A BUS TERMINAL IN JERUSALEM.

By

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INTRODUCTION TO THE PROBLEM.

(A) Statement of the problem.

The motor bus as a means of transport has gained greatly in public favour since its first appearance in Palestine, and its use will doubtless become more general in the future. Buses have many advantages that commend them for increased use. They are fast, quiet, and have unlimited flexibility in routings. In cases of unusual interruptions to the service, or other unforeseen emergency, the bus system is readily susceptible to quick recovery by diversion of routes. Buses moreover afford safety in the alighting and boarding of passengers next to the curb. The increasing importance of buses necessitates the presence of a Bus Terminal.

The term "Bus Terminal" is often misunderstood. It is usually thought to be some sort of a garage or repair shop. This apprehension is entirely false. A bus terminal is a beautiful building erected for the purpose of collecting and discharging passengers at a convenient point in town.

(B) Choice of the problem.

I have chosen a Bus Terminal as the subject for my Thesis,because to my mind it is greatly warranted in a city where such an organization is lacking. One would be provoked to note that in a city like Jerusalem, where all activities center around it, there is not yet a properly organized terminal to direct the whole passage of travellers, whether they be business men,or for leisure purposes.

One of the most important factors in a country's life is its transportation system; the implementation of which is of prime importance for the whole being and existence of that country. If its transportation facilities are inadequate then the whole country's economic and social relations would be crippled. Thus to insure a steady and normal stream of operation, an organized starting point is absolutely necessary.

From the public point of view it would be greatly desirable to improvise a bus terminal for various reasons, one of which being that it provides shelter against spasmodic changes in the weather. With such a place, the weather would no longer become an obstacle to would-be travellers. Another reason is that a traveller would prefer to start from an organized starting point, mainly because he derives from it a feeling of safety and reliability. Furthurmore it has to be noted that such an organization would entail many helpful functions and would include all the necessary particulars that pertain to the matter of travel. It furnishes the traveller with all the scheduled times of departure and arrival of the buses, and the different lines that it operates.

Last, but by no means least is the fact that a country like Palestine, and more specifically Jerusalem, is visited year after year by hundreds of tourists,who come to visit the various places and sites of historical significance. The first impression that a foreigner forms as he enters a city, is perhaps the order and regularity of traffic in that city since his primary aim of coming is to go from one place to another and from one city to another, in order to get the correct conception of those places. As such a bus terminal in the main centers of Palestine would be of essential need to create the right impression in the mind of a tourist whose first landing would be the end of the bus line, which in this case would be a Bus Terminal.

DESCRIPTION OF THE SUBJECT.

The Bus Terminal that I have planned and designed, comprises of a building 50.0m by 16.0m, surrounded by a 3.0m sidewalk. The exterior bearing walls shall consist of Limestone with a back-fill of concrete, making the total thickness of the walls 40 cms. The interior walls shall consist of cement blocks, 20 cms thick.

The Ground Floor includes a Restaurant, Baggage Room, Ticket Office, Waiting Room and Telephone Booths. There is a central staircase that leads to the first floor, which includes the Administration and the Toilet Rooms.

The parking system.

At the back of the bus terminal, there is a large parking space, enclosed by a high wall, composed of three lanes each 10.0m wide. This space is covered by means of roof trusses with moniters. The arrangement of the system of parking is clearly indicated in figure 1.

Detail of the Parking System

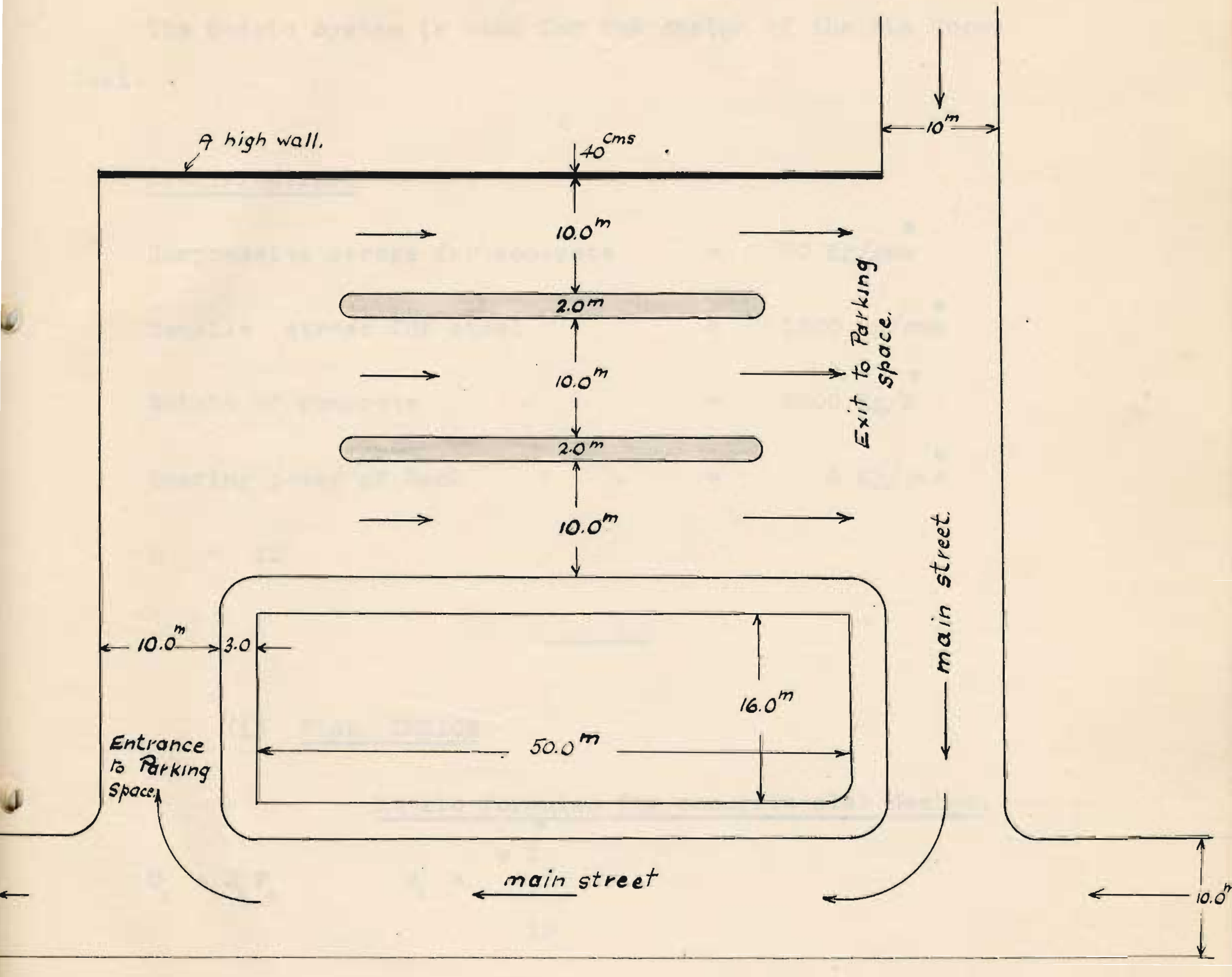


FIG-1.

DESIGN PART

The Metric System is used for the design of the Bus Terminal.

SPECIFICATIONS

Compressive stress for concrete	=	50 Kg/cms^2
Tensile stress for steel	=	1200 Kg/cms^2
Weight of concrete	=	2500 Kg/M^3
Bearing power of Rock	=	6 Kg/cms^2

$n = 12$

(1) SLAB DESIGN

Metric formulae for concrete slab design.

$U_1 = M_1 B_1$ $\qquad M_1 = \frac{w l_1^2}{10}$

$U_2 = M_2 B_2$ $\qquad M_2 = \frac{w l_2^2}{10}$

For

$n = 12,\ f_s = 1200\ Kg/cms^2 \qquad f_c = 50\ Kg/cms^2$

$h^{cms} = 0.367 \sqrt{U}$

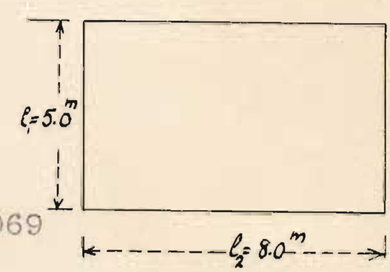
$A_s^{cms^2} = 0.694\ h$

The values of B_1 and B_2 are obtained from the tables.

Load on floor.

15cms Slab	= 15 ÷ 100 x 2500	=	375 Kgs./M^2
Assumed Live Load		=	200 "
2.50 cms Tiles	= 2.5 ÷ 100 x 2500	=	65 "
3.0 cms of Sand	= 3.0 ÷ 100 x 2000	=	60 "
			700 "

Class A 8.0 M x 5.0 M



$a = \frac{l_1}{l_2} = \frac{5.0}{8.0} = 0.62$

From the tables $B_1 = 0.772$ and $B_2 = 0.069$

$l_1 = 5.0$ M

$M_1 = 700 \times \frac{5.0^2}{10} = 1750$ Kg/ - M

$U_1 = 0.772 \times 1750 = 1350$ Kg - M

$h_1 = 0.367 \sqrt{1350} = 13.50$ cms

$A_{s1} = 0.694 \times 13.5 = 9.45$ cms^2

$l_2 = 8.0$ M

$M_2 = 700 \times 8.0^2 \div 10 = 4480$ Kg - M

$U_2 = 0.069 \times 4480 = 315$ Kg - M

$h_2 = 0.367 \sqrt{315} = 6.50$ cms

$A_{s2} = 0.694 \times 6.50 = 4.55$ cms^2

Results.

1. Have thickness of slab = 13.5 + 1.5 = 15.0 cms
2. Reinforcement for long span

   use 9 - 12 m/m ∅ per M ($A_s = 10.20$ cms^2)
3. Reinforcement for short span

   use 6 - 10 m/m ∅ per M ($A_s = 4.71$ cms^2)

<u>Class B</u> <u>5.0 M x 5.0 M.</u>

$$a = \frac{l_1}{l_2} = \frac{5.0}{5.0} = 1.0$$

$$B_1 = 0.333 = B_2$$

$$M_1 = 700 \times \frac{5.0^2}{10} = 1750 \text{ Kg - M.}$$

$$U_1 = 0.333 \times 1750 = 580 \text{ Kg - M.}$$

$$h_1 = 0.367 \sqrt{580} = 9.0 \text{ cms.}$$

$$S_1 = 0.694 \times 9.0 = 6.21 \text{ cms}^2.$$

<u>Results</u>

Have thickness of slab = 9 + 2.0 = 11.0 cms.
Reinforcement in both directions,

8 - 10 m/m ∅ per M ($As = 6.28 \text{ cms}^2$.)

<u>Class C</u> <u>10.0 M x 5.0 M.</u>

$$a = \frac{l_1}{l_2} = \frac{5.0}{10.0} = 0.5$$

$$B_1 = 0.888 \text{ and } B_2 = 0.03$$

$l_1 = 5.0$ M.

$$M_1 = 700 \times \frac{5.0^2}{10.0} = 1750 \text{ kg - M.}$$

$U_1 = 0.888 \times 1750 = 1555$ Kg - M.

$h_1 = 0.367 \sqrt{1555} = 14.0$ cms.

$S_1 = 0.694 \times 14.0 = 9.8$ cms^2.

$l_2 = 10.0$ M.

$$M_2 = 700 \times \frac{10.0^2}{10} = 7000 \text{ Kg - M.}$$

$U_2 = 0.03 \times 7000 = 210$ Kg -M.

$h_2 = 0.367 \sqrt{210} = 5.5$ cms.

$S_2 = 0.694 \times 5.5 = 3.85$ cms^2.

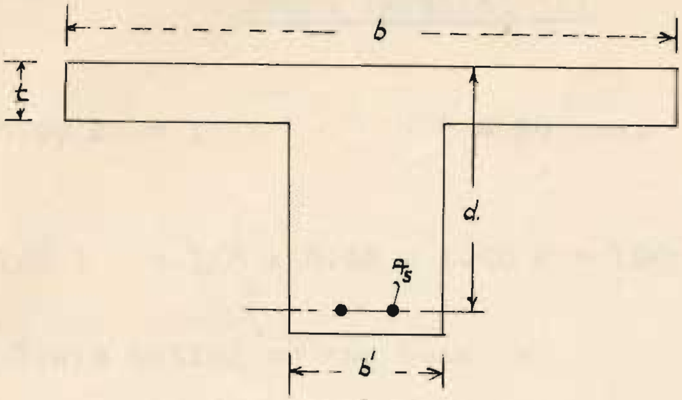
Results

1. Have thickness of slab = 14.0 + 2,0 = 16.0 cms.
2. Reinforcement for long span use,

   9 - 12 m/m ∅ per M. ($As = 10.2$ cms^2.)
3. Reinforcement for short span use,

   8 - 8 m/m ∅ per M. ($As = 4.02$ cms^2.)

(ii) BEAM DESIGN



Metric formulae for concrete T - Beam design.

$$b^{m} = \frac{1}{3}\, l$$ where l = span in meters.

$$d = 0.4 \sqrt{M \div b} \text{ cms.}$$ where M = Moment in Kg. - cms. and b = Width in cms.

$$A_s = \frac{M}{f_s\,(d - t/2)} \text{ cms}^2$$

Beams labelled (1)

Span = 5.40 M = l $\quad b' = 20$ cms.

b = 1/3 l = 1/3 x 5.40 = 1.80 M = 180 cms.

Weight of Slabs acting on the beam =

$$700 \left[5/2 \times 5/2 + \frac{8+3}{2} \times 5/2 - 3/2 \times 5/2 \right] = 11400 \text{ Kgs.}$$

Weight of wall acting on the beam = 2500 5.4 x 4.0 x 0.2 = 10800 Kgs.

Assuming weight of beam = 400 Kgs./-M

$$w = \frac{11400 + 10800}{5.40} = 4100 \text{ Kgs. / M}$$

Therefore total w = 4500 Kgs. / M

$M = 1/10\ wl^2 = 1/10 \times 4500 \times 5.4^2 \times 100 = 1{,}305{,}000$ Kgs. - cms.

$d = 0.4 \sqrt{M/b} = 0.4 \sqrt{1305000 \div 180} = 35$ cms.

Let d = 70 cms. $\quad A_s = \frac{1305000}{1200(70-15/2)} = 17.4 \text{ cms}^2$

use 9 - 16 m/m Ø $\quad (A_s = 18.09 \text{ cms}^2)$

Beams labelled (2)

$l = 5.40$ M. $b' = 20$ cms.

$b = 5.4 / 3 = 1.80$ M $= 180$ cms.

Weight of slabs = 700 (5/2 x 5/2 + 3/2 x 5/2 + 2/2 x 5/2)= 8750 Kgs.

Weight of wall = 2500(5.4 x 4.0 x 0.20) = 10800 Kgs.

Assume weight of beam = 2700 Kgs.

Total weight acting on beam = 22250 Kgs.

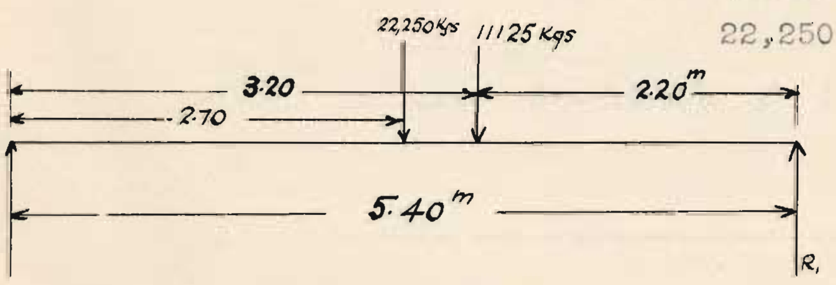
Load transmitted by beam (1)

Slabs - 700 (2 x 5/2 x 5/2) = 8750 Kgs.

Wall - 2500 (5.4 x 4.0 x 0.2)=10800 Kgs.

Beam - 1.0 x 0.2 x 2500 x 5.40 = 2700 Kgs.

22,250 Kgs.



$R_1 = (22250 \times 2.70 + 11125 \times 3.2) \div 5.4 = 17,710$ Kgs.

$M_1 = 17,710 \times 2.2 = 3,896,200$ Kgs.-cms.

$d = 0.4 \sqrt{3896200 \div 180} = 58.4$ cms. Let d = 120 cms.

$$A_s = \frac{3896200}{1200(120-15/2)} = 28.10 \text{ cms.}^2$$

use 9 - 20 m/m Ø/ ($A_s = 28.20$ cms.^{2})

Beams labelled (3)

These beams are to be designed as L - Beams.

Span = 5.2 M

For L - Beams -- $b = 1/5\ l = 1/5 \times 5.2 = 1.04$ M = 104 cms.

Weight of slabs = 700 (5/2 x 5/2) = 4375 Kgs.

Weight of Balcony balustrades = 2500 x 1.00 x 0.10 x 5.2 = 1400 Kgs,

Total weight = 5800 Kgs.

$w = 5800 \div 5.2 = 1115$ Kgs./ M

Assuming weight of beam = 200 Kgs./ M

Therefore total $w = 1315$ Kgs. / M

$M = 1/10 \times 1315 \times 5.20^2 \times 100 = 355000$ Kgs. - cms.

$d = 0.4 \sqrt{355000 \div 104} = 24$ cms.

Let $d = 30$ cms.

$$A_s = \frac{355000}{1200\ (30-15/2)} = 9.6 \text{ cms.}^2$$

use 5 - 16 m/m ϕ ($A_s = 10.05$ cms.^{2})

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Beams labelled (4)

Span = 5.40 M

$b = 5.4/3 = 1.80$ M = 180 cms. $b' = 20$ cms.

Weight of Slabs = 700 (2 x 5/2 x 5/2) = 8750 Kgs.

Assuming weight of beam = 150 Kgs. /– M

$w = 8750 \div 5.4 = 1620$ Kgs./M

Therefore total $w = 1770$ Kgs./M

$M = 1/10 \times 1770 \times 5.40^2 \times 100 = 513{,}300$ Kgs. - cms.

$d = 0.4 \sqrt{513{,}300 \div 180} = 22.0$ cms.

Let $d = 30$ cms.

$$A_s = \frac{513{,}300}{1200(30-15/2)} = 13.87 \text{ cms.}^2$$

use 6 - 18 m/m ∅ ($A_s = 15.26$ cms.^{2})

Beams labelled (5)

Span = 10.20 Meters.

b = 10.2/3 = 3.40 M. = 340 cms.

Weight of Slabs = 700(2 x 15/2 x 5/2) = 26,250 Kgs.

Assume weight of beam = 500 Kgs./M.

w = 26,250/10.2 = 2,574 Kgs./M.

Therefore total w = 3,074 Kgs. /M.

$M = 1/10 \times 3074 \times 10.2^2 \times 100 = 3,200,000$ Kgs.-cms.

$d = 0.4\sqrt{3,200,000/340} = 38.8$ cms.

Let d = 120 cms.

$$A_s = \frac{3,200,000}{1,200(120 - 15/2)} = 23.70 \text{ cms.}^2$$

use 7 - 22 m/m Ø $(A_s - 26.60 \text{ cms.}^2)$

Beams labelled (6)

(above porch)

Span = 5.00 M.

This is to be designed as an L-Beam.

For L-Beams, $b = 1/5\,l = 1/5 \times 5.0 = 1.0$M. = 100cms.

Weight of Slab $= 700(5/2 \times 5/2) = 700 \times 25/4 = 4{,}375$

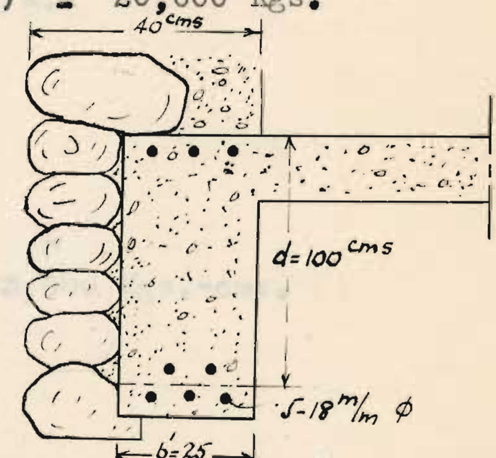
Weight of Wall $= 2500(5.0 \times 4.0 \times 0.40) = 20{,}000$ Kgs.

Total Weight = 24,375 Kgs.

$w = 24{,}375/5 = 4{,}875$ Kgs./M.

Assuming weight of beam = 500 Kgs./M.

Total $w = 5{,}375$ Kgs./M.



$M = 1/10 \times 5{,}375 \times 5.0^2 \times 100 = 1{,}343{,}750$ Kgs. - cms.

$d = 0.4\sqrt{1{,}343{,}750/100} = 46.5$ cms.

Let d = 100 cms.

$$A_s = \frac{1{,}343{,}750}{1{,}200(100 - 15/2)} = 12.70 \text{ cms.}^2$$

use 5 - 18 m/m ∅ ($A_s = 12.72$ cms.^{2})

Beam labelled (7)

Span = 6.0 M.

This beam is to be designed as an L-Beam.

For L-Beams b = 6/5 = 1.20 M. = 120 cms.

Weight of slab = 700(5/2 x 5/2) = 4,375 Kgs.

Weight of wall = 2,500(6.0 x 4.0 x 0.4) = 24,000 Kgs.

Total weight acting on beam = 28,375 Kgs.

Therefore w = 28,375/6 = 4,730 Kgs./M.

Assuming weight of beam = 500 Kgs./M.

Therefore total w = 5,230 Kgs./M.

$M = 1/10 \times 5,230 \times 6.0^2 \times 100 = 1,883,000$ Kgs.-cms.

$d = 0.4\sqrt{1,883,000/120} = 50.0$ cms.

Let d = 100 cms.

$$A_s = \frac{1,883,000}{1,200(100 - 15/2)} = 16.96 \text{ cms.}^2$$

use 6 - 20 m/m Ø (A_s = 18.84 cms.^{2})

WEB REINFORCEMENT.

For beams labelled (1)

Spacing of Stirrups $= \dfrac{A_v f_s j d}{V'}$

Where A_v = Cross-sectional area of Stirrups.

Shear taken care by the concrete

$$V = V_c\, b\, j\, d = 6 \times 20 \times 0.88 \times 70 = 7390 \text{ Kgs.}$$

V' = Total shear - V

$V' = 4500 \times 5.40 \div 2 - 7390 = 4{,}760$ Kgs.

$A_v = 1.69$ cms.^{2} (6-6 m/m prongs)

$$S = \frac{1.69 \times 1200 \times 0.88 \times 70}{4{,}760} = 25.2 \text{ cms.}$$

Make the spacing of Stirrups = 25 cms.

For beams labelled (2)

$V = V_c\, b\, j\, d = 6 \times 20 \times 0.88 \times 120 = 12{,}670$ Kgs.

Maximum Reaction = 44,500 - 17,710 = 26,800 Kgs.

$V' = 26{,}800 - 12{,}670 = 14{,}130$ Kgs.

$$S = \frac{1.69 \times 1200 \times 0.88 \times 120}{14{,}130} = 15.2 \text{ cms.}$$

Make the spacing of Stirrups = 15 cms.

For beams labelled (3)

$V = 6 \times 20 \times 0.88 \times 30 = 3{,}170$ Kgs.

$V' = 1315 \times 5.2 \div 2 - 3170 = 249$ Kgs.

Using 6 - 6 m/m prongs

$$S = \frac{1.69 \times 1200 \times 0.88 \times 30}{250} = 215 \text{ cms.}$$

Make the spacing of the Stirrups = d = 30 cms.

For beams labelled (4)

$V = 6 \times 20 \times 0.88 \times 30 = 3{,}170$ Kgs.

$V' = 1770 \times 5.4 \div 2 - 3170 = 1610$ Kgs.

Using 6 - 6 m/m prongs

$$S = \frac{1.69 \times 1200 \times 0.88 \times 30}{1610} = 33.5 \text{ cms.}$$

Make the spacing of the Stirrups = d = 30 cms.

For beams labelled (5)

$V = 6 \times 20 \times 0.88 \times 120 = 12{,}670$ Kgs.

$V' = 3074 \times 10.2 \div 2 - 12{,}670 = 3010$ Kgs.

Using 6 - 6 m/m prongs

$$S = \frac{1.69 \times 1200 \times 0.88 \times 120}{3010} = 71.0 \text{ cms.}$$

Make the spacing of Stirrups = 70 cms.

For beams labelled (6)

V = 6 x 25 x 0.88 x 100 = 13,200Kgs.

$V' = 5375 \times 5.0 \div 2 - 13,200 = 13,440$ Kgs.

Using 6 - 6 m/m prongs

$$S = \frac{1.69 \times 1200 \times 0.88 \times 100}{240} = 750 \text{ cms.}$$

Make the spacing of Stirrups = d = 100 cms.

For beams labelled (7)

V = 6 x 25 x 0.88 x 100 = 13,200 Kgs.

$V' = 5230 \times 6.0 \div 2 - 13,200 = 2,490$ Kgs.

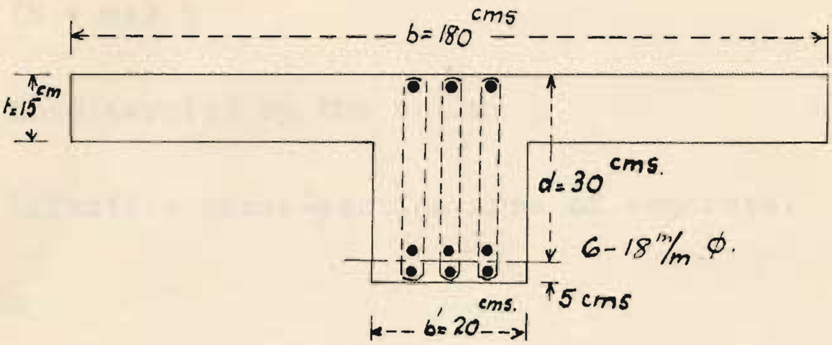
Using 6 - 6 m/m prongs

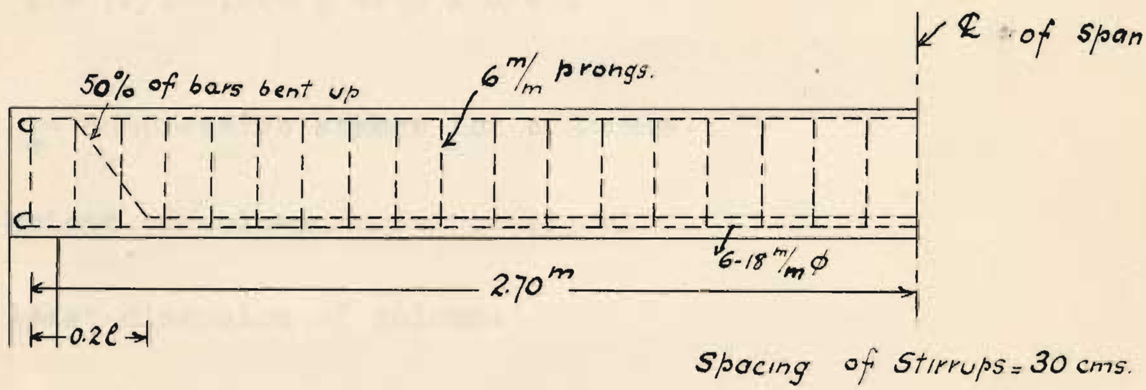
$$S = \frac{1.69 \times 1200 \times 0.88 \times 100}{2490} = 72 \text{ cms.}$$

Make the spacing of Stirrups = 70 cms.

The detail of a beam is found on page 22.

DETAIL OF BEAMS LABELLED (4).





(iii) Column Design

Metric formulae for concrete column design.

$$Q = (S + ms)\, r_1$$

where Q = Load carried by the column

S = Effective cross-section area of concrete.

m = n

s = Area of steel.

$$r_1 = \frac{r}{1 + (r/100{,}000 \times 0.75 \times h/a)^2}$$

where $r = f_c$ = compressive stress for concrete.

h = height of column

a = least dimension of column.

For n = m = 12

and p = percentage of steel = 0.5%

$$Q = (S + 0.06S)\, r_1$$

Therefore $Q = 1.06\, S\, r_1$

(a) First Floor Columns.

Columns labelled (A)

Load carried by the column

$Q = 4500 \times 5.2/2 + 4500 \times 5.2/2 + 26,800$

$Q = 50,200$ Kgs.

$h/a = 4.0/0.30 = 13.3$ (say 14)

Therefore r_1 (which is obtained from the tables) = 46.6

$S = 50200/46.6 = 1077$ cms².

Assume section of column = 30 cms x 36 cms (effective area)

$A_s = 0.5/100 \times 1080 = \underline{\underline{5.40}}$ cms²

use 6 - 12 m/m ∅ ($A_s = 6.78$ cms²)

Column labelled (B)

Load carried by the columns

$Q = 4500 \times 5.2/2 + 4500 \times 5.2/2 + 1770 \times 5.2/2 + 4500 \times 5.2/2$

$Q = 39,700$

$h/a = 4.00 / 0.30 = 13.3$ (say 14)

Therefore $r_1 = 46.6$

$S = 39700/46.6 = 852 cms^2$ (effective area)

Assume section = 30 cms x 30 cms. (effective area)

$A_s = 0.5/100 \times 900 = \underline{\underline{4.50}} \ cms^2$

use 6 - 10 m/m ∅ ($A_s = 4.71 \ cms^2$)

Columns labelled (C)

Load carried by the column

$Q = 1315 \times 5.0/2 + 1315 \times 5.0/2 + 1770 \times 3.0/2 + 3074 \times 10/2$

$Q = 24,910$ Kgs. (say 25,000 Kgs.)

$h/a = 4.0/0.2 = 20$

Therefore $r_1 = 43.5 \ kgs./cms^2$.

$S = 25,000/43.5 = 575 \ cms^2$.

Assume section of column = 20 cms x 30 cms.(effective area)

$A_s = 0.5/100 \times 600 = \underline{\underline{3.0}} \ cms^2$.

use 6 - 8 m/m ∅ ($A_s = 3.01 \ cms^2$)

(b) Ground Floor Columns

Columns labelled (A)

Load carried by the column

$Q = 50{,}200 \times 2 + (4.0 \times 0.35 \times 0.41 \times 2500)$

$Q = 101{,}900$ Kgs. (say 102,000 Kgs.)

$h/a = 4.0/0.4 = 10$

Therefore $r_1 = 48.2$

$S = 102{,}000/48.2 = 2120$ cms².

Assume section of column = 40 cms x 55 cms.

$A_s = 0.5/100 \times 2200 = \underline{\underline{11.0}}$ cms².

use 6 - 16 m/m ∅ ($A_s = 12.06$ cms²)

Columns labelled (B)

Load carried by the column

$Q = 39{,}700 \times 2 + 4.0 \times 0.35 \times 0.35 \times 2500$

$Q = 80{,}650$ Kgs.

$h/a = 4.0/0.4 = 10$

Therefore $r_1 = 48.2$

$S = 80{,}650/48.2 = 1673$ cms²

Assume section of column = 40 cms x 42 cms (effective area)

$A_s = 0.5/100 \times 1680 = \underline{8.40}$ cms ^{2}

use 6 - 14 m/m ∅ $(A_s = 9.23 \text{ cms}^2)$

Columns labelled (C)

Load carried by the column

$Q = 25,000 \times 2 + 4.0 \times 0.25 \times 0.35 \times 2500$

$Q = 50,875$ Kgs.

$h/a = 4.0/0.3 = 13.3$ (say 14)

Therefore $r_1 = 46.6$

$S = 50,875/46.6 = 1092 \text{ cms}^2$

Assume section of column = 30 cms x 40 cms (effective area)

$A_s = 0.5/100 \times 1200 = \underline{6.0} \text{ cms}^2$

use 6 - 12 m/m ∅ $(A_s = 6.78 \text{ cms}^2)$

(c) Underground Columns

Columns labelled (A)

Load carried by the column

$Q = 50,200 \times 2 + 12,960 + 13500/2 + 8.0 \times 0,4 \times 0.5 \times 2500$

$Q = 124,100$ Kgs.

$h/a = 2.0/0.5 = 4$

Therefore $r_1 = 50$ Kgs/cms^2

$S = 124,100/50 = 2482$ cms^2

Assume section of column = 50 cms x 50 cms (effective area)

$A_s = 0.5/100 \times 2500 = \underline{12.5}$ cms^2

use 6 - 18 m/m Ø

Columns labelled (B)

Load carried by the column

$Q = 39,700 \times 2 + 12960 \times 3/2 + 8.0 \times 0.3 \times 0.4 \times 2500$

$Q = 101,300$ Kgs.

$h/a = 2.0/0.4 = 5.0$

Therefore $r_1 = 50$ Kgs/cms^2

$S = 101{,}300/50 = 2{,}025 \text{ cms}^2$

Assume section of column = 45 cms x 45 cms (effective area)

$A_s = 0.5/100 \times 2025 = \underline{\underline{10.12}} \text{ cms}^2$

use 6 - 16 m/m ∅ ($A_s = 12.06 \text{ cms}^2$)

Columns labelled (C)

Load carried by the column

$Q = 25{,}000 \times 2 + 8.0 \times 0.3 \times 0.4 \times 2500$

$Q = 52{,}500$ Kgs.

$h/a = 2.0/0.35 = 5.7$ (say 6)

Therefore $r_1 = 49.3 \text{ Kgs/cms}^2$

$S = 52{,}500/49.3 = 1065 \text{ cms}^2$

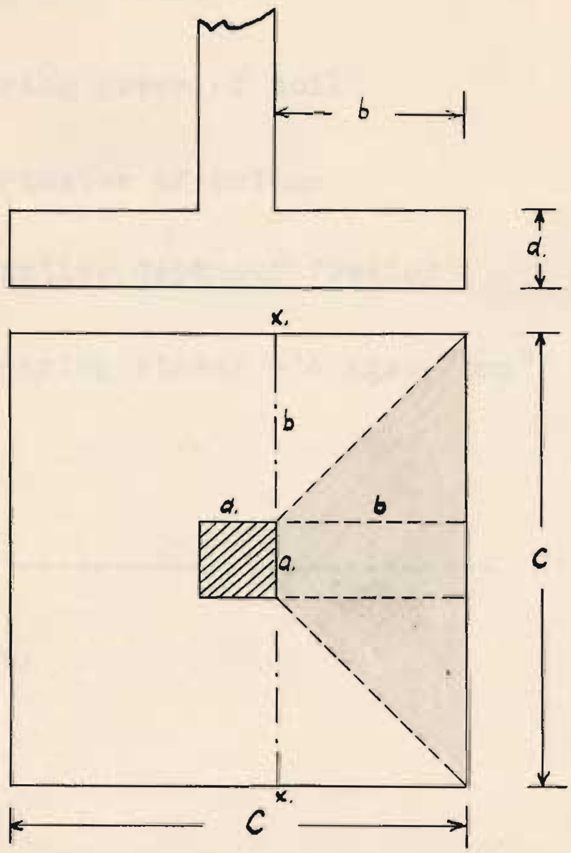
Assume section of column = 35 cms x 35cms (effective area)

$A_s = 0.5/100 \times 1225 = \underline{\underline{6.12}} \text{ cms}^2$

use 6 - 12 m/m ∅ ($A_s = 6.78 \text{ cms}^2$)

(iv) FOOTING DESIGN

Metric formulae for concrete footing design



$M_{x-x} = w/2(a + 1.2b)\ b^2$ Where w = bearing power of soil

Depth of footing below the ground

$Y = P/W(1 - Sin\phi \div 1 + Sin\phi)^2$

Where- P= unit vertical pressure

w= bearing power of soil

ϕ= angle of repose

$d = (A - a)w / P_e \times S_s$

Where - A = Area of footing

a = Area of column

w = Bearing power of soil

P_e = Perimeter of column

d = Effective depthoof footing

S_s = Shearing stress = 6 Kgs. /cms^2

For columns labelled (A)

Required area of footing = 124,200/6 = 20,700 cms^2

Have section of footing = 140 cms x 140 cms

Therefore total dimension of footing with 5 cms of covering

on each side = 150 cms x 150 cms

$M_{x-x} = 6/2\ (55 + 1.2 \times 47.5)\ 47.5^2 = 758,000$ Kgs. - cms.

$d = (22,500 - 3,025)6\ /\ 4 \times 55 \times 6 = 88.5$ cms.

$A_s = M_{x-x}\ /f_s\ jd = 758,000/1200 \times 0.88 \times 88.5 = \underline{\underline{8.09}}$ cms^2

use 6 - 14 m/m ∅ per M. ($A_s = 9.23$ cms^2)

For columns labelled (B)

Required area of footing = 102,000/6 = 17,000 cms^2

Have section of footing = 130 cms x 130 cms.

Therefore total dimension of footing with 5 cms of covering

on each side = 140 cms x 140 cms.

$M_{x-x} = 6/2(50 + 1.2 \times 45)45^2 = 637,900$ Kgs. - cms.

$d = (19,600 - 2,500)6\ /\ 4 \times 50 \times 6 = 85.5$ cms

$A_s = 637,900 / 1200 \times 0.88 \times 85.5 = \underline{\underline{7.08}}$ cms^2

use 5 - 14 m/m Ø per M. ($A_s = 7.69$ cms^2)

<u>For columns labelled (C)</u>

Required area of footing = 52,500 / 6 = 8,750 cms^2

Have section of footing = 95 cms x 95 cms.

Therefore total dimension of footing with 5 cms. of covering on each side = 105 cms x 105 cms.

$M_{x-x} = 6/2(40 + 1.2 \times 32.5)32.5^2 = 250,300$ Kgs. - cms.

$d = (11,025 - 1,600)6 / 4 \times 40 \times 6 = 58.9$ cms. (say 59)

$A_s = 250,300 / 1200 \times 0.88 \times 59 = \underline{\underline{4,00}}$ cms^2

use 4 - 12 m/m Ø per M. ($A_s = 4.52$ cms^2)

T H E E N D