 lb. in
+ve Ma = 1.26 k. ft = 15120 lb. in
+ve Mb = 1.4 k. ft =16800 lb. in

-ve discontinue moment= 0.47 k. ft =5678lb.in

Calculating As:
Generally:

Mu =φ*As*fy*(d- $\frac{a}{2}$) R.1 (3.40 a) P.83

a=As*$\frac{fy}{0.85}$ *fc′*b R.1 (3.39) P.83

a = ($\frac{60*As}{0.85*3*12}$) = 75 As ……..……………… (1)

Mu = 0.90*As(60000)*(5- $\frac{1.96*As}{2}$)

Solve & get :

$As^2 - (5.1*As) + \frac{\mu u}{52920} = 0$

a= 1 ; b= -5.1 ; c= $\frac{\mu u}{52920}$

$$As = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

Maximum moment is -ve μb = 41760lb.in

As^2 - (5.1*As) + $\frac{41760}{52920}$ = 0

As^2 – (5.1*As) + 0.79 = 0

$$As = \frac{5.1 \pm \sqrt{(5.1)^2 - 4 * 1 * 0.79}}{2 * 1}$$

As = 0.16 in^2/ft

ρ Minimum = 0.0018 ACI-Code 7.12.2.1-b

As minimum = ρ Minimum*b*d

= 0.0018*12*5

= 0.11 in^2/ft

0.16 > 0.11

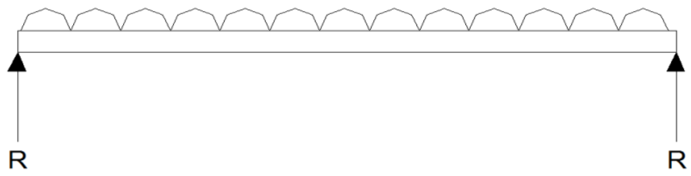
- O.k. Use 0.16 in^2/ft

- Use #3 at 8″ center to center

As provide = 0.17 > 0.16 ❖ O.k.

Use the same (As) for the other direction.

Check for Shear:



V = R = $\frac{w}{2}$*wt*l

W a = 0.33

R1 table 8.6

W b = 0.67

- V maximum = $\frac{0.67}{2}$*250*11.1 = 930 lb

$V_c = V_{all} * b * d$ ACI-Code 11.3.1.1

$V_{c\,all} = 2*\varphi*\sqrt{fc'}$

$= 2*0.85*\sqrt{3000}*12*5$

$= 5586$ lb

$5586 > 930$

- O.k. Safe

No Shear design.

Check for development length:

From table A-10 [design of concrete structure]:

By Arthur H. Nilsson & George Winter:

$L\,d_{min}$ for basic bar = 12″

$L\,d_{min}$ for top bar = 13″

Check these lengths with formulas:

$L\,d_b = 0.04 * Ab * \frac{fy}{\sqrt{fc'}}$

Or:

$L\,d_b = 0.004 * db * fy$

Or:

Ld = 12″

- $L\,d_{min} = (0.04 * 0.11 * 60000) / \sqrt{3000}$

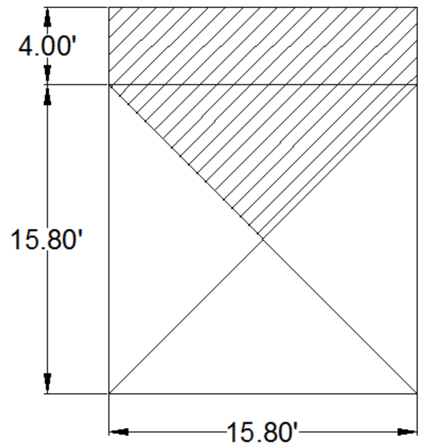
$L\,d_{min} = 0.004 * \frac{3}{8} * 60000 = 9''$

- Use ld for bottom bar = 12″

And

Use ld for Top bar = 13″

Footing



Tributary area = $\frac{15.8*15.8}{4}$ = 126 ft^2

Un factored loads:

Slab = 126 * 170 = 21420 lb

Load of wall/1′ = $\frac{8}{12}$ * 10 *1*150 = 1000lb

$(\frac{10}{12} + \frac{16}{12} + \frac{8}{12}) = \frac{34}{12}$ ft

$\frac{34}{12}$ *1*1*150 = 425 lb/ft length

Total un factored load = 2781 lb/ft length

Factored loads:

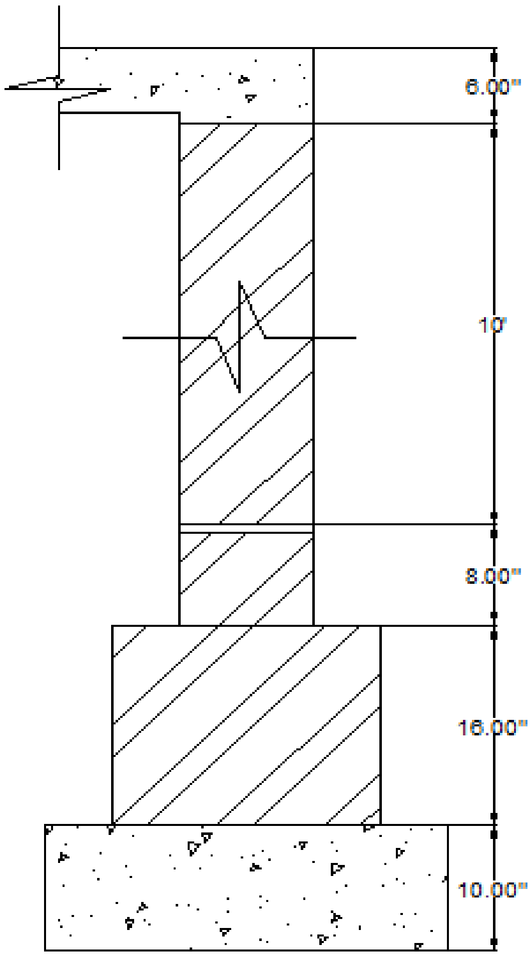
126*250 = 31500lb

Load/1′ length = 1994lb

Load of wall/1′ length = 1995lb

Total factored loads = 3989 lb per 1′ length

q a = 8ksf



q un = q a - γ * d

= 3000 – 2 * 120 = 2760psf

A= B*1 = $\frac{pun}{qun}$ = $\frac{2781}{2760}$ = 1.007ft^2

Use pedestal on footing which thickness of 2 blocks equal to 16″

So its favorable those make the width of the footing 2ft for best workability.

The common depth for wall footing is shallow foundation

If $B \geq D$

Let B = D = 2ft

$q_{\text{Unfactored}} = \frac{pun}{\text{qun}} = \frac{pfac}{\text{qfac}} = \frac{2781}{3000} = \frac{3938}{\text{qfact}}$

- q fact = 4303psf

Finding thickness of footing considering shear forces:

Applied shear = (4-d)*4303*144

Resistance shear = $2*\varphi*\sqrt{fc'}*b*d$

$=2*0.85*\sqrt{3000}*12*d$

Applied shear = Resistance shear

$(4-d)*4303*144 = 2*0.85*\sqrt{3000}*12*d$

4-d = 0.0018*d

$d = 3.99in \approx 4in < 6in$ minimum depth

- adopt d = 6in

Find As considering moment in critical section

$\mu = q\ fact * L^2/2$

$= (4303/144) * (8^2/2)$

=956lb.in

$(\mu_u)/(\varphi*b*d^2) = (956)/(0.9*12*6^2) = 2.49psi \approx 2.5psi$

2.5 psi << 200psi (minimum reinforcement required)

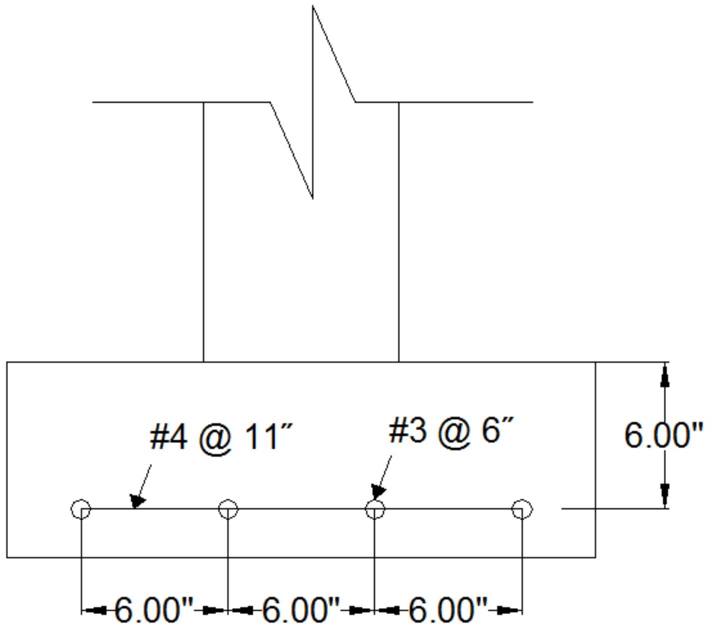
For 200psi → $\rho = 0.0033$

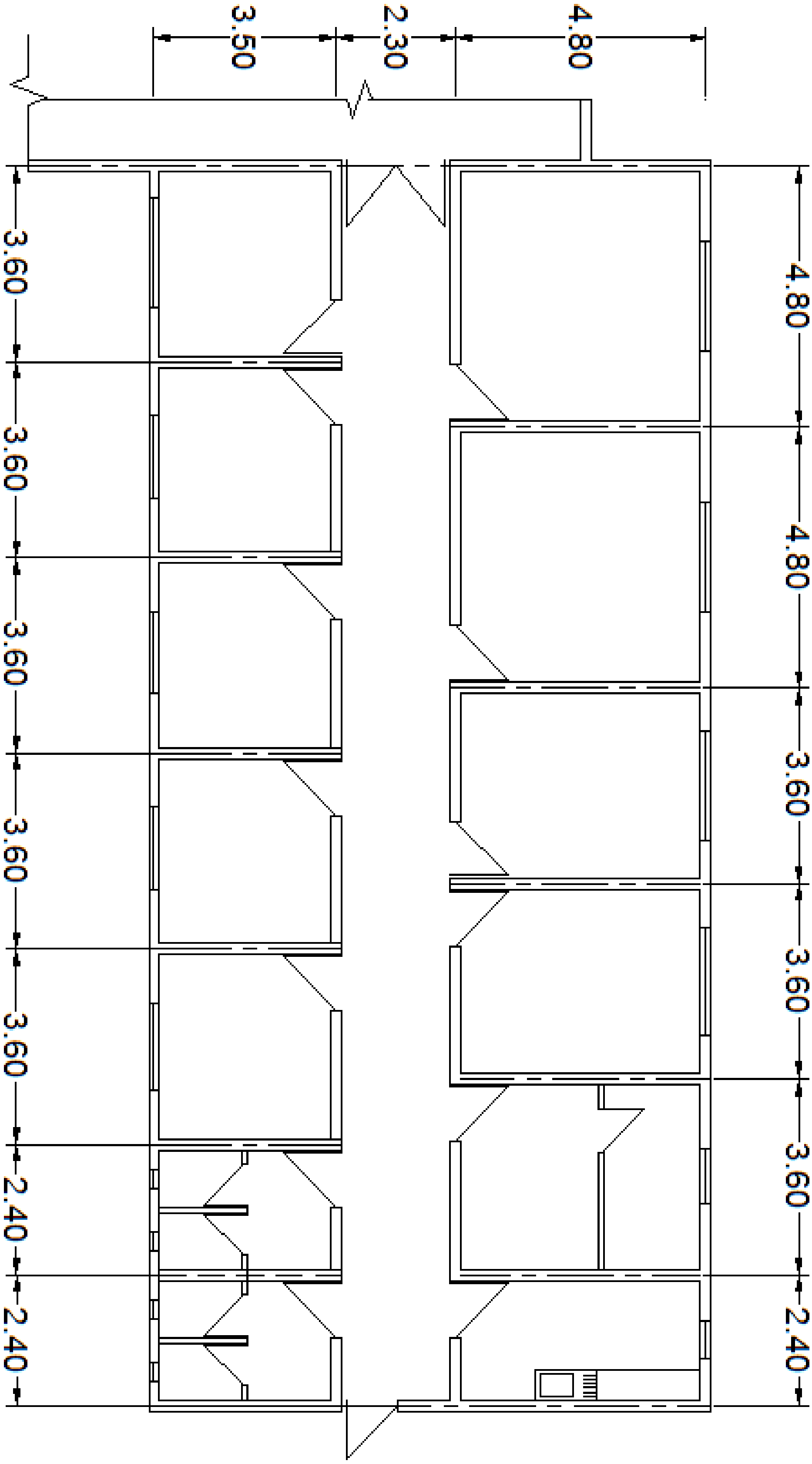
As = 0.24 in^2 per 1ft length

Use #4 @ 11″ c/c

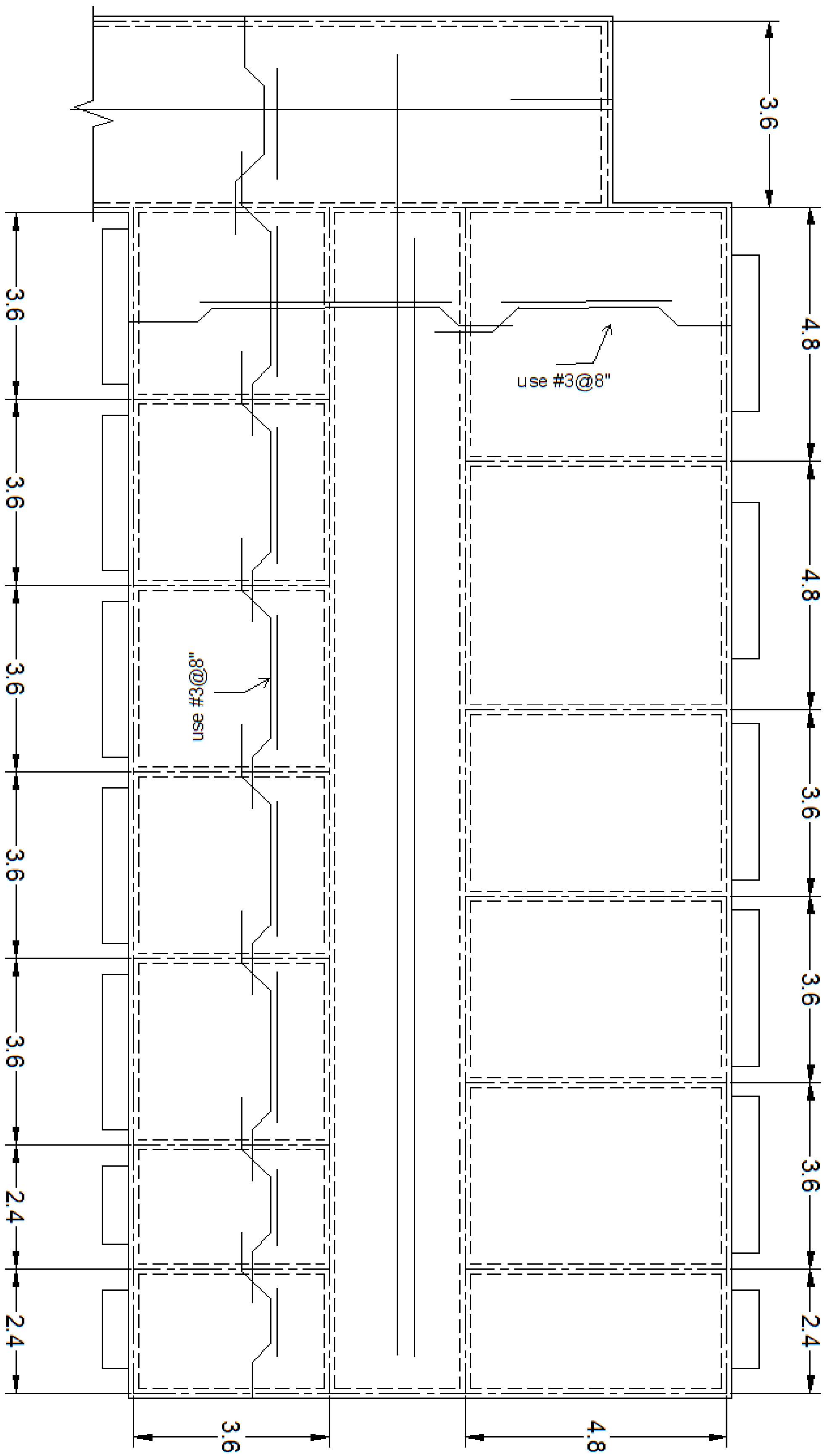
And

Use #3 @ 6″ c/c for longitude direction.

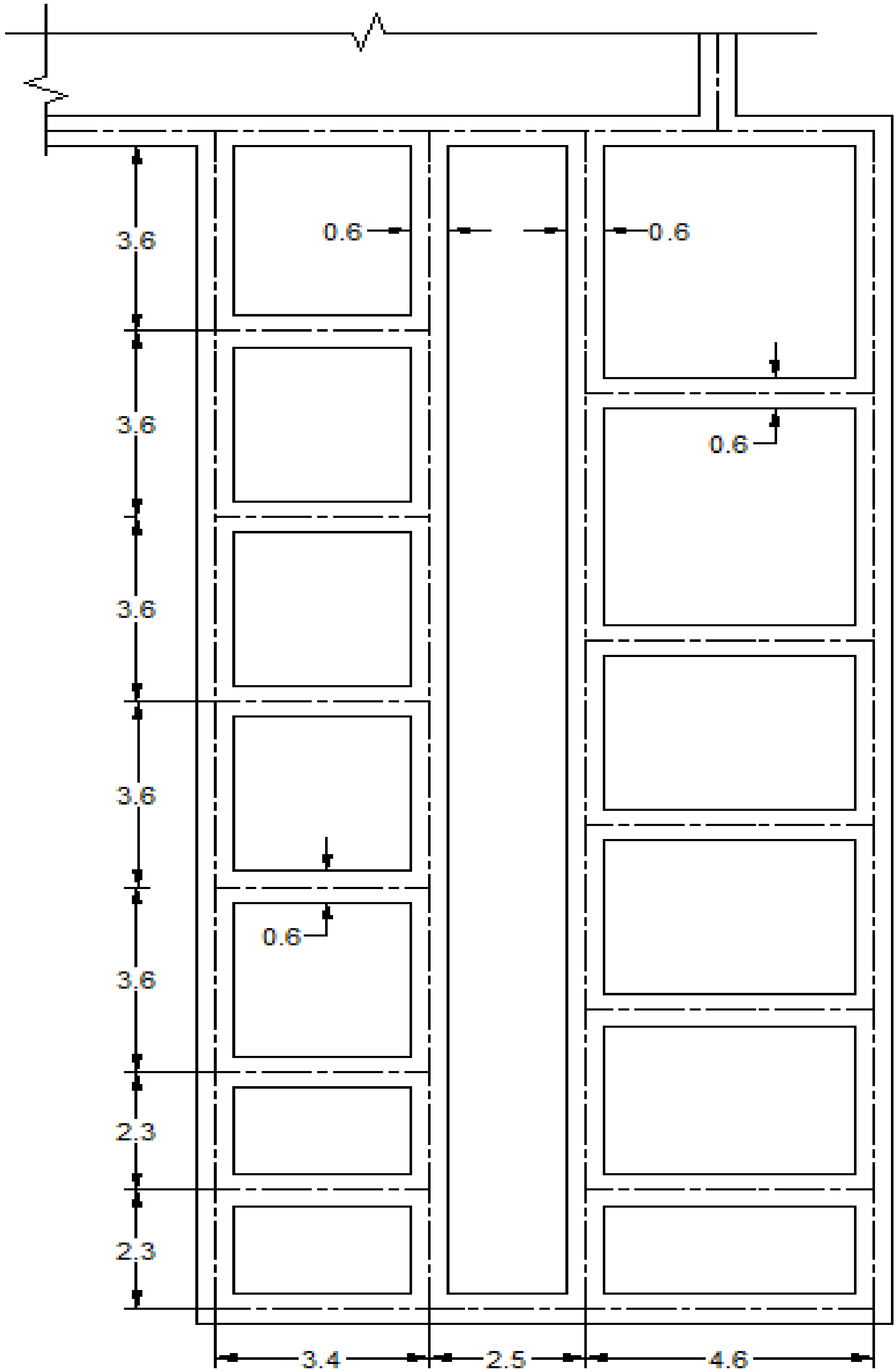




MANAGEMENT DEPT.



REINFT. PLAN-MANAGEMENT DEPT.

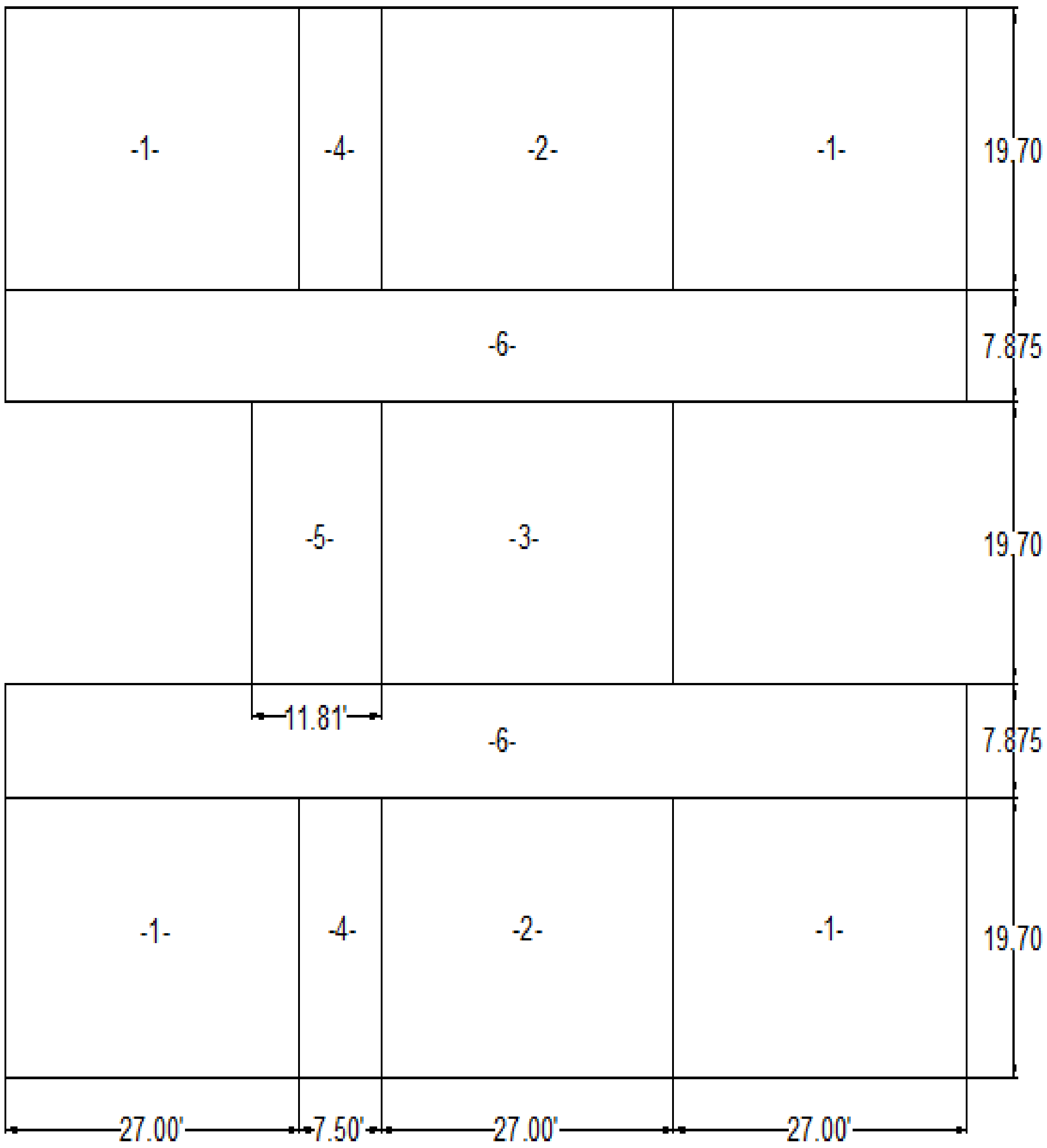


FOOTING PLAN-MANAGEMENT DEPT.

Chapter Two

Laboratory Department

LABORATORY DEPT.



Slabs	Clear Span	m	direction	Case	-ve c	+ve c d.l.	+ve c l.l.	-ve M con.	-ve M disco.	+ve M
S 1 l_a	19	0.725	a	4	0.078	0.045	0.054	7.04	1.42	4.28
l_b	26.30		b		0.022	0.012	0.015	3.8	0.74	2.21
S 2 l_a	19	0.725	a	8	0.0645	0.038	0.0515	5.62	1.25	3.76
l_b	26.5		b		0.0325	0.012	0.016	5.62		2.21
S 3 l_a	19	0.725	a	9	0.032	0.048	7.17	1.38	1.38	3.28
l_b	26.3		b		0.0125	0.0065	0.012	2.11		1.38
S 4 l_a	6.53				+ve $M = w L^2 / 24$			1.17		0.58
l_b One-way slab	19				-ve $M = w L^2 / 12$					
S 5 l_a	11.1	0.6	a	8	0.08	0.048	0.07	2.6	0.53	1.6
l_b	19		b		0.02	0.01	0.01	1.60		0.70
S 6 l_a	7.21				+ve $M = w L^2 / 24$			1.30		0.65
l_b One-way slab	57.83				-ve $M = w L^2 / 12$					

As before we choose the Maximum moment among the slab's moments. And it's in slab-3- S_3= 7.17k.ft

-ve Ma = 7.17k.ft = 84074lb.in

-ve Mb = 2.16k.ft = 25920lb.in

+ve Ma = 3.26k.ft = 39360lb.in

+ve Mb = 1.38k.ft = 16560lb.in

-ve moment at discontinuous edge = 1.38k.ft = 16560lb.in

Generally: see previous slabs :

$As^2 - (5.1*As) + \frac{84474}{52920} = 0$

$As^2 - (5.1*As) + 1.596 = 0$

$$As = \frac{+5.1 \pm \sqrt{5.1^2 - 4*1*1.596}}{2*1}$$

As = 0.335 in^2

By using table A-4 winter:

Use # 4bar at 7″ c/c

As = 0.34 > 0.335in^2

- o.k.

Use the same As for other direction.

Check for Shear:

$V = R = \frac{w}{2} * W_t * L$

W a = 0.89

W b = 0.14

- V maximum = $\frac{0.89}{2}$ *250 * 19 = 2113.75 lb

V c = V c all * b * d

V c all = 2*φ*$\sqrt{fc'}$

V c all = 2 * 0.85 * $\sqrt{3000}$

= 93.113 lb/in^2

- V c all = 93.113 * 12 *5 = 5586.77 lb

5586.77 > 2113.75

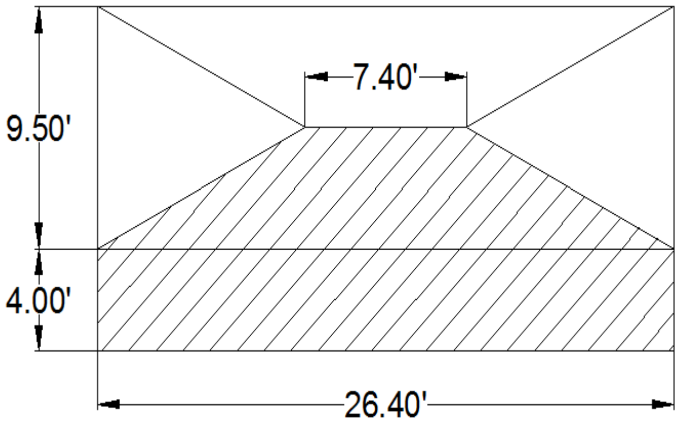
- o.k.

- Shear reinforcement not required.

Footing of laboratory department

Tributary area:

$[\,(\frac{7.4+26.4}{2}) * 9.5] + [\,4 * 26.4\,] = 266.15ft^2$



Un factored load:

Slab:

266.15 * 170 = 4524.55lb

Load/1` length = 1714lb

Load of wall/1` length = 1000lb

Total load of first floor = 2714lb

And weight of ground floor = 2714lb

A weight of sub base = 425lb

❖ total weight = 5853lb/1` length

Factored load:

266.15 * 250 = 66537.5lb

Load/1`length of first floor = 2520lb

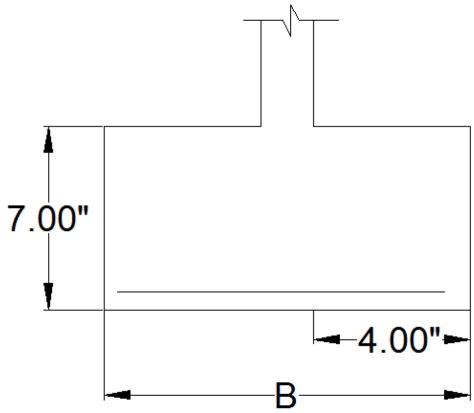
Load/1`length of ground floor = 2520lb

Load/1`length of wall at first floor = 1400lb

Load/1`length of wall at ground floor = 1400lb

Load/1`length of sub base = 595lb

❖ total load = 8435lb



$A = B = \frac{pfac}{qunf} = \frac{5853}{3000} = 1.951$

B = 2ft

D = B = 2ft

$A = \frac{punf}{qunf} = \frac{pfac}{qfact} = \frac{5853}{3000} = \frac{8435}{qfact}$

❖ qfact = 4323psf

Applied shear = Area * qfact

= (4-d) * 4325 *144

Resisting Shear = $2*\varphi*\sqrt{fc'}*b*d$

Applied Shear = Resisting Shear

(4-d) * 4325 * 144 = 1117.35 * d $\rightarrow$ d = 4″ < 6″

Adopt d = 6″

$\mu = qfact * \frac{L2}{2} = \frac{4323}{144} * \frac{(8)2}{2} = 960lb.in$

$\mu u/(\varphi*b*d)$

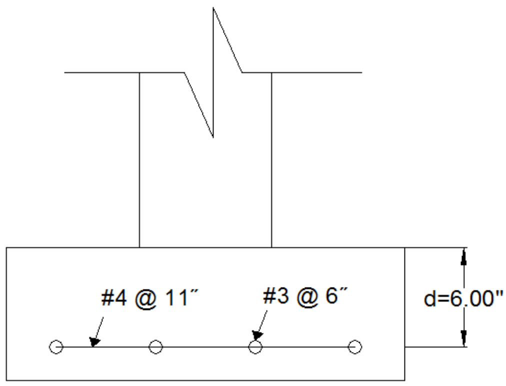
$(960) / (0.9 * 12 * 12 * 6^2) = 2.5$ psi << 200

❖ minimum reinforcement is required

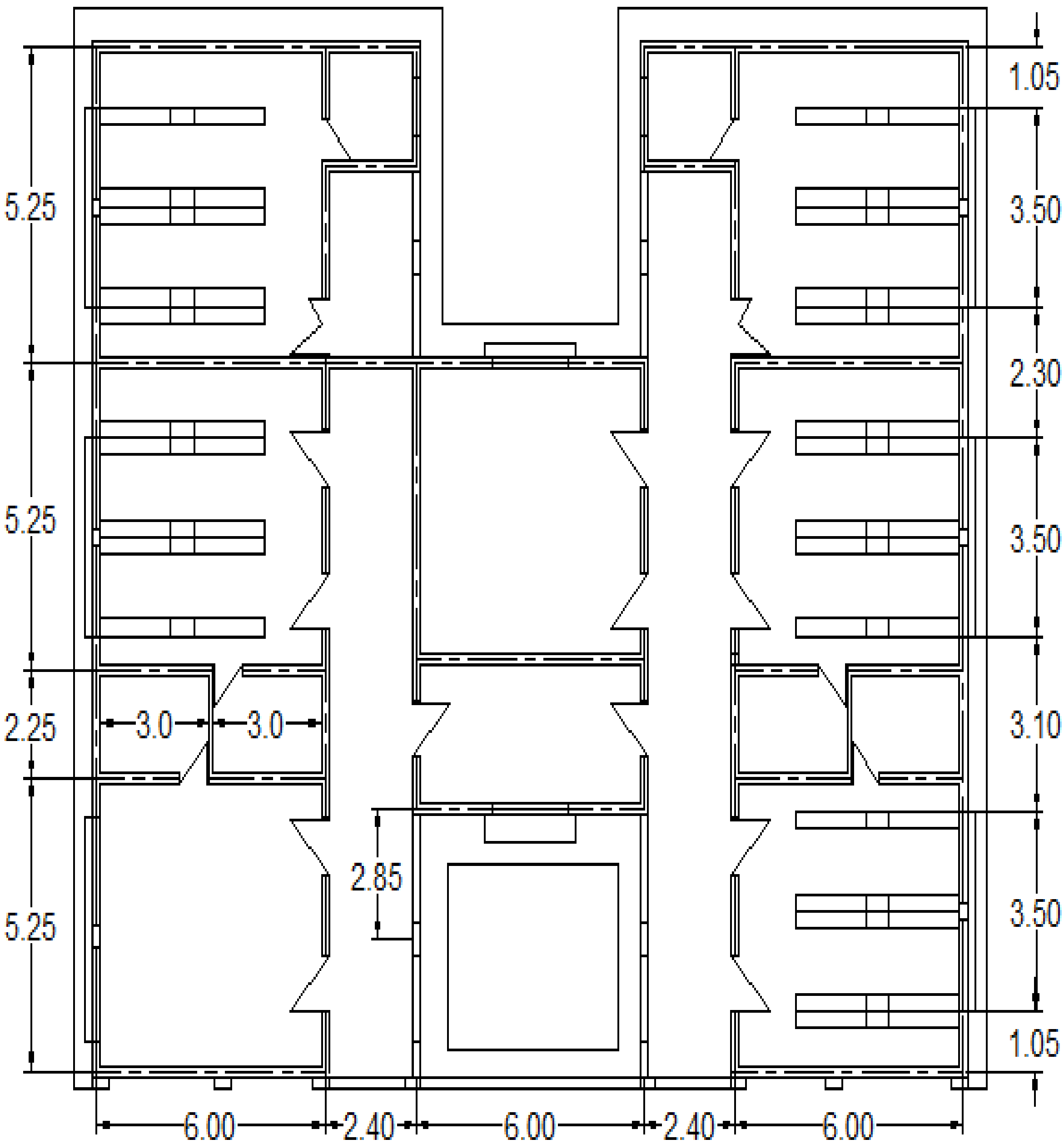
$\rho = 0.0033$ & $As = 0.24in^2/1$` length

Use #4 @ 11″ c/c And

Use #3 @ 6″ c/c for longitude direction



LAB. DEPT.



5.25
5.25
2.25
5.25
3.0
3.0
2.85
1.05
3.50
2.30
3.50
3.10
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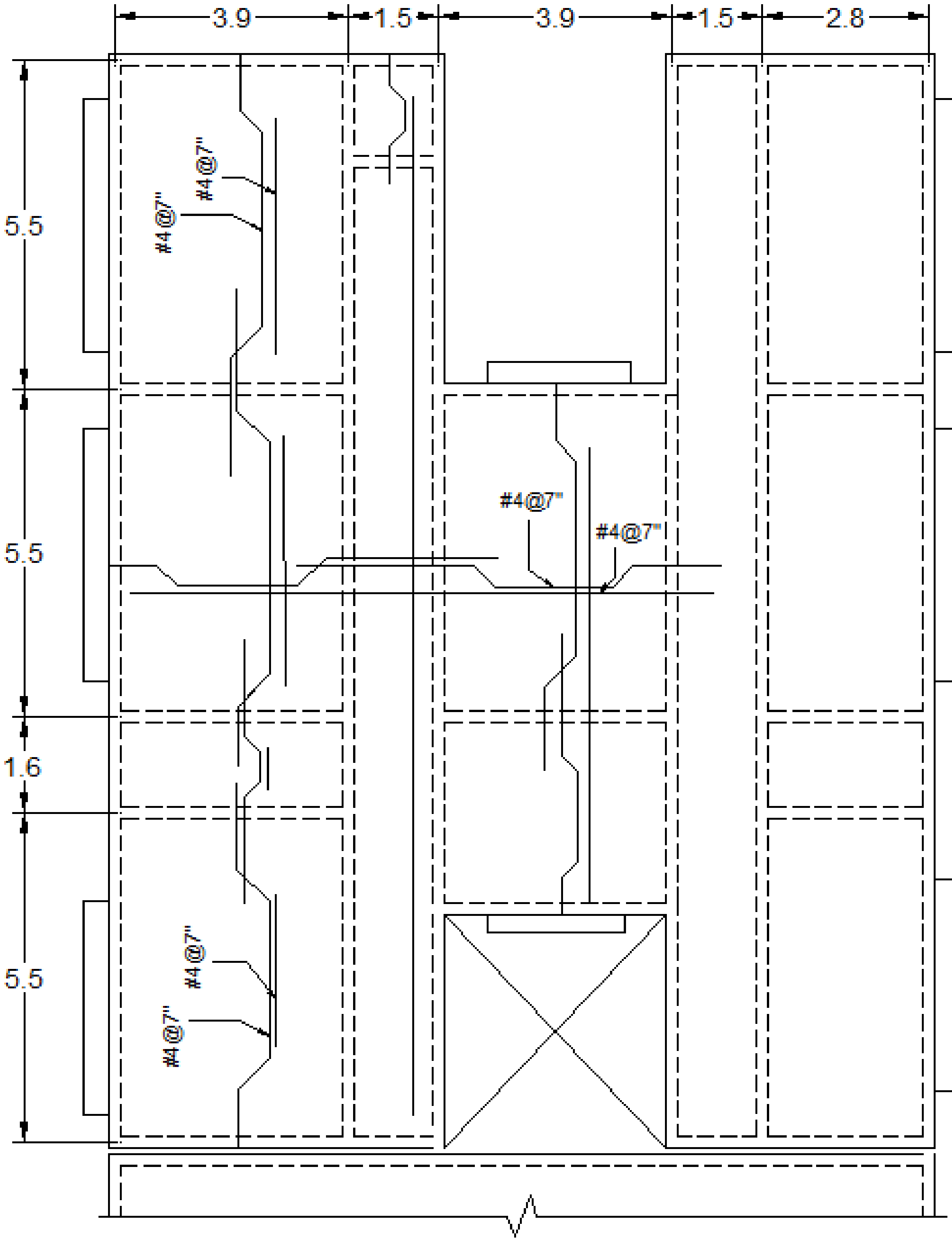
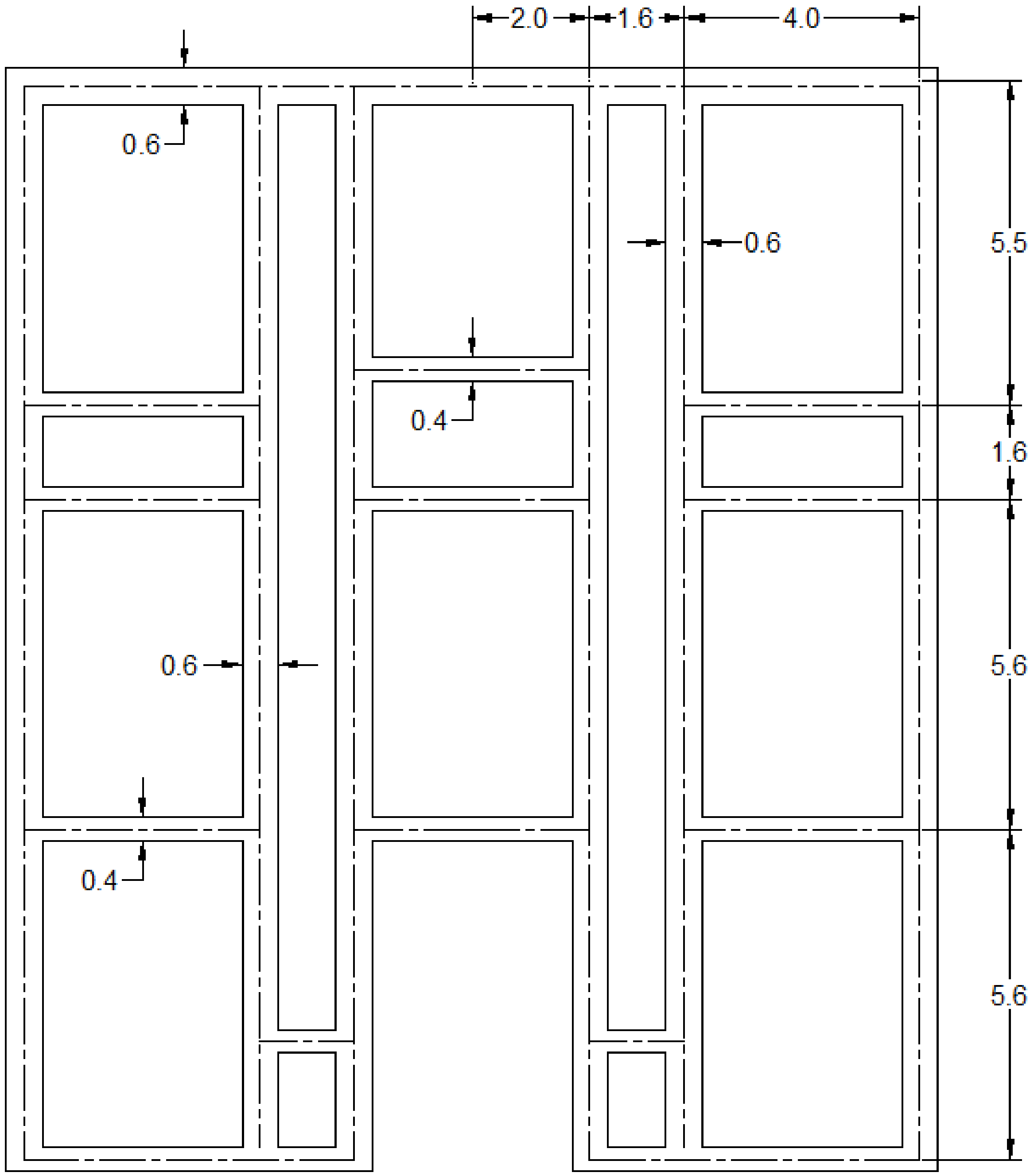
6.00



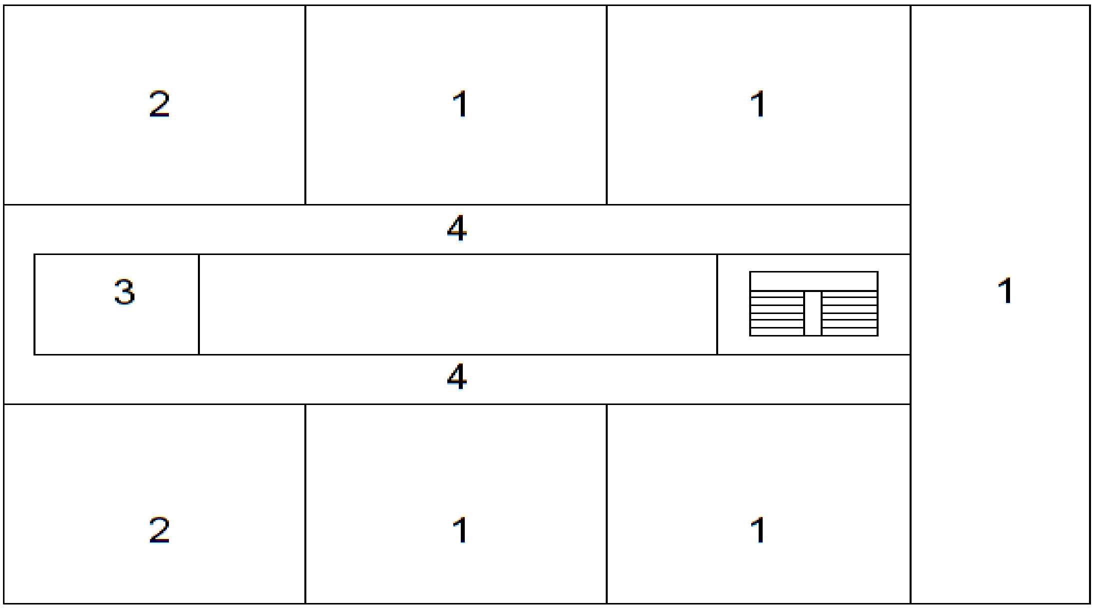
REINFT. PLAN-LAB.DEPT.



FOOTING PLAN-LAB. DEPT.

Chapter Three

Classroom Department



A	B	C	D	E	F	G	H	I	J	K
Slabs	Clear Span	Case	m	direction	-ve c	+ve c d.l.	+ve c l.l.	-ve M con.	-ve M disco.	+ve M
S 1 (a)	19	8	0.85	a	0.052	0.0315	0.042	4.693	1.03	3.1
(b)	22.90			b	0.0435	0.016	0.021	5.7		2.28
S 2 (a)	19	4	0.83	a	0.0685	0.037	0.046	6.2	1.2	3.6
(b)	22.9			b	0.032	0.018	0.022	4.2	0.833	2.5
S 3 (a)	11.1	8	0.60	a	0.08	0.048	0.065	2.5	0.53	1.6
(b)	19			b	0.018	0.007	0.009	1.6		0.7
S 4 One-way slab (a)	7.21				+ve M =	$wL^2/24$		1.1		0.54
(b)	70.1									
S 5 One-way slab (a)	11.1				-ve M =	$wL^2/12$		2.57		1.28
(b)	74.2									

Maximum moment among the table is = 6.2k.ft = 7440lb.in

Generally:

$As^2 – (5.1*As) + \frac{74400}{52920} = 0$

$As^2 - (5.1 * As) + 1.405 = 0$

- $As = \frac{+5.1 \pm \sqrt{(5.1)^2 - (4*1*1.405)}}{(2*1)}$

$As = 0.2922 in^2$

Using table A-4 (winter):

- Use #4 @ 7.5″

And use the same (As) for the other direction.

Check for Shear:

$V \text{ maximum} = R = \frac{w}{2} * w_t * l$

W a = 0.69 & W b = 0.31

- $\mu \text{ maximum} = \frac{0.69}{2} * 250 * 19 = 16391lb$

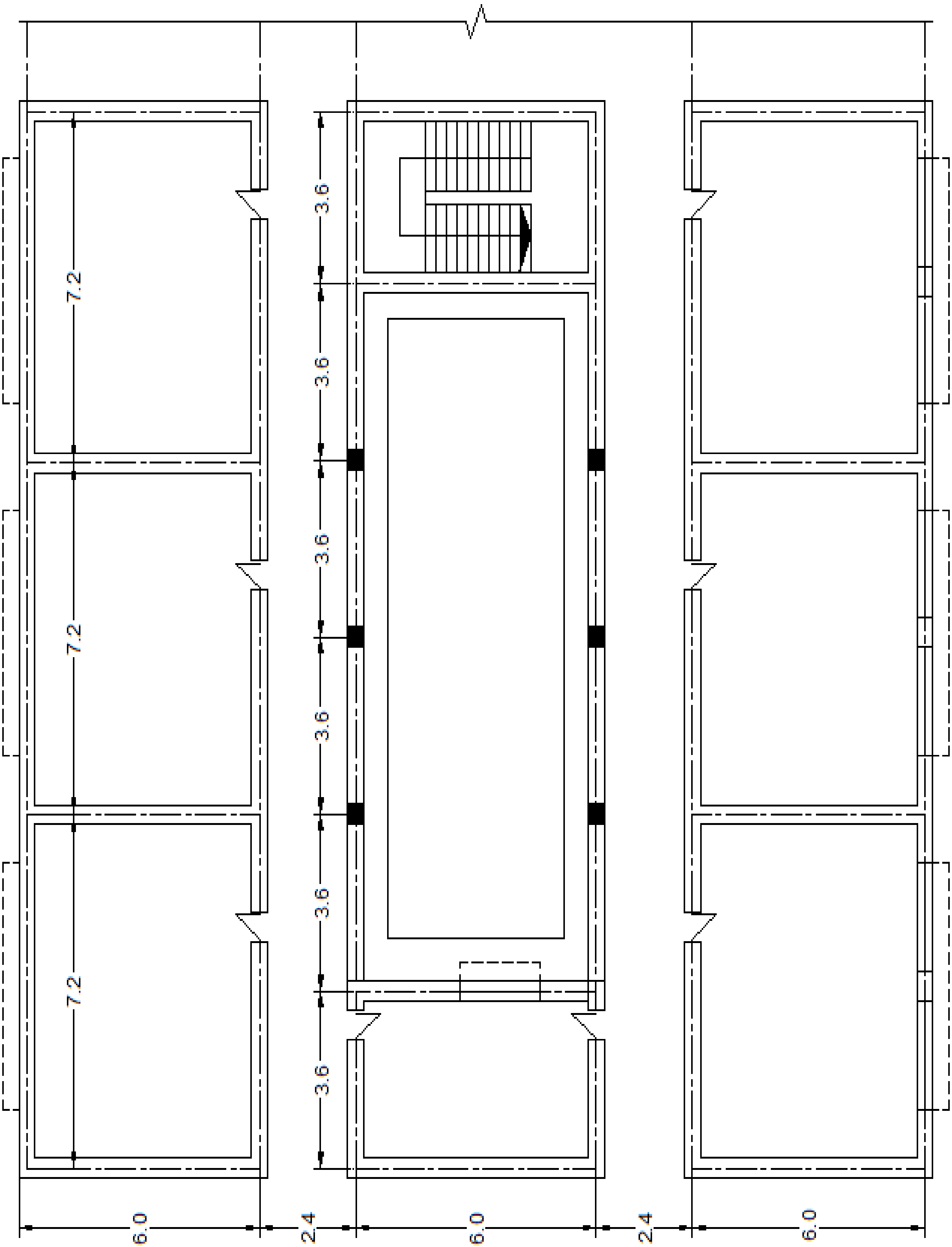
$V c = 2*\varphi*\sqrt{fc'} *b*d$

$= 2 *0.85 * \sqrt{3000} * 12 * 5 = 5586.771lb$

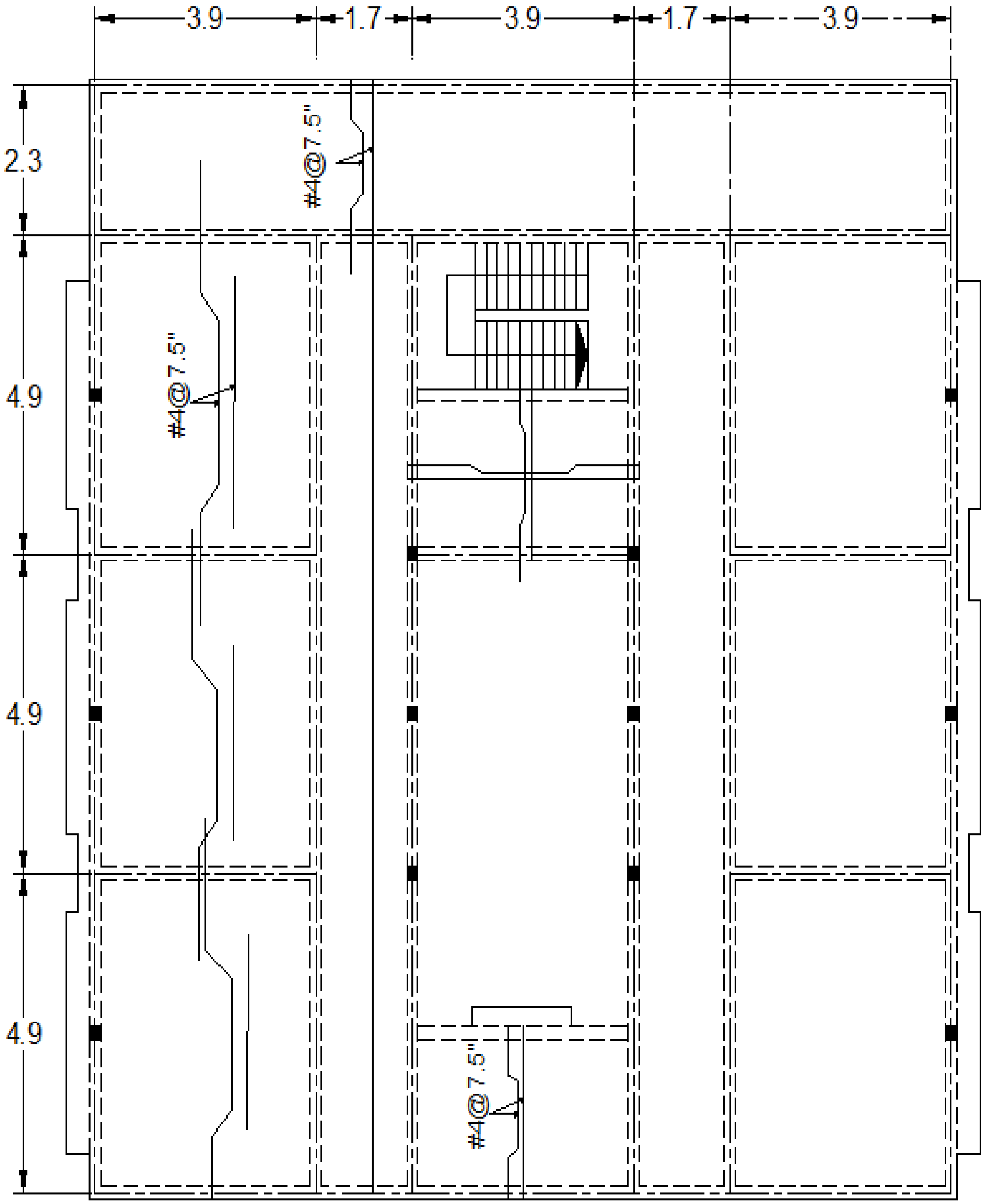
$5586.77 > 1639lb$

- o.k.

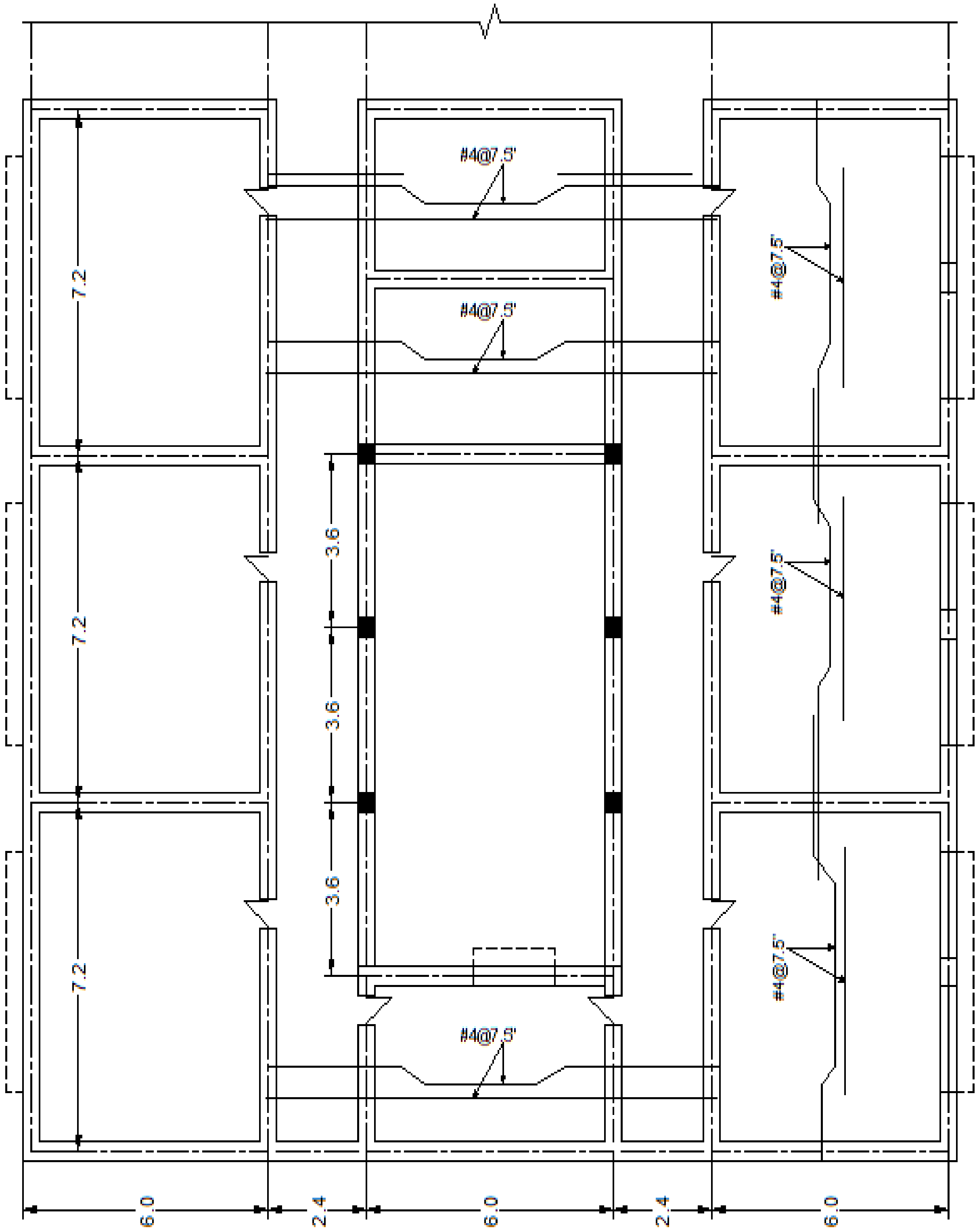
shear reinforcement not required.



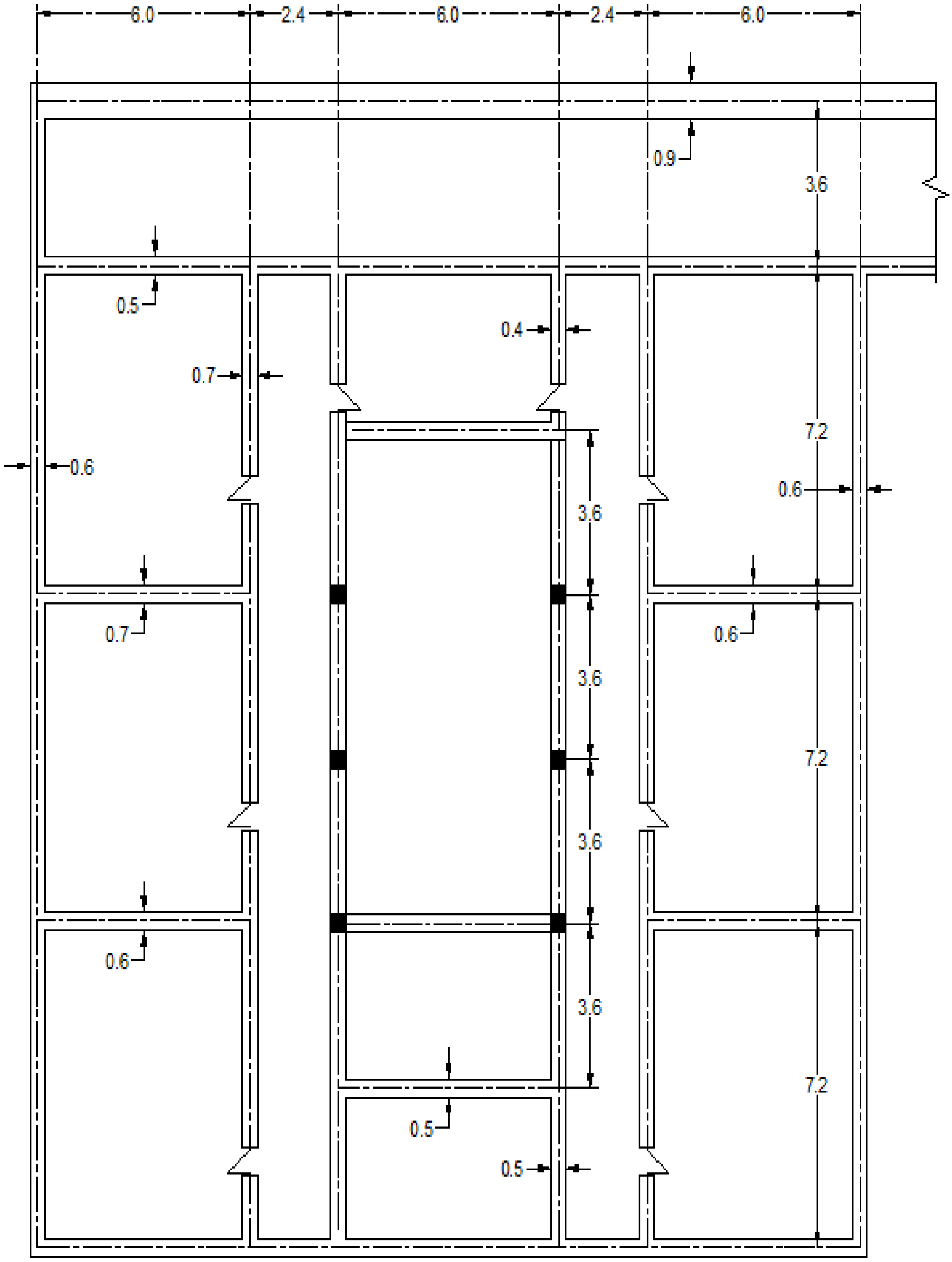
CLASSROOM DEPT.



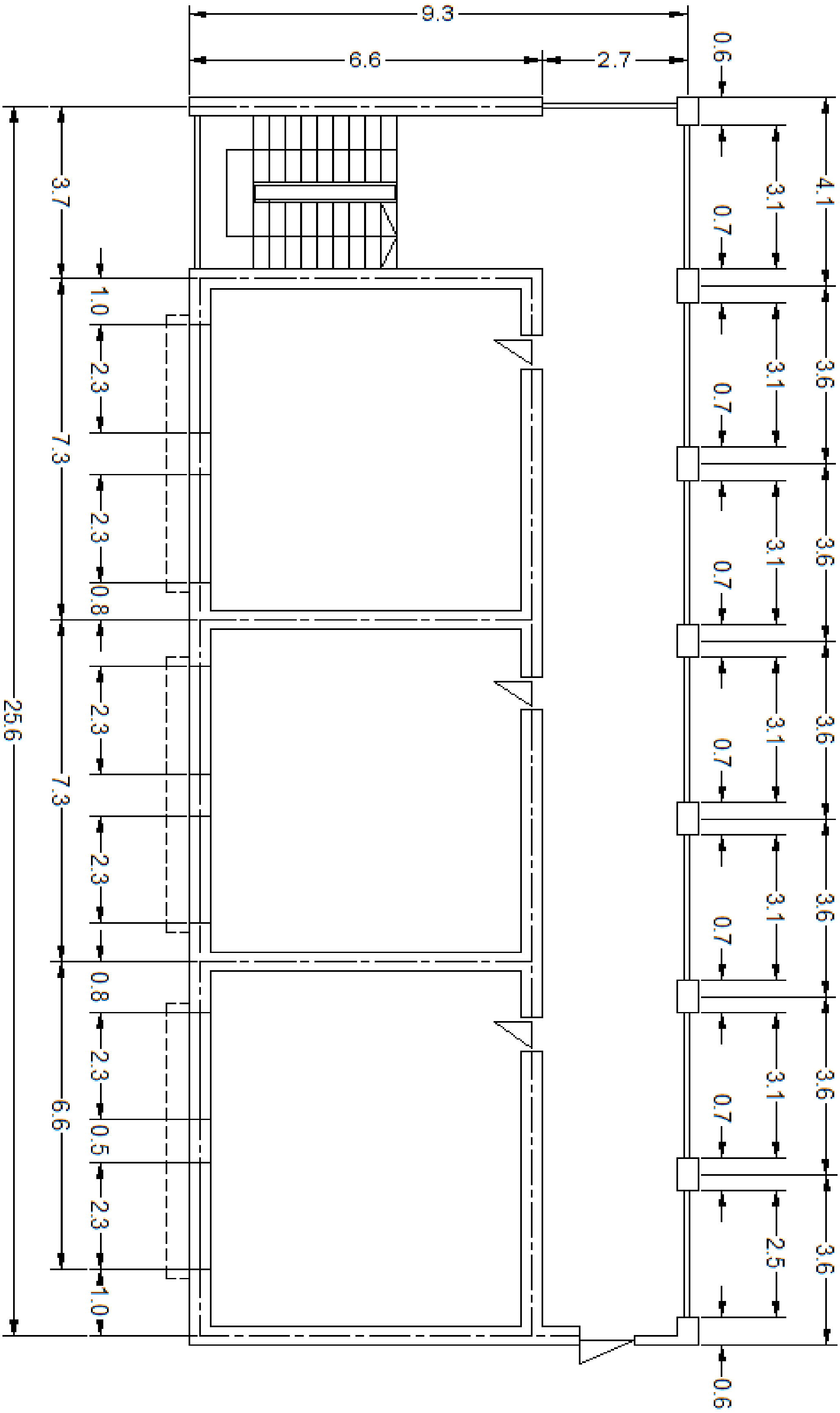
REINF.PLAN – GROUND FLOOR CLASSROOM DEPT.



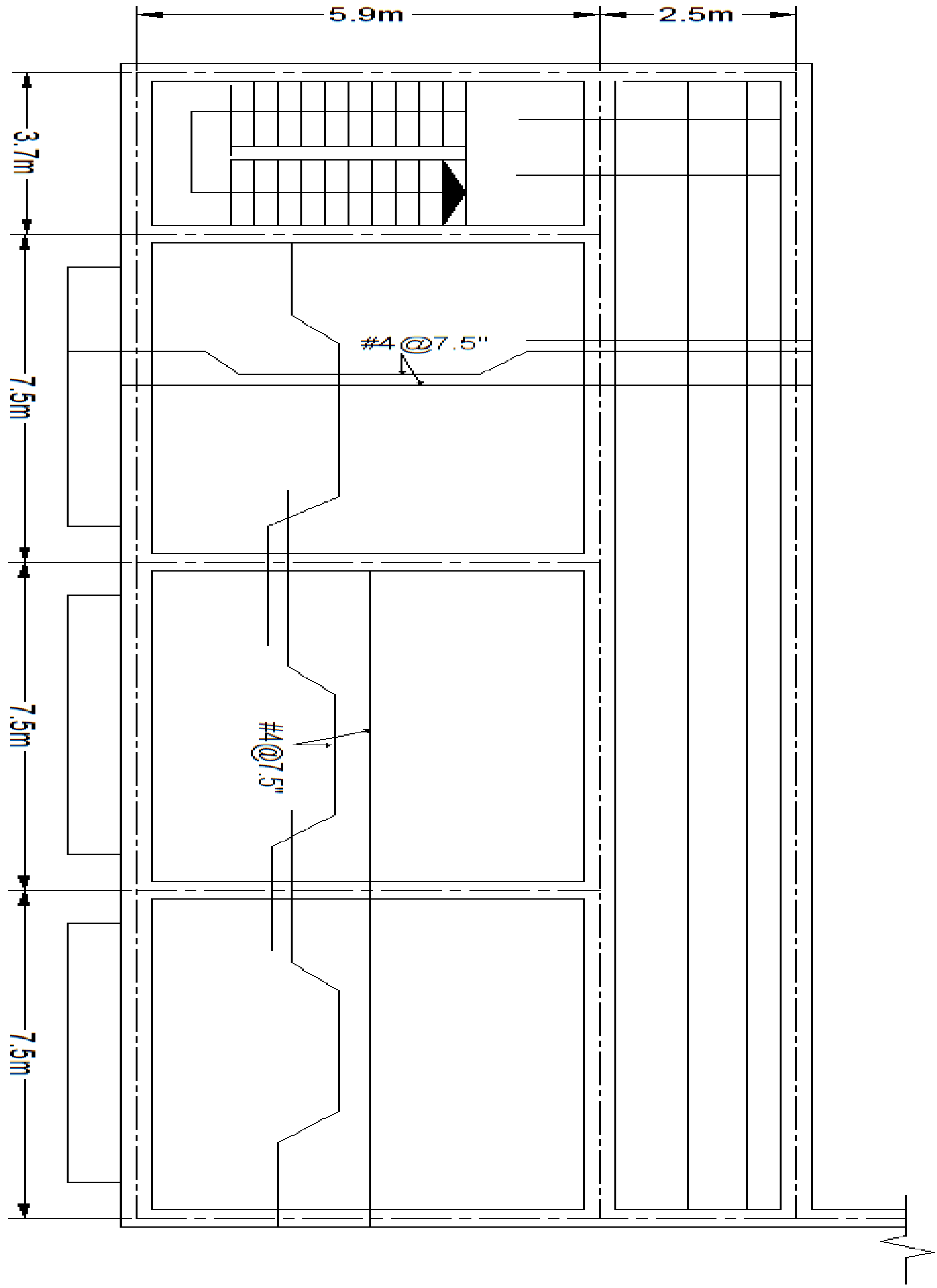
REINFORCEMENT PLAN FIRST FLOOR CLASSROOM DEPT.



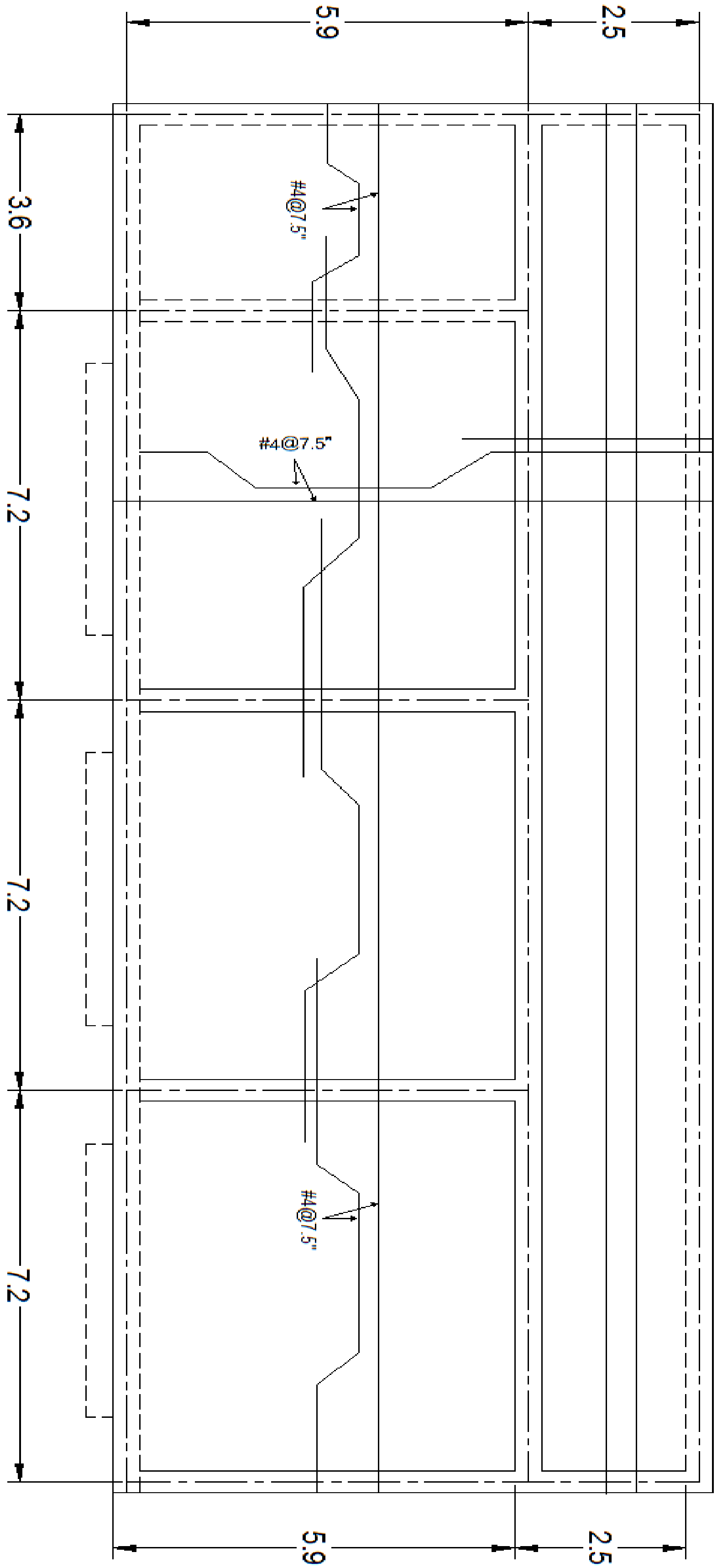
FOOTING PLAN – CLASSROOM DEPT.



FIRST FLOOR PLAN - CLASSROOM DEPT.

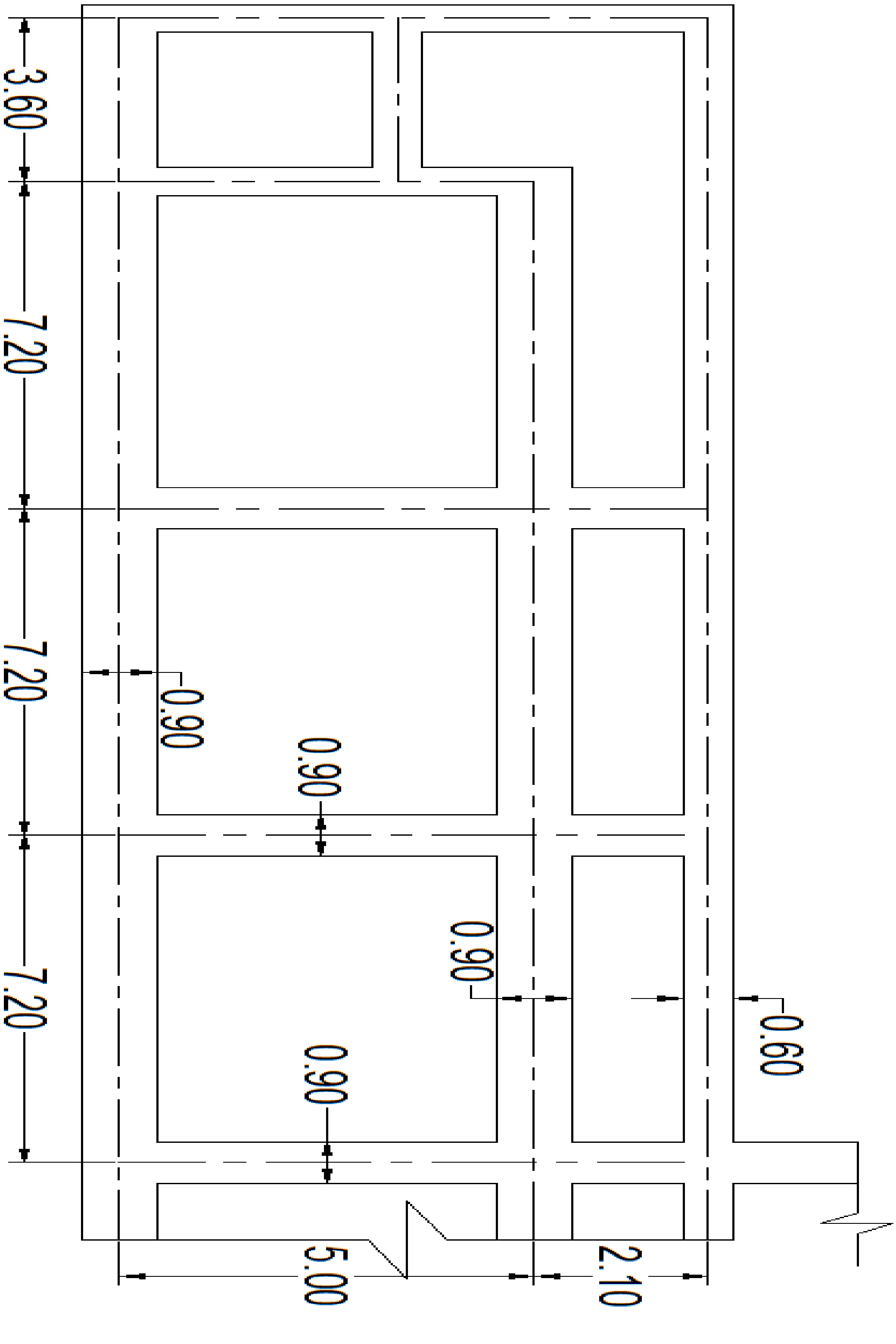


REINFT.PLAN – GROUND FLOOR CLASSROOM DEPT.



REINFT. PLAN – FIRST FLOOR CLASSROOM DEPT.

FOOTING PLAN – CLASSROOM DEPT.



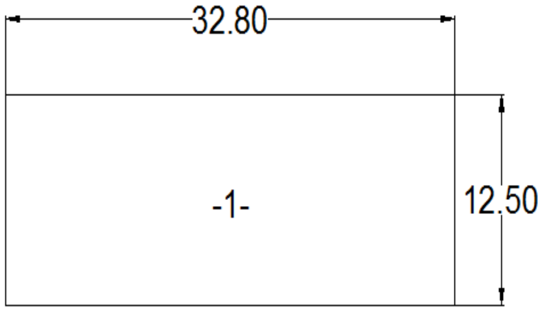
Chapter Four

Chapter Four

Health-Sport Department

Health-Sport Department

$\frac{32.8}{12.5} = 2.6$



- one way slab.

+ve Moment = $\frac{w(l)2}{24}$ = 1.69k.ft

-ve Moment = $\frac{w(l)2}{12}$ = 3.26k.ft

- Maximum moment = 3.26k.ft = 39063lb.in

$As^2 - (5.1*As) + \frac{39063}{52920} = 0$

As = 0.3in^2/ft

Use # 3 @ 4″ c/c

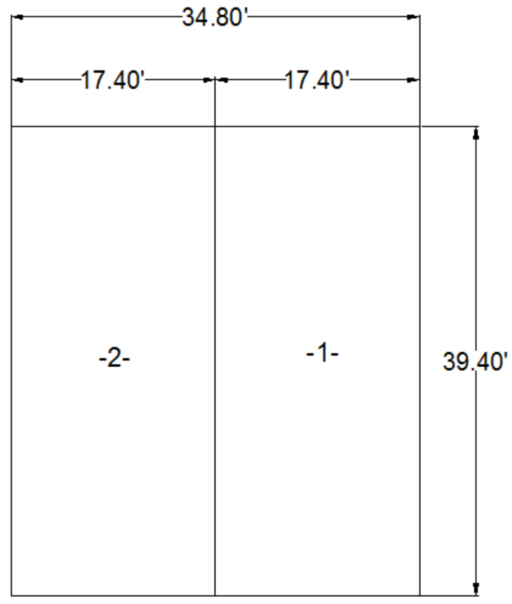
As provide = 0.33 > 0.3

- O.k.

- Use the same As for other direction.

W.C. & Hand Wash

S1:



$\frac{39.4}{17.4} = 2.3$ ❖ one way slab.

+ve Moment = $\frac{w(l)2}{24} = \frac{(0.25)*(17.4)2}{24}$ = 3.15k.ft

-ve Moment = $\frac{w(l)2}{12} = \frac{(0.25)*(17.4)2}{12}$ = 6.7k.ft

❖ Maximum moment = 6.3k.ft

$As^2 – (5.1*As) + \frac{75600}{52920} = 0$

❖ As = $0.6in^2/ft$

Use # 5 @ 6″ c/c

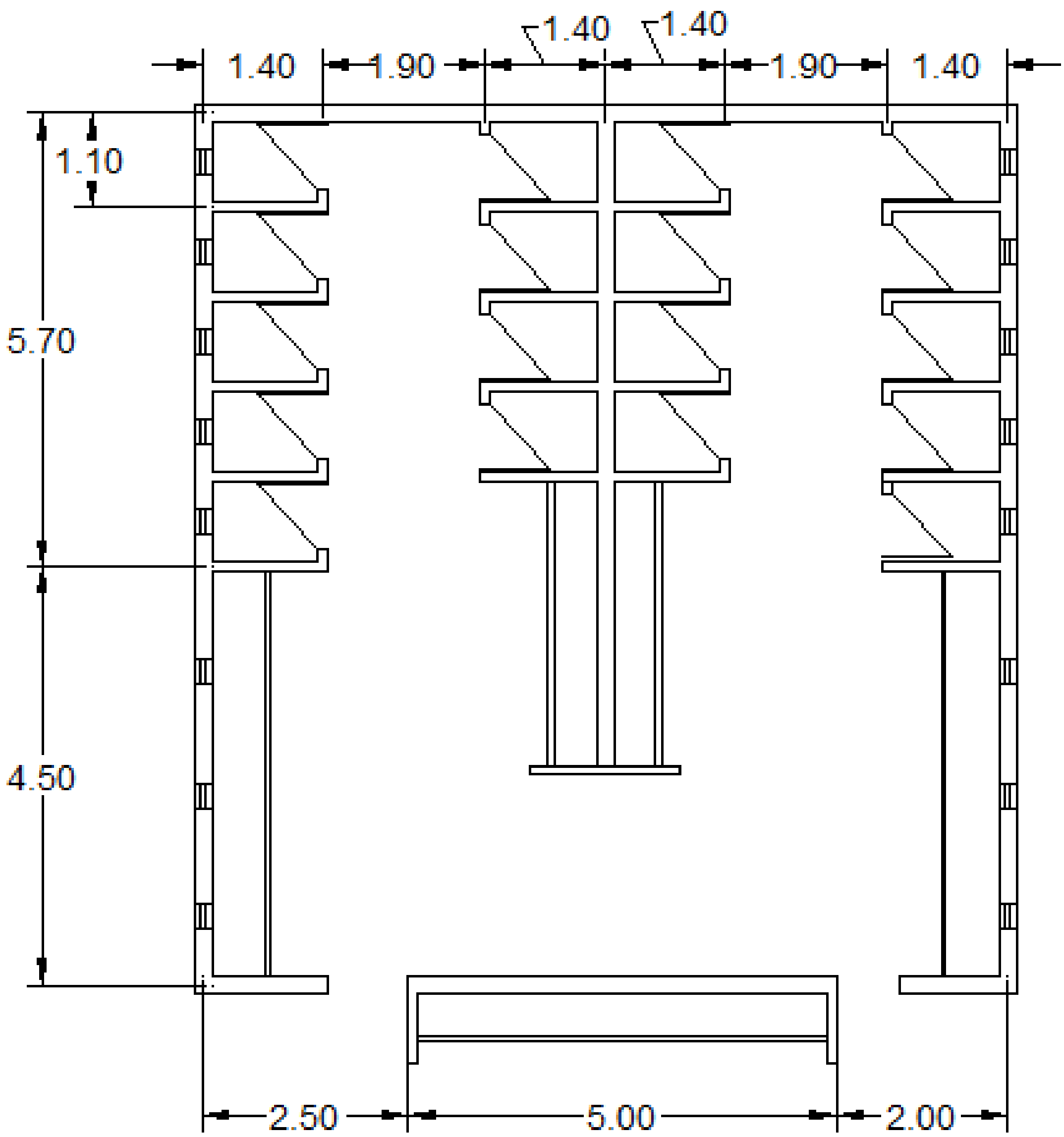
As provide = $0.61in^2/ft$

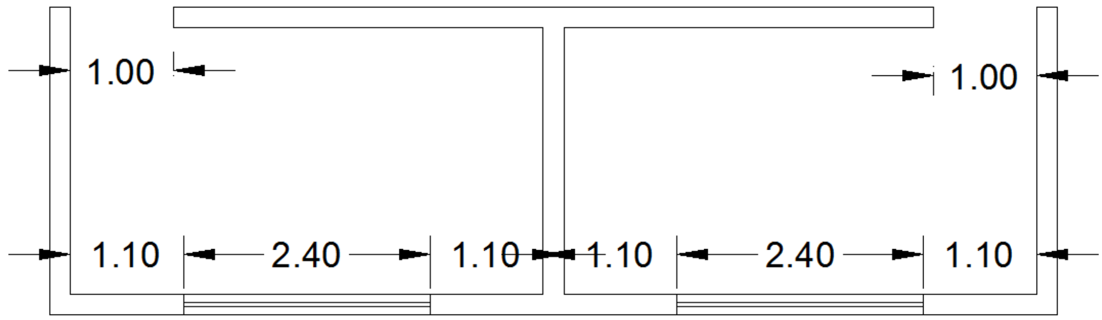
0.61 > 70.6

❖ o.k.

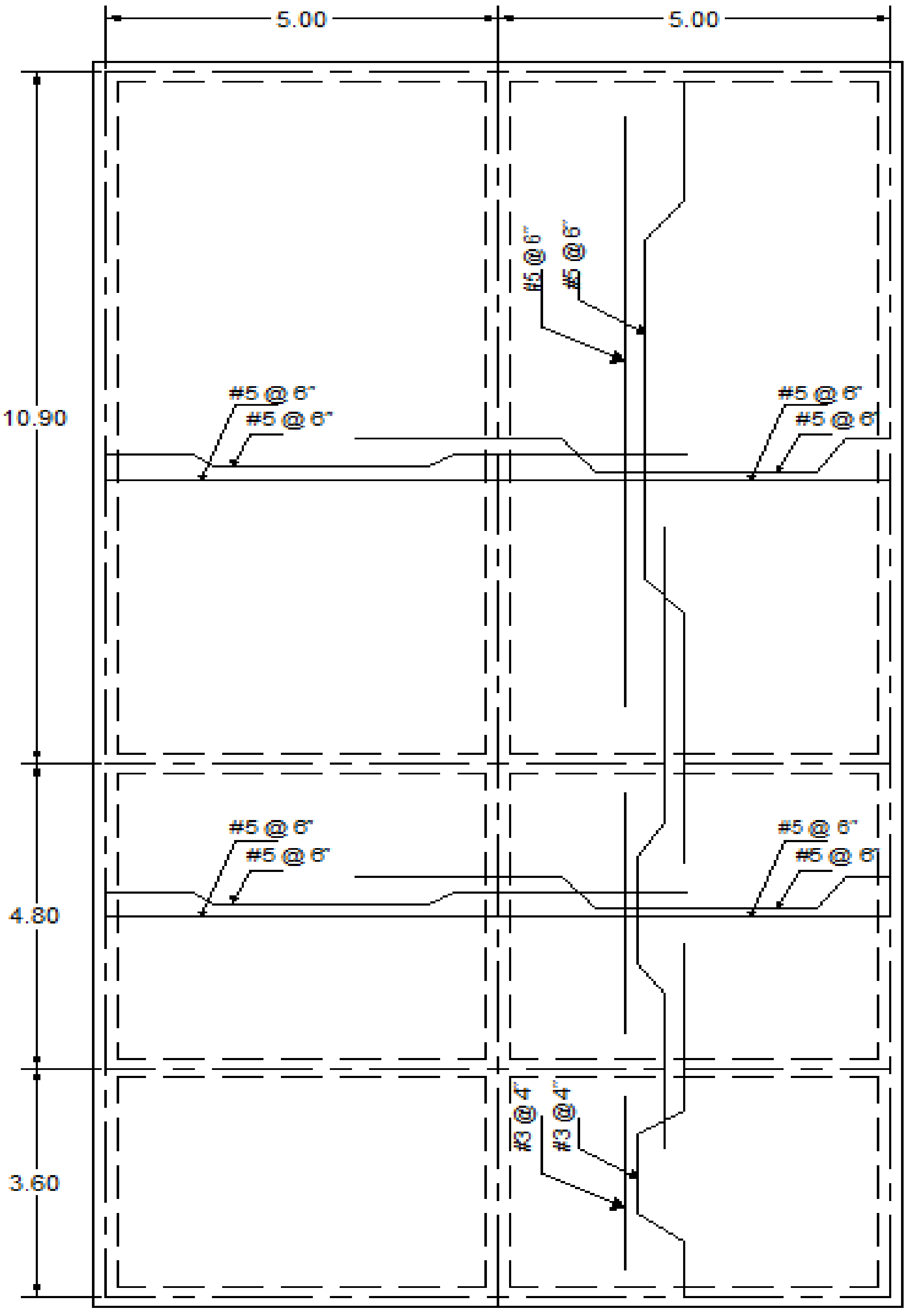
Use the same As for other direction.

And use the same as for S2.



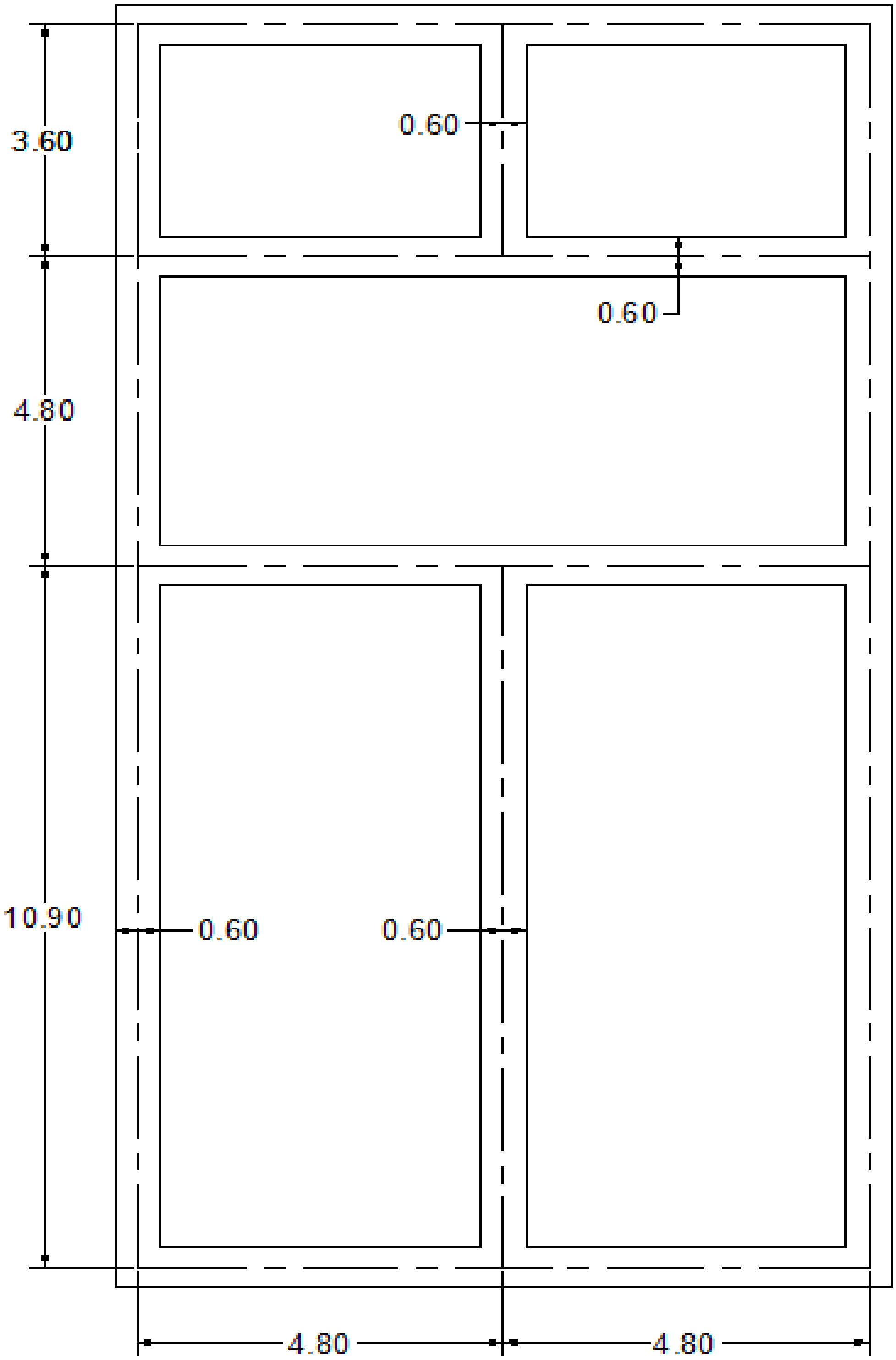


HEALTH SERVICE PLAN



REINFT. PLAN

HEALTHY SERVICE DEPT.

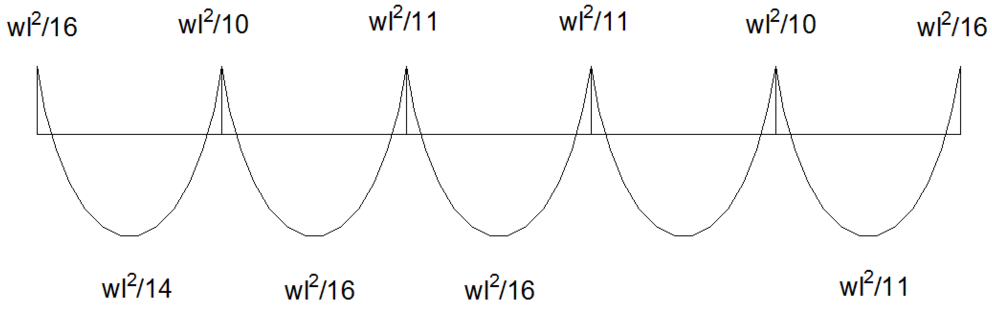


FOOTING PLAN
HEALTHY SERVICE DEPT.

Chapter Five

Design Of Lintles & Beams

According to ACI-Code 8.33 moments are approximately distributed as shown:



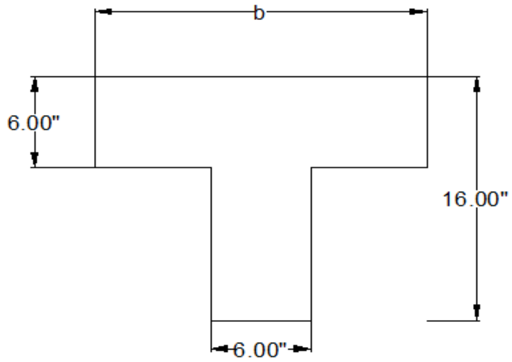
Factored L.L = 80*1.7 = 136

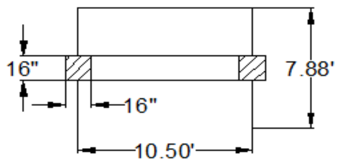
Factored D.L = 131*1.4 = 184

L.L + D.L = 136 + 184 = 320psf

Total load of tributary area:

320*10.5*7.88 = 26476.8lb





Load per 1` length = $\frac{26476.8}{10.5}$ = 2521.6lb/ft

Choose the largest moment from the diagram of moment distribution:

Maximum moment = $\frac{w(l)2}{10} = \frac{2521.6*(10.5)2}{10}$ = 27800lb.ft   = 27.8k.ft

Check for T-beam:

Span = 10.5`

Distance to next web = 7.88`

b w = 16″   d= 15″

Choose "b"

(16*h f) + (b w) = (16*6)+(16) = 112″

Span/4 = $\frac{10.5*12}{4}$ = 31.5″

Distance to next web = 7.88*12 = 94.44″

❖ adopt (b) = 31.5″

Assume h f = 6″

(d-a/2) = 15- (5/2) = 12″

$$As = \frac{Mu}{\varphi * fy * (d - \left(\frac{a}{2}\right))} = \frac{333.6}{0.90*60*12} = 0.05in^2$$

$$a = \frac{As*fy}{\varphi * fc' * b} = \frac{0.05*60}{0.85*3*16} = 0.075 < 6''$$

❖ Rectangular beam is required.

Using A-1b R_2

$$\frac{Mu}{\varphi * b * (d)2} = \frac{333600}{0.90*16*(15)2} = 102.96psi < 200psi$$

Minimum (ρ) is required by ACI-Code 10.5.1 is 0.0033 which is adequate 200psi in the graph.

Use ρ=0.0033

As = ρ*b*d = 0.0033*16*15 = 0.793in^2

Use 2#6 bars with area 0.4in^2

As provide = 2*0.44 = 0.88 > 0.793in^2 ❖ O.k.

Check for shear reinforcement choosing most critical section end member at face of first interior support, from ACI-Code 8.3.3:

$$V\ maximum = 1.15*w*(l)^2/2 = \frac{15*2521*(10.5)2}{2}\ 15224lb$$

V u = V maximum – wd

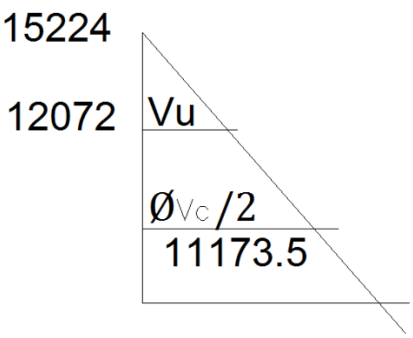
= 15224.16 – (2521.6-(15/12)) = 12072lb

$V_c = 2*\varphi*\sqrt{fc'}*b*d$

$= 2*16*15*\sqrt{3000}$

$= 26290.5lb$

$\frac{\varphi * Vc}{2} = 11173.5lb \quad < \quad 12072$



Web reinforcement is required

Use strip of #3 bar with 2-leg u strip

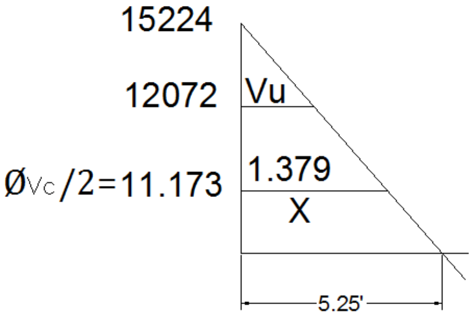
Au =2*0.11 =0.22

Check for maximum spacing ACI-Code 11.5.4.1

S maximum = $\frac{Au*fy}{50*bw} = \frac{0.22*50000}{50*16}$ 16.5"

S maximum = d/2 = 15/2 = 7.5"

S maximum = 24"



❖ adopt 7.5″ Spacing

$\frac{X}{15.224-11.173} = \frac{5.25}{15.224}$

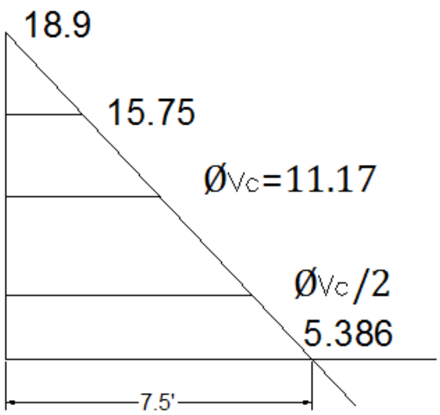
X = 1.397` = 16.76"

To distance "x" from each support beams between columns in the other points of the building.

Main iterance:

Beam section d=16" , b=8"

$Wu_{\text{total}} = 250*\frac{7.5*15}{2} = 14062.5lb$



$Wt = \frac{14062.5}{15} = 937.5lb/ft$

$Mu = \frac{w(l)2}{12} = \frac{937.5*(15)2}{12} = 1757.8lb.ft$

Mu = 210.94k.in

$\frac{Mu}{\varphi*b*(d)2} = \frac{210.94}{0.90*8*(15)2} = 130psi < 200psi$

ρ minimum provided ACI-Code 10.5.1

- ρ =0.0033

As= ρ*b*d

= 0.0033*8*15

= 0.396in^2

Use 2#4 bars with Ab=0.2in^2

As provided = 0.2*2 = 0.44in^2

Shear reinforcement:

$V\ maximum = \frac{2.52*(15)2}{2} = 18.9kip$

V u = V maximum – W d

= (18.9) – (2.52*1.25) = 15.75

$V_c = 2 * \sqrt{fc'} * b * d$

$= 2*\sqrt{3000}*8*15$

$= 13145.34lb$

$V_c = 13.145kip$

$\frac{\varphi * Vc}{2} = 5.586kip \quad < \quad 15.75kip$

- web reinforcement is required.

Select #3 bars with Ab = 0.4 for stirrups

$A_v = 2*0.11 \quad = 0.22in$

Maximum spacing ACI-Code 11.5.4.1

$$S\ maximum = \frac{Av * fy}{50 * bw}$$

$$= \frac{0.22 * 60000}{50 * 8}$$

$$= 33''$$

$$S\ maximum = \frac{d}{2}$$

$$= \frac{15}{2}$$

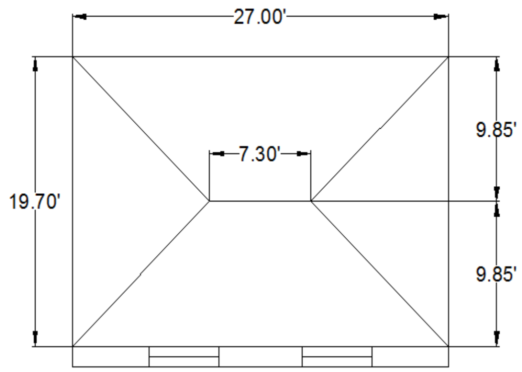
$$= 24''$$

- adopt 7.5″ as spacing.

$$S = \frac{\varphi * Av * fy * d}{Vu - Vc} = \frac{0.85 * 0.22 * 60 * 15}{15.75 - 11.17} = 36''$$

Lintels

Check the longest span of the lintels:



Factored load = 250psf

Tributary area (to AB wall) = $(\frac{7.3+27}{2}) * 9.85 = 169ft^2$

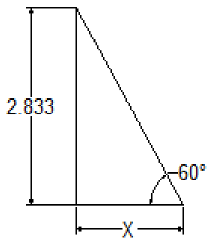
Load of tributary area = 169*250 = 42.250lb

The load of the beam under the slab = $27*\frac{8}{12}*\frac{8}{12}*150 = 1800lb$

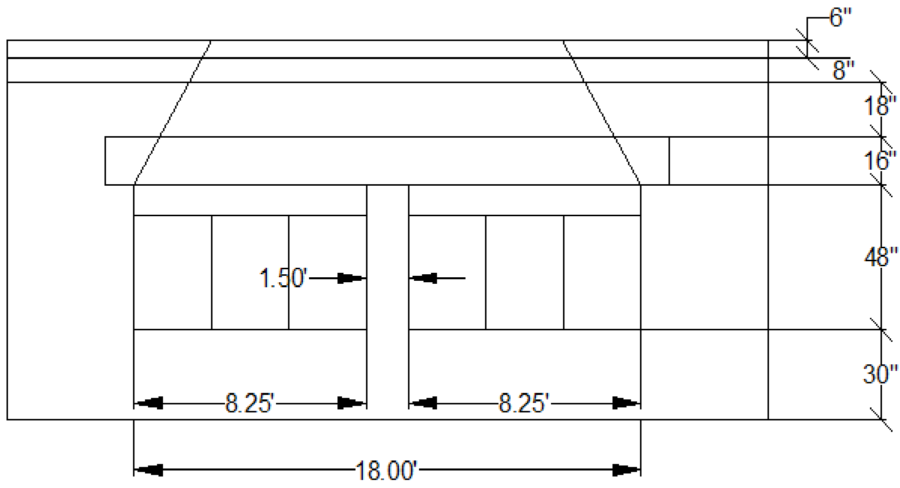
Factored load = 1.4*1800 = 2520lb

Total load = 44770lb

Total load per 1` length of the beam = $\frac{44770}{27} = 1658lb/ft$



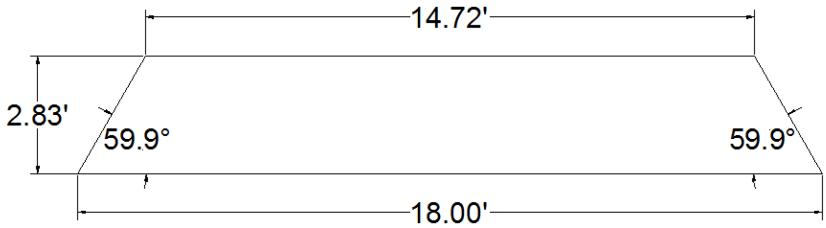
X=1.634'



Load which participate is effect load on the lintel = 1658*4*2 = 24405lb

Load of trapezoid area = $\frac{14.72+18}{2}*2.833*150*\frac{8}{12} = 4635lb$

Factored load = 6490lb



Total load act on the lintel = 6490+24405 = 48810lb = 48.81kip

Distributed loads upon lintels:

$W\ u = \frac{48.81}{18} = 2.74k/ft$

$M\ maximum = \frac{w(l)2}{9} = \frac{2.711*(8.25)2}{9} = 20.5k.ft\ \ = 246k.in$

$\frac{Mu}{\varphi*b*(d)2} = \frac{246000}{0.90*8*(15)2} = 152psi\ < \ 200psi$

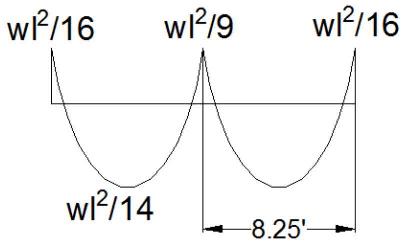
- minimum ρ is required

ρ =0.0033

As = ρ*b*d

=0.0033*8*15

= $0.396in^2$



Use 2#4 bars in tension zone which is lay up in (-ve) moment.

Totally use 4#4 bars.

Check for shear:

$V\ maximum = \frac{2.711*8.25}{2} = 11.18kip$

$V\ u = (11.81-2.711)*\frac{15}{12} = 7.8kip$

$V\ c = 2*\sqrt{3000}*b*d$

$= 2*\sqrt{3000}*8*15$

= 13145lb

$\varphi * Vc = 11.17kip$

$$\frac{\varphi * Vc}{2} = \frac{11.17}{2} = 5.586 \; < \; 7.8$$

Select #3 bars as a strip of “U” shape.

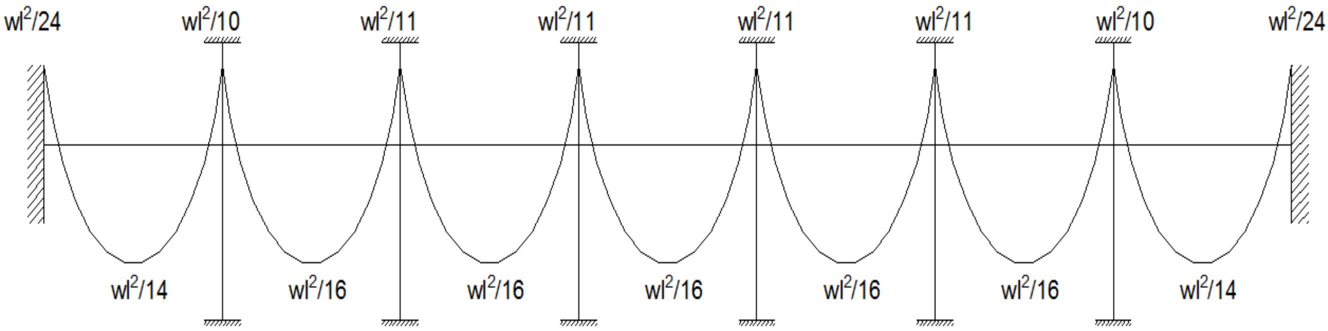
$Au = 0.22$

$$S = \frac{\varphi * Av * fy * d}{Vu - Vc} = \frac{0.85 * 0.22 * 60 * 15}{7.8}$$

Chapter Six

Design Of Columns

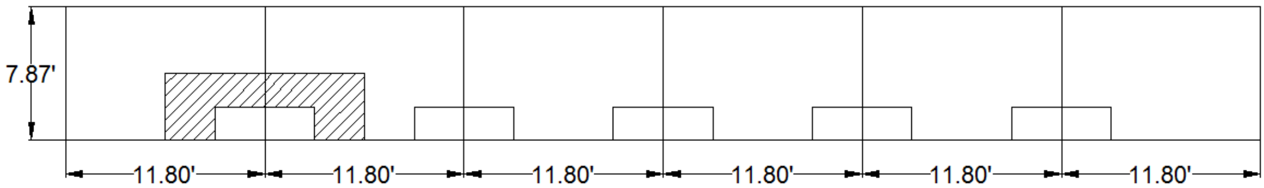
Braced columns; from table 16.1(page524)



<u>Intermediate column; (column 4):</u>

1. Axial load

a. Weight of slab = $\frac{7.87*11.8}{2} * \frac{6}{12}$ *150 = 3482.4lb



b. Load from weight of beam = $\frac{16*16}{12*12}$ * 11.8 *150 = 3146.67lb

c. Weight of column = $\frac{16*16}{12*12}$ * 9.17 *150 = 2445.34lb

- Total D.L. = 2445.34 +3482.4+3196.67 = 9079.5lb

Factored D.L. = 9079.5*1.4 = 12704.174lb

Factored L.L. = 80*1.7 = 136lb/ft^2

Total L.L. for slab = 136*$\frac{7.87*11.8}{2}$ = 6314.88lb = 6.31kip

- Total axial load = 12.7+6.31 = 19kip

Check for shear or long column:

1 . Find "K" from chart:

$$\varphi\ A = \frac{\left(\frac{\sum EI}{L}\right)\text{column } at\ A}{\left(\frac{\sum EI}{L}\right)\text{column at A}}$$

$$\varphi\ B = \frac{\left(\frac{\sum EI}{L}\right)\text{column } at\ B}{\left(\frac{\sum EI}{L}\right)\text{column at B}}$$

$$\text{I Column} = \frac{b*(h)3}{12} = \frac{16*(16)3}{12} = 5461.34\text{in}^4$$

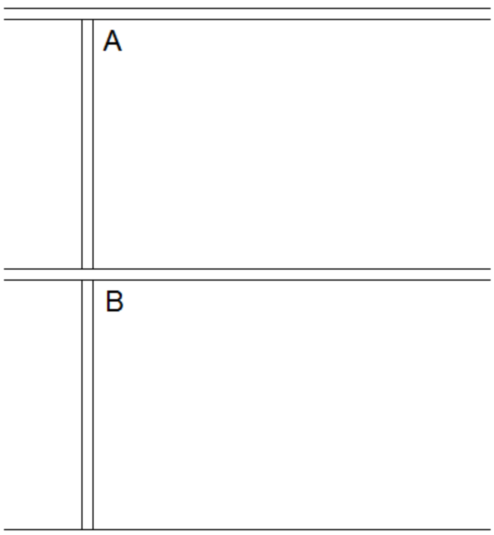
$$\text{E c} = 57000*\sqrt{fc'} = 57000*\sqrt{3000} = 3.12*10^6$$

$$\text{EI Column} = 5461.34*3.12*10^6 = 1.9*10^{10}$$

$$\text{(EI/L)Column} = \frac{1.9*(10)10}{9.17*12} = 1.7*10^8$$

$$\text{I beam} = \frac{b*(h)3}{12} = \frac{16*(16)3}{12} = 5461.34\text{in}^4$$

$$\text{(EI/L) beam} = \frac{1.9*(10)10}{11.8*12} = 1.34*(10)^8$$



❖ $\varphi\ A = \frac{1.7*(10)8}{2*1.34*(10)8} = 0.63$

❖ $\varphi\ B = \frac{2*1.7*(10)8}{2*1.34*(10)8} = 1.26$

From chart (un braced column):

K= 1.29

r= o.3*h = 0.3*16 = 4.8″

$$\frac{k*l}{r} = \frac{12*9.17}{4.8} = 28.8 > 22$$

❖ it is long Column.

Design of Long Column

According to ACI-Code – 318 – 89 assume use 4#6

As Total = $1.77 in^2$

$\mu\ 1 = 0 \quad , \quad \mu\ 2 = 0$

According to ACI-Code 89 (chapter 10 section (10.11.5.4)

$$\frac{\mu\ 1}{\mu\ 2} = 1$$

- $cm = \frac{\text{factored load}}{\text{factored total load}} = \frac{12.7}{19} = 0.66$

$E\ s = 29*10^6 psi$

$I\ s = As*(\frac{h}{2} - cover)^2$

$I\ s = 1.77*(\frac{16}{2} - 2)^2 = 63.72 in^4$

$I\ g = \frac{b*(h)3}{12} = \frac{16*(16)3}{12} = 5461.34 in^4$

$$E\ I = \frac{\left(\frac{Ec*Ig}{5}\right) + (Es*Is)}{1+Bd}$$

$$= \frac{\left(\frac{(3.12*10)6*(5461.34)}{5}\right) + (29*10)6*63.72)}{1+0.66}$$

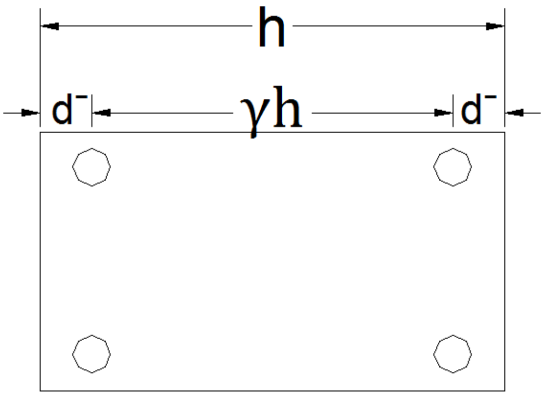
$= 3.16*10^9 psi$

$P\ c = \frac{(\pi)2*EI}{(K*lu)2} = \frac{(\pi)2*3.16*(10)9}{(1.29*9.17*12)2} = 1547769.4 lb = 1547.7 kip > P\ u$ ❖ O.K.

e = e minimum = 0.1*h = 0.1*16 = 1.6in

e/h = 1.6/16 = 0.1

$\gamma = \frac{h-(2*d`)}{h} = \frac{16-(2*2)}{16} = \frac{16-4}{16} = \frac{12}{16} = 0.75$



$\frac{Pu}{Ag} = \frac{19}{16*16} = 0.066ksi$

$\frac{Pu}{Ag} * \frac{e}{h} = 0.066*0.1 = 0.0066ksi$

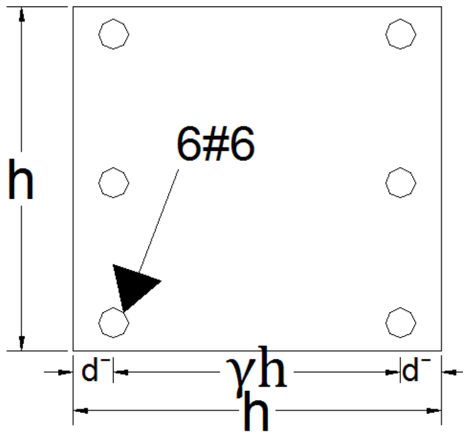
From graph A-7

ρ g minimum = 0.01

- Use ρ g minimum

As = ρ g minimum * b * h = 0.01*16*16 = 2.56in^2 > As assumed

- Use As = 2.56in^2



From table A-2 R1

Use 6#6 bars As = 2.65in^2

<u>Design the ties</u>:

Use #3 bars for ties;

Spacing [not more than (16* φ longitudinal bar)

Or not more than (48* φ Tie)

Or not more than (width of section)] which is smaller

1 . 16 * (6/8) =12″

2 . 48 * (3/8) = 18″

3 . Width = 16″

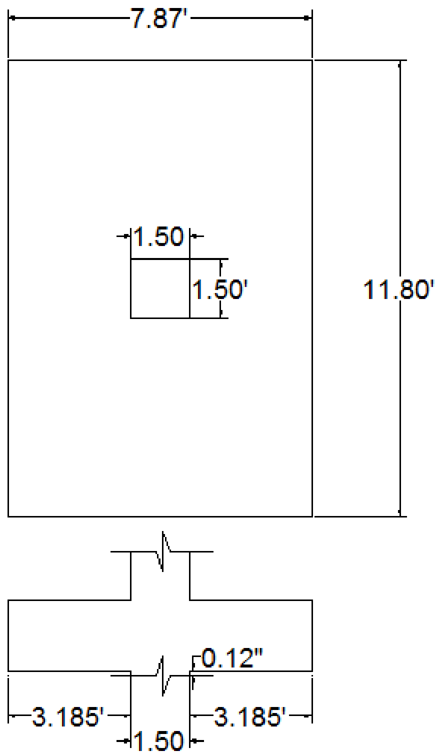
- Use 12″ @ spacing

- Number of ties = $\frac{9.17*12}{12}$ = 9.17 ≈ 9 ties

- As = 9*0.11 = 0.99in^2

Footing

Tributary area participate in the beam between the columns = 11.81*7.87= 93ft^2



Load at first slab = 93*170 = 15810lb

Load at ground slab = 93*170 = 5810lb

$\sum$ = 31620lb

Weight of the columns = 1.5*1.5*22*150 = 7425lb

Weight of the beams = [(18*12)/144]*11.8*150 = 2655lb

Total un factored load = 41700lb

$$A= B^2 = \frac{(p)\text{unfactored}}{(q)\text{unfactored}} = \frac{41700}{3000} = 13.9$$

- B = 3.37\` = 44.7″ → say 45″

Factored load = 93*250 = 23250 (at first floor & ground floor)

Factored load of columns = 10395

Factored load of beams = 3717

Total factored load $\sum$ = 60612lb

$$q \text{ factored} = \frac{(p)\text{factored}}{A} = \frac{60612}{(3.75)2} = 4310\text{psf}$$

Find thickness of the footing considering shear:

Applied shear = Resisting shear

Applied shear = $(B)^2$ * q factored – $(18+d)^2$ * q factored

$= [(45)^2 – (18+d)^2] *(4310/144)$

$= [(45)^2 – (18+d)^2] *29.93$

Resisting shear = $4 * \varphi * \sqrt{fc'} * b * d$

$= 4*0.85*\sqrt{3000}*4*(18+d)*d$

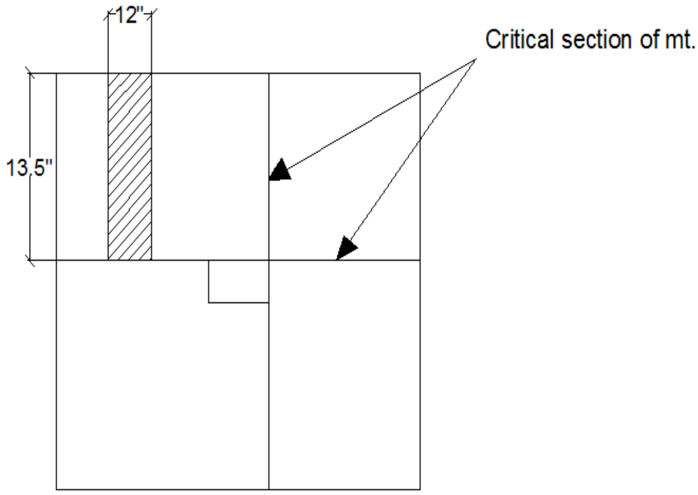
$[(45)^2 – (18+d)^2] *29.93 = 745*(18*d+d^2)$

$d^2 + 18.69*d – 65.7 = 0$

$d = 3.025'' < 6''$ adopt $d = 6''$ minimum thickness

μ = q factored * $(L^2/2) = (4310/144) * (13.5^2/2) = 2727.4$in/lb

$\frac{Mu}{\varphi * b*(d)2} = \frac{2560}{0.9*12*(6)2} = 6.6$psi



6.6 << 200psi

Use ρ minimum permitted by ACI-Code

As = ρ minimum*b*d

= 0.0033*12*6

As total = $0.891in^2$

Use #4 bars; Ab = $0.2in^2$

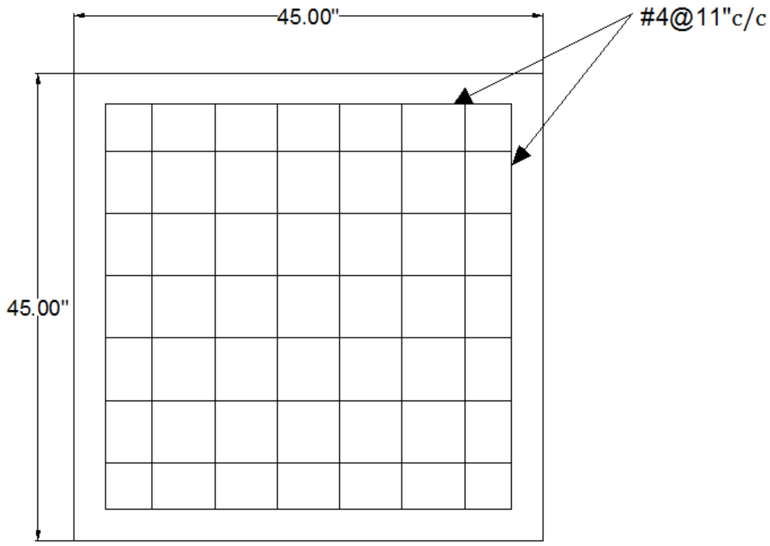
Number of bars required

= (0.891/0.2) = 4.45 bars

≈ Use 5#4 bars

As provide = 0.2*5 = $1in^2$

Spacing = (45/4) = 11″ c/c of bars



Use the same amount numbers & spacing for opposite direction.

SUMMARY

1. Depth and width of the structure's foundation is limited by "2ft" for all parts of the building. So it's adequate the limitation of shallow foundation (B equal or greater than D). Because it's alight building, commonly it reinforced as minimum steel required by the ACI-Code.

2. Thickness of the footing either wall footing and columns footing is minimum thickness permitted by ACI-Code which is 6″.

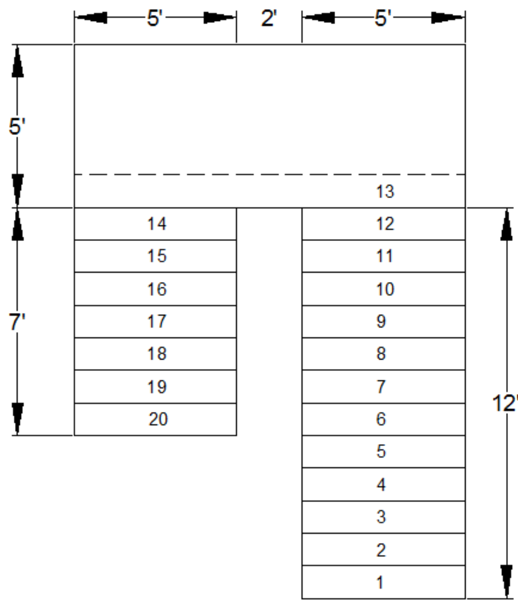
Chapter Seven

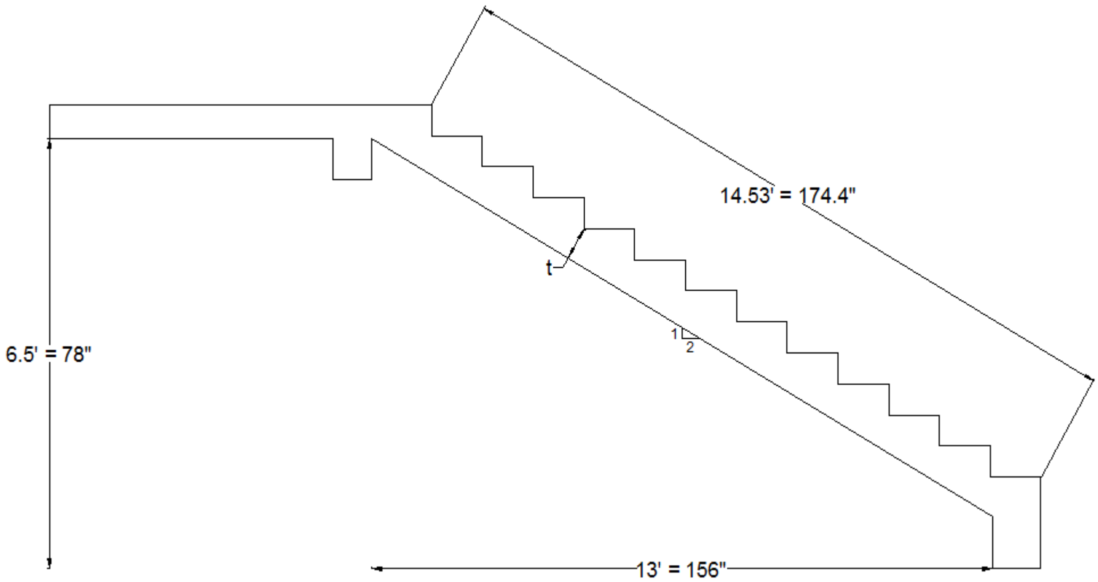
Design Of Stairs

Height of the story = 10′ = 120″

Thickness of the slab = (6″/126″) = ?

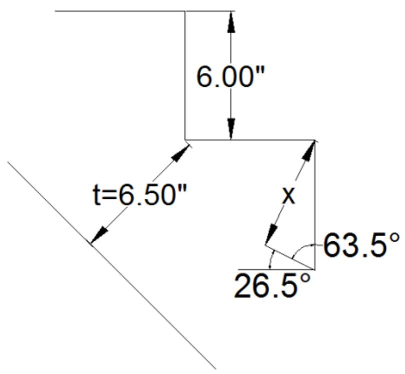
Assume R = 6″ and T = 12″ =1′





Thickness (t) = (L/24) = (13/24) = 0.541′ = 6.5″

Sin θ = x/6   ❖ x = 5.37″



Total thickness = 11.87″

D.L. = (11.87/12)*1*1*150 = 150psf

Factored D.L. = 210psf

Because of the stairs are connected with corridors treat it as partial of corridors.

❖ L.L.=80psf & factored L.L. = 80*1.7 = 136psf

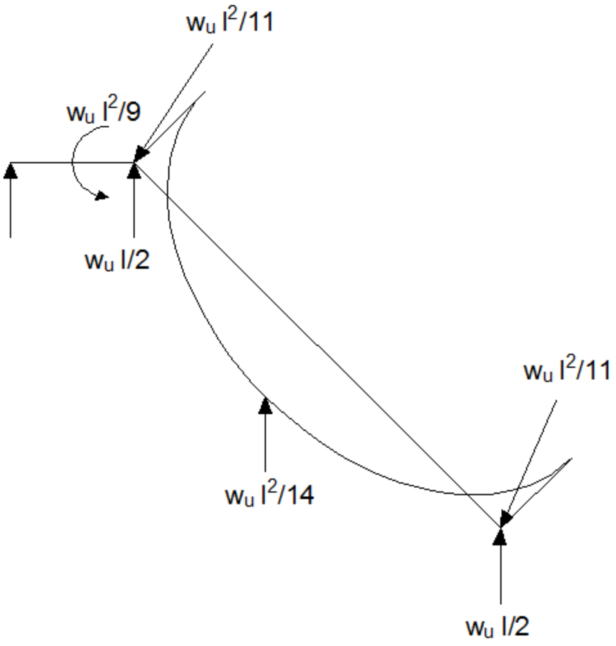
Total factored load = 346psf

Choose the largest moment design:

$M = [(Wu*L^2)/(9)]$

$= [346*(14.53)^2 / 9]$

$= 8116.4 lb.ft \quad = 97450 lb.in$



$\frac{Mu}{\varphi*b*(d)2} = \frac{97450}{0.9*12*(5.5)2} = 298psi > 200psi$

ρ = 0.0054 ; ❖ $As = 0.356in^2$

$As\ total = 1.782in^2$

Use #5 bars ; Ab= 0.31

Numbers of bars required = (1.782/0.31) = 5.75

❖ Use 6#5 bars.

$As\ provide = 6*0.31 = 1.86in^2 > 1.782in^2$

❖ O.K.

Reaction at each support = (Wu*L)/2 = (346*14.53)/2 = 2514lb/1′ length.

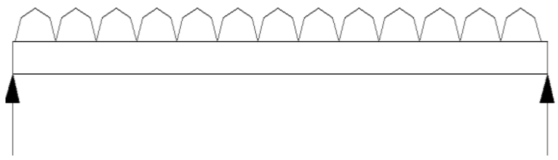
Design of Beam under Slab of the Stair Case Turn

Assume that the slab load comes to other side "rising stair" participate in beam

Load. For safety

D.L. of weight of the beam = [(10*8) /144] * 1 * 1 * 150 = 134lb/1′length

Total load per 1′ length = 2648lb



$M = [2648*(12)^2 /8] = 47669lb.ft = 571968lb.in$

$\frac{Mu}{\varphi * b * (d)2} = \frac{571968}{0.9*8*(15)2} = 353psi \quad > \quad 200psi$

$\rho = 0.0064 \quad \rightarrow \quad As = 0.768in^2$

$K = 0.3 \; ; \quad j = 0.9 \quad \rightarrow$ Check for double reinforcement

$M = \frac{fc'}{2} * k * j * b * (d)^2$

$= \frac{3000}{2} * 0.3 * 0.9 * 8 * (15)^2$

$= 72900in.lb \; > \; 571968in.lb$

- Single reinforcement is required
- O.K.

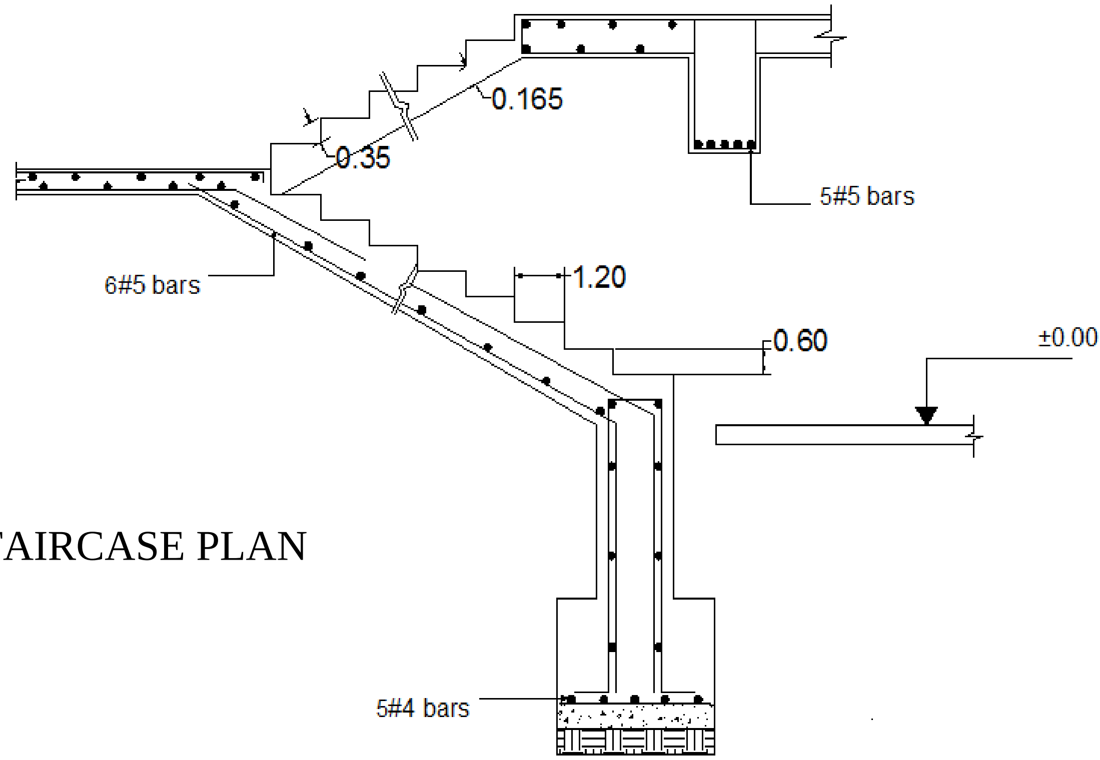
Use #5 bars $\quad As = 0.31in^2$

Number of bars $= \frac{0.768}{0.31} = 2.47$ bars

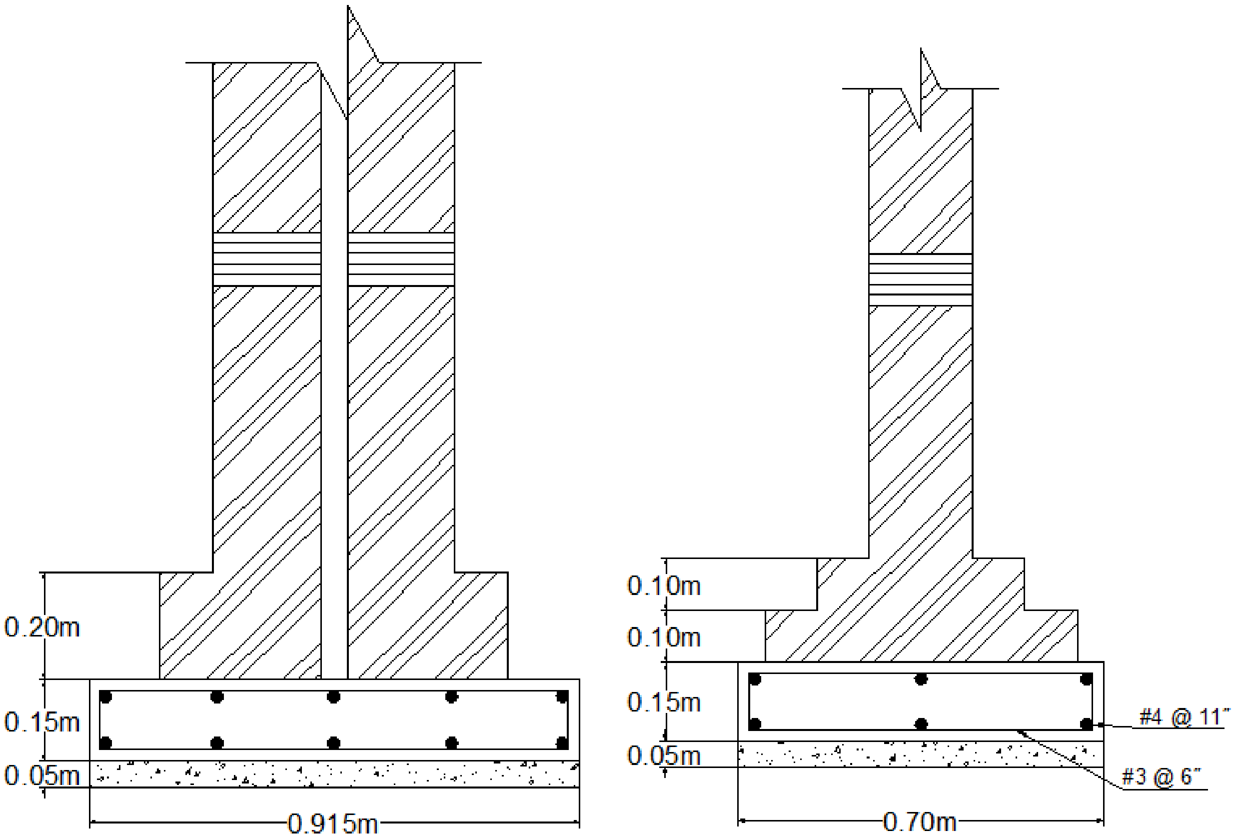
Use 2#5 bars + 1#4 bars

As provide $= (2*0.31) + (1*0.2) = 0.82in^2$

$0.82 \; > \; 0.768$ ❖ O.K.



STAIRCASE PLAN



SECTION OF FOOTINGS