STRUCTURAL DESIGN OF A SCHOOL

USING REINFORCED CONCRETE

DOANY, ATALLAH

1952

A.D

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A SCHOOL

Using

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ATALLAH DOANY.

THE ARCHITECTURAL REQUIREMENTS OF THE SCHOOL

In a suburban small town in the vicinity of Beirut.

School for : Kindergarden
Elementary
and two classes of secondary grade

Number of students:	400	Kindergarden	20
		Elementary	316
		Secondary	64

Classes are considered to hold around 32 students each as an average (some going as high as 35 students)

It is understood that students will eat their noon meal at school, hence the need of a "refectory"

Beside regular classroom, one large classroom is required for education in euthenics, another old purpose room can be used for different activities.

For the servicing, explicite requirements cover a garage for 2 cars, 2 store rooms, a janitor's room, adequate toilet rooms, an office and waiting room for administration, a teacher's room which can be used also for conference between teachers and principal, a combination library - auditorium and a chapel to hold 200 people.

Living quarters to include living space for the local teachers and living space for the principal which includes an adequate living room in which they can entertain visitors.

The latter two living rooms can be included in one building but should be conceived with somewhat of a physical separation.

I N T R O D U C T I O N

In trying to decide upon a subject for my thesis, I was naturally drawn towards design in reinforced concrete. In trying to analyse this natural attraction I have come to these two main reasons:

1. I felt that I had spent in my academic training more time in Architectural design than in structural and thus felt that to arrive to the refinement of the latter I had to spend some time on it.
2. Having noticed that only a few engineers in this part of the world use the modern American method and bind themselves usually to the empirical approach of the French method, I decided myself, to try as far as possible to stick to the former as will clearly appear when studying this Thesis.

Having decided on the general subject of my thesis I began to look for a construction for which to prepare the structural design, and could not find better than my first presentation in Architectural design: A school in the suberbs of Beirut, and that for the following reasons:

1. When I designed the above school architecturally I tried to tackle it from an aesthetic point of view eliminating all architectural flaws as far as possible without considering fully the difficulties that a structural engineer will have to face in designing it. The challenge being there

I accepted the architectural design as it is and tried to solve all these difficulties in this thesis.

2. Another purpose in accepting this particular subject is that it helped me to synchronise both my architectural and structural knowledge to produce better plans without giving preference to either lines.

I wish to bring out at this point some of the problems that I had to face in the structural design of the above mentioned school:

1. Due to the fact that the School has been initially designed to accomodate 400 students only, within a range of classes from first elementary to second year preparatory, a possibility exists for the expansion of this school, that is why it should be noted, that Beams and Slabs of the top floor and the Columns have been designed to carry an additional story except in the case of the Chapel and Auditorium, where the possibility is non-existant. It will be noted that the water reservoir is placed on the top over the staircase in the Administration Building as this is the only natural place for it considering that it had been completely left out by the Architect and it should be understood that incase of expansion the Architect should design in the additional story ~~enough~~ a water reservoir, to satisfy the whole building as the present one has to

therefore No water reservoir during construction?

be removed to allow the use of the present Staircase for its proper function.

2. Another irregularity is that, in teachers bedrooms, it is apparent that the partition walls as designed by the Architect do not rest on the roof beams of the lower offices - taking into consideration that the position of these beams is incumbent upon the partition of the lower offices. Thus it will be noticed that beams were designed as broad as the walls they carry irrespective of their depth. The only weakness is that this forbids the opening up of doors to connect the said bedrooms which actually is unnecessary. Two other possible solutions for this problem would have been:
   a) To make a hidden beam in the slab.
   b) To strengthen the slabs to such an extent, as to carry any partition. Both these solutions are more classical than the one I applied, but the cost was the determining factor in the final choice, considering that mathematically this very narrow beam is enough to carry the load.

3. Another point of interest is in the design of the Beams C - 1 - 2 - 3 in the Administration Building. The Architect in his design has determined the position of Columns 20,21 in such a [redacted] manner as to oblige the structural

engineer to make Columns 20, 21 carry Beams C1 - 2 - 3 by shear. Yet he has helped him successfully complete this when he designed Columns 20, 21 battered, thus allowing the broader top.

4. I wish also to draw the attention of the reader to the beams carrying the inclined slabs of the Chapel's roof. The load on these beams is both horizontal and vertical and the reinforcing bars had to be checked for both, so as to carry the inclined resultant.

In my calculations I have used in the main body of Thesis the approximative methods in finding the moment for the continuous beams. To make this thesis more comprehensive and most beneficial, I have in Appendix A, redesigned the slab of the Chapel using the Hardy-Cross Analysis. And in Appendix C designed two frames chosen from the Auditorium and the Class-rooms devision, also using the Hardy-Cross Principal.

So as to be able to fulfill my aims in this thesis as described previously, i.e. to aquire refinement in structural design, I have in Appendix B redesigned the main Staircase both architecturally and structurally into a hanging staircase, which design has only been recently applied in our country.

<u>Caution:</u> With this change the place of the water tank should be chosen on the other staircase leading to the top floor.

Finally I wish to state that in my designing the structural plans, I have used more than one type of slab, and that is to bring out the different aspects of the purpose of the design as much as possible.

I wish also to point out that the specifications I have used in this design are taken from "Design of Concrete Structures" by O'Rourke, and that the graphs and tables of the same book have been used as much as possible. This should not mean however that those specifications, nor any of the others which are in use, are best suited for this part of the world, due to the fact that they have been arrived at as results of experiments performed under different geographical and climatic conditions.

g f

C O N T E N T S

The Architectural and Structural Drawings are submitted along with this book but in a separate casing.

REFERENCES

1. Design of Concrete Structures by URQUHART and O'ROURKE
Pages, 61, 77, 80, 97, 106, 115, 124, 129, 162, 229,
Chapter VIII, Chapter IX, Appendix B, D.

2. Reinforced Concrete Design by SUTHERLAND and REESE
Chapters V, VI, VII, X, XIII, XV, XVI, XVII.

3. Time - Savor Standards.

4. Concrete plain and reinforced Vol. II. TAYLOR and SMULSKI
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5. Building Construction by HUNTINGTON

6. Builders Materials by B.H. and R.G. KNIGHT.

S P E C I F I C A T I O N S

v for reinforced concrete = 150 p.s.i. = 10.6 $K.G/c.m^2$

v for plain concrete = 60 p.s.i. = 4.24 $K.G/c.m^2$

fs = 20,000 psi = 1410 $K.G/c.m^2$

f'c = 2500 = 70.8 $K.G/c.m^2$, 7 bags for fc = 1000 p.s.i.

f'c = 2000 = 56 make it 60 $K.G/c.m^2$, 6 bags, fc = ? p.s.i. ?

bond U = 7.08 $K.G/c.m^2$

CHAPTER ONE

Design of Chapel Slab and Beams.

Roof beams and T-Beams.

Ground beams are rectangular Beams.

Chapel columns and footings are under the Chapter of Columns.

CHAPEL SLAB

Roof or floor ?

The Chapel slab is divided into sections as shown in the figure

Design of Slab
What method are you using?

DESIGN OF SECTION I. Span 5.4m assume 16 cm slab

$L.L. = 200\ K.G/m^2$

$D.L. = 1m^2 \times 0.15 \times 2500 = 375\ K.G/m^2$ (.16)

$Total = 200 + 375 = 575\ K.G/m^2$

$M = 575 \times 5.4^2/10 = 1670\ K.G. - m.$ ($\frac{wl^2}{10}$ how about)

$= 167000\ K.G. - cm.$

$M = \frac{1}{2}\ fckjbd^2$

$10.9\ bd^2 = 1090\ d^2$

$167000 = 1090\ d^2$

$d^2 = \frac{167000}{1090} = 153$

d = 12.4 cm, + 2.6 = 15 cm slab.

$As = \frac{Ms}{fsxjxd} = \frac{167,000}{1200x0.87x12.4} = 12.9\ cm^2/m.$

1 - 12 m.m ∅ = 1.13

12.9 ÷ 1.13 = 12.3 bars/meter

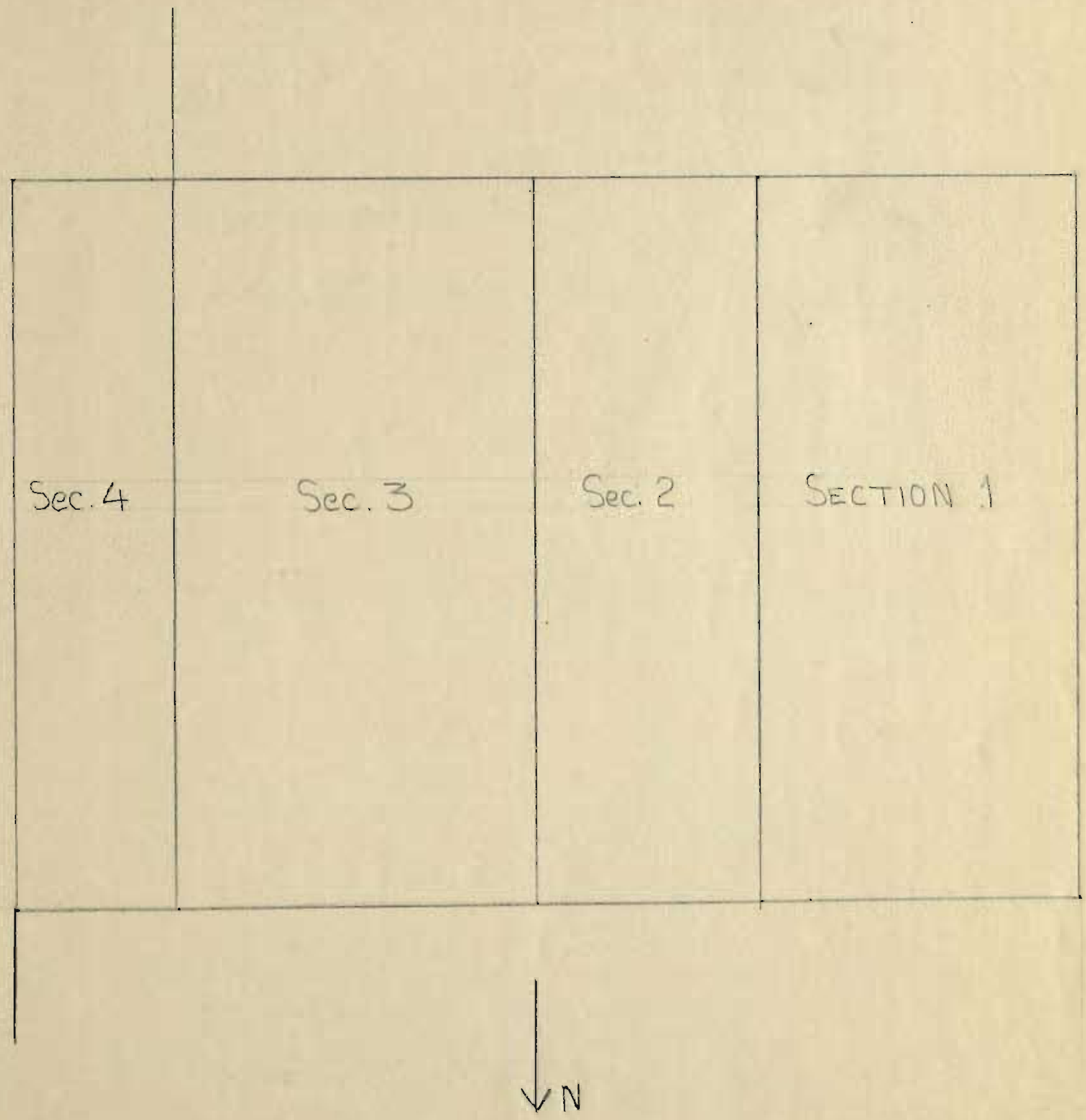
i.e. $\frac{100}{12.3} = 8.1$ cm.

use 12 m.m ∅ at 8 cm.

Top or bottom ?

Use 3 intermediate hidden beams for vibrations.

For longitudinal reinforcement use 6 - 8 m.m ∅/m.



<u>DESIGN OF SECTION 2</u> Span 3.8m assume 16 cm slab.

$$M = \frac{575 \times 3.8^2}{12} = 690 \text{ K.G - m.}$$

$$100 \times \tfrac{1}{2} \times 60 \times 0.87 \times 0.42\ d^2 = 69000$$

$$1090\ d^2 = 69000 \text{ K.G. - cm.}$$

use the same d

$$As = \frac{69000}{1200 \times 0.87 \times 12.4} = 5.3 \text{ cm}^2/\text{m}$$

$$5.3 \div 1.13 = 4.7$$

$$\frac{100}{4.7} = 20.4$$

use 12 m.m ∅ at 20 cm

Same longitudinal reinforcement and intermediate beams.

DESIGN OF SECTION 3 Span 6.1m assume 17 cm slab

D.L. = 1 x 0.17 x 2500 = 425

L.L. = 200

625 K.G/m^2

$$M = \frac{625 \times 6.1^2}{12} = 1930 \text{ K.G - m}$$

$$193000 \text{ K.G - cm} = 1090\ d^2$$

therefore d = 13.4 cm.

$$As = \frac{193000}{1200 \times 0.87 \times 13.5} = 13.7 \text{ cm}^2/\text{m}$$

A of 1 - 12 m.m ⌀ = 1.13 cm^2

13.7 ÷ 1.13 = 12.1 bars

S = 100 ÷ 12.1 = 8 cm.

Same longitudinal reinforcement and intermediate beams.

<u>DESIGN OF SECTION 4</u> Span 2.7m assume 12 cm. slab.

D.L. = 1 x 0.12 x 2500 = 300

L.L. = 200

500 K.G/m^2

$$M = \frac{500 \times 2.7^2}{12} = 305 \text{ K.G. - m}$$

$$d^2 = \frac{30500}{1090} = 28$$

d = 5.3 cm, use d = 10 cm

$$As = \frac{30500}{1200 \times 0.87 \times 10} = 2.93 \text{ cm}^2/\text{m}$$

using 10 m.m ∅, 2.93 ÷ 0.785 = 3.75

100 ÷ 3.75 = 26.6 cm

use 10 m.m ∅ at 20 cm

For longitudinal use 8 m.m ∅ at 20 cm.

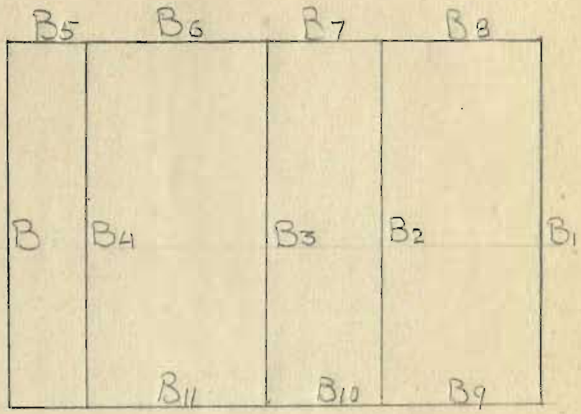
No top bars over the beams?

Refer to working drwgs

ROOF BEAMS. CHAPEL

All are T beams.

B1 is designed to the fullest detail, finding the actual j, and exact As, as an example, while the rest of the beams have been designed on the assumption that j = 0.87



<u>DESIGN OF B1</u> Span 12.2m. $v = \frac{V}{b'jd}$

$v = 10.6\ K.G/cm^2$

$10.6 = \frac{V}{b'd \times 0.87}$

$b'd = \frac{V\ K.Grams}{9.2}$

$w \text{ per linear m} = \frac{5.4}{2} \times 575 = 1550$

$w \text{ of beam (assume } 40 \times 50) \ 04 \times 0.5 \times 2500 = \underline{550}$

2050

$V = 2050 \times (12.2 \div 2) = 12500\ KG$

$b'd = 12500 \div 9.2 = 1350$

$b' = 30 \quad d = 45$

$w \text{ of beam} = 0.30 \times 0.45 \times 2500 = 338$

$\underline{1550}$

$1888\ K.G/m.$

$$M = 1888 \times \frac{12.2^2}{8} = 35300 \text{ K.G - m}$$

$t = 0.15$

$d = 0.56$

$$As = \frac{3530000}{1200(56-7.5)} = 60.5 \text{ cm}^2$$

$$P = \frac{60.5}{56 \times 300} = 0.0036$$

$Pn = 15 \times 0.0036 = 0.05$

$$\frac{t}{d} = \frac{15}{56} = 0.268$$

$j = 0.91 \quad K = 0.27$

$$As_{,} = \frac{3530000}{1200 \times 0.91 \times 56} = 58 \text{ cm}^2$$

$1 - 26 \text{ m.m}\varnothing = 5.31 \text{ cm}^2$

$11 - 26 \text{ m.m } \varnothing = 58.5 \text{ cm}^2$

$$3530000 = fc\left(1 - \frac{15}{2 \times 0.27 \times 56}\right) \times 300 \times 15 \times 0.91 \times 56$$

$$= 115000 \text{ fc.}$$

$fc = 30.8 \text{ K.G/cm}^2$ smaller than 60 K.G/cm^2

Check for shear

$$v = \frac{V}{b'jd} = \frac{12500}{30 \times 0.87 \times 56} = 8.6 \text{ K.G/cm}^2 \quad \text{O.K.}$$

Perimeter of $1 \varnothing = \pi \times 2.6 = 8.15$ cm

$$U = \frac{12500}{6 \times 8.15 \times 0.87 \times 56} = 5.1 \text{ smaller than } 7.08 \quad \text{O.K.}$$

? Bend for what?

Where should we bend 5 bars

1st bar $\frac{1}{11}$ x 100 = 9.1%, 0.34x12.2 = 4.15 — 4.00

1st pair $\frac{3}{11}$ x 100 = 27.3%,0.24x12.2 = 2.93 — 2.75

2nd pair $\frac{5}{11}$ x 100 = 45.5%,0.16x12.2 = 1.95 — 1.75.

If no stirrups are used $v \neq 150$ lb/in^2

it is 60 i.e. 4.24 K.G/cm^2

Shear carried by concrete only:

$V = v \times b \times j \times d$

$V_c = 4.24 \times 30 \times 0.87 \times 56 = 6200$ Kg.

Total V = 12500 K.G

12500 - 6200 = 6300 K.G. carried by stirrups

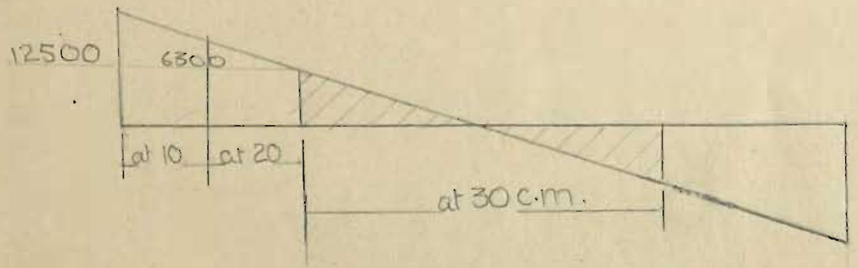
Using 8 m.m Ø stirrups

$S_a = \frac{2 \times 0.5 \times 1410 \times 0.87 \times 56}{6300} = 10.9$ cms

place where V = 6200

12500 - 1888x = 6200

$x = \frac{6300}{1888} = 3.35$m.



$S_b = \frac{2 \times 0.5 \times 1410 \times 0.87 \times 56}{3150} = 21.8$ cm.

DESIGN OF BEAM B2   Span 12.2m.

$$w = \frac{5.4 + 3.8}{2} \times 575 = 2640$$

w of beam: $0.35 \times 0.60 \times 2500 = \underline{522}$

3162

$522 + 2640 = 3162$

$V = 3162 \times 6.1 = 19300$

$$b'd = \frac{19300}{9.2} = 2100 \text{ cm}^2$$

$b' = 35 \quad d = 60$ cm   O.K.

$$M = \frac{3162 \times 12.2^2}{8} = 58{,}000 \text{ K.G - m}$$

$$As = \frac{5{,}800000}{1200 \times 0.87 \times 71} = 78 \text{ cm}$$

1 - 24 m.m Ø = 4.52 cm^2

18 - 24 m.m Ø = 81.43   O.K.

Perimeter of 1 bar = 3.14 x 24 = 75 m.m

If we bend 8 bars

$$U = \frac{19300}{10 \times 7.5 \times 0.87 \times 71} = 4.17 \text{ K.G/cm} \quad \text{O.K.}$$

Where to bend

1st pair $\frac{2}{18} \times 100 = 11\%$, 0.34 x 12.2 = 4.15 m - 4.00m

2nd pair $\frac{4}{18} \times 100 = 22\%$, 0.26 x 12.2 = 3.17 - 3.00m

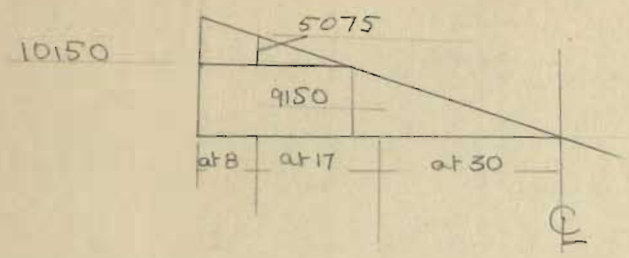
3rd pair $\frac{6}{18} \times 100 = 33\%$, 0.21 x 12.2 = 2.56 - 2.50m

4th pair $\frac{8}{18} \times 100 = 44\%$, 0.16 x 12.2 = 1.95 - 1.75m

Stirrups: v = 60 p.s.i. = 4.24 K.G./ cm^2

Shear carried by concrete alone:

Vc = vbjd = 4.24 x 0.87 x 71 x 35 = 9150 K.Gs.



S at end using 8 m.mØ stirrups

$$s = \frac{2 \times 0.5 \times 1410 \times 0.87 \times 71}{10150} = 8.5 \text{ cm.}$$

s at 3.2m

$$S = \frac{2 \times 0.5 \times 1410 \times 0.87 \times 71}{5075} = 17 \text{ cm.}$$

at 30 cm spacing in the middle.

<u>DESIGN OF BEAMS B3 AND B4</u> taking B3.

$$w = \frac{10}{2} \times 650 = 3250$$

w of beam = 0.40 x 0.65 x 2500 = 650

3250 + 650 = 3900 K.G/m.

$$V = \frac{12.2}{2} \times 3900 = 23700$$

$$b'd = \frac{23,700}{9.2} = 2620$$

b = 40 , d = 65.5 cm.

use 40 by 66

$$M = \frac{3900 \times 12.2^2 \times 100}{8} = 7200000 \text{ K.G. cm.}$$

$$As = \frac{7200000}{1200 \times 0.87 \times 78} = 87 \text{ cm}^2$$

$$20 - 24 \text{ m.m } \phi = 89.72 \text{ cm}^2$$

Perimeter of 1 bar = 7.5 cm.

We will bend 10 bars

$$U = \frac{23700}{9 \times 7.5 \times 0.87 \times 80} = 5.05 \quad \text{O.K.}$$

Where to bend bars:

1st pair $\frac{2}{20}$ = 10% at 12.2 x 0.36 = 4.40 make it 4.40m.

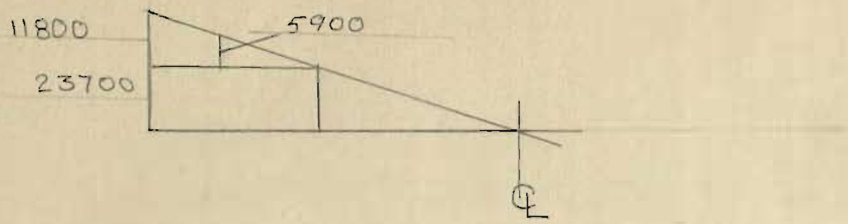
2nd pair $\frac{4}{20}$ = 20% at 12.2 x 0.30 = 3.65 " " 3.60m.

3rd pair $\frac{6}{20}$ = 30% at 12.2 x 0.25 = 3.05 " " 2.80m.

4th pair $\frac{8}{20}$ = 40% at 12.2 x 0.21 = 2.56 " " 2.00m.

5th pair $\frac{10}{20}$ = 50% at 12.2 x 0.17 = 2.07 " " 1.20m.

$$Vc = 4.24 \times 0.87 \times 78 \times 40 = 11900 \text{ Kg.}$$



$$s = \frac{2 \times 0.5 \times 1410 \times 0.87 \times 80}{11800} = 8.85 \text{ cm.}$$
say 8.5 cm.

$$s = \frac{2 \times 0.5 \times 1410 \times 0.87 \times 80}{5900} = 17.70 \text{ cm.}$$
= 17.5 cm.

Check for slope effect.

$4.75 \times 750 \times 0.51 \div 4.75 = 382$ Kg/m.

$$M = \frac{382 \times 12.2^2}{10} = 570000 \text{ K.g.cm.}$$

$570000 = 9.2 \times 82 \times d^2$, $d^2 = 760$, $d = 27.6$ cm.

We have d = 37 cm ∴ O.K.

$$As = \frac{570000}{1200 \times 0.87 \times 37} = 14.7 \text{ cm}^2$$

3 - 24 m.m ∅ = 13.57

1 - 18 m.m ∅ = 2.54

16.11 cm^2 greater than 14.7 cm^2

This result is from one row only the other rows help in carrying this moment, therefore the inclination of the roof is safe and can be taken care of by the beams ~~only~~. easily.

? Where is it from where to where? Refer to drawings

DESIGN OF BEAM B-11

$$W = \frac{4.32^2}{2} = 9.3 \text{ m}^2$$

$$\frac{9.3 \times 575}{6.1} = 880 \text{ K.G/m.}$$

Assume 0.30 x 0.30 x 2500 = 225 K.G/m

Total W = 225 + 880 = 1105 K.G/m.

$$V = 1105 \times \frac{6.1}{2} = 3370$$

b' = 30   d = 12.2

use 30 x 30

$$M = \frac{1105 \times 6.1^2}{12} = 3400 \text{ K.G - m.}$$

$$As = \frac{340000}{1200 \times 0.87 \times 45} = 7.3 \text{ cm}^2$$

use - 5 - 14 m.m Ø at 7.70 cm^2

Bend 2 bars at 0.18 x 6.1 = 1m.

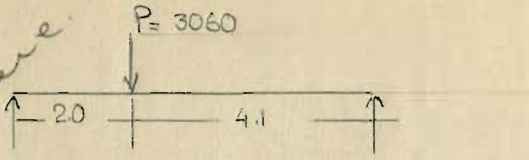
$$U = \frac{3370}{3 \times (3.14 \times 1.4) \times 0.87 \times 45} = 6.5$$ smaller than 7.08 O.K.

Vc = 4.24 x 0.87 x 30 x 45 = 4960 greater than 3370

Using 8 m.m Ø stirrups, start with s = 10 cm for 1-m. 15 cm for 2nd meter and 30 cm in the middle.

Use the same design for B-7, B-8, B-9, B-10.

DESIGN OF BEAM B6   Span 6.1m



Where does P = 3060 come from?
Where is it?
Refer to drawings you'll know where

find P:

$5.5 \times 0.7071 = 3.9$

$\frac{3.9^2}{2} \times 650 = 4880$

$\frac{4880}{2} = 2440$ K.g.

Assume slab beam = 30 x 30

$W = 0.30 \times 0.30 \times 2500 = 225$

$225 \times \frac{5.5}{2} = 620$

$P = 225 + 620 + 2440 = 3060$

$R_A = \left(3060 \times \frac{4.1}{6.1} + \frac{1105 \times 6.1}{2}\right)$

$= 2060 + 3370 = 5430$

Max $V = 5430$ Kg.

$b'd = \frac{5430}{9.2} = 592$

$b' = 30 \quad d = 19.8 \quad$ use 35

$V = 0$ at: $5430 - 1105x - 3060 = 0$

$x = \frac{2370}{1105} = 2.15$m

$M = 5430 \times 2.15 = 11700$

$3060 \times 0.15 = 406$

$\frac{1105 \times 2.15^2}{2} = 2550$

2956

$\therefore$ M = 11700 - 2956 = 8744

$$As = \frac{874400}{1200 \times 0.87 \times 45} = 18.6 \text{ cm}^2$$

use 8 - 18 m.m Ø at 20.36 cm^2

bend 4ϕ

$$\frac{5430}{4 \times 5.65 \times 0.87 \times 45} = 6.18 \text{ smaller than } 7.08 \quad O.K.$$

$$Vc = 4.24 \times 0.87 \times 30 \times 45 = 4960$$

Using 6 m.m Ø stirrups

$$S = \frac{2 \times 0.28 \times 1410 \times 0.87 \times 45}{(5430 - 4960)} = 6.6 \text{ cm}$$

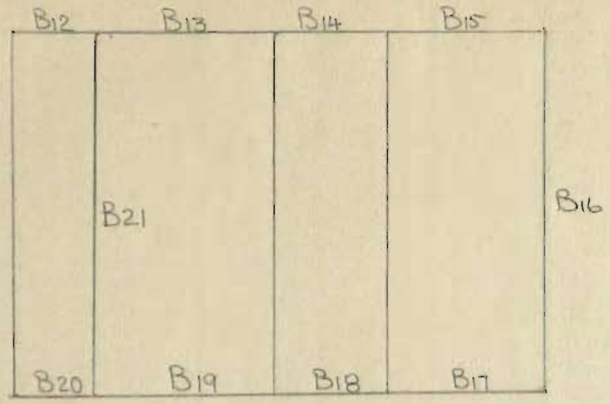
all over the beam.

Bending

1st row $\frac{2}{8}$ = 25%, 6.1 x 0.25 = 1.75m.

2nd row $\frac{4}{8}$ = 50%, 6.1 x 0.14 = 0.85m.

GROUND BEAMS CHAPEL



DESIGN OF B-13 AND B 19

Wall:1 x 0.30 x 5.8 x 2000 = 3470

1 x 0.40 x 0.60 x 2500 = 600

4070 K.g/m.

$M = \frac{4070 \times 6.1^2}{12} = 12600$ K.g.m

$M = 10.9\ bd^2$

$V = 4070 \times \frac{6.1}{2} = 12500$ K.g.

$d^2 = \frac{1260000}{10.9 \times 40} = 2890$

d = 54 cm, + 4 = 58

Use 40 x 60 cm.

$As = \frac{1260000}{1200 \times 0.87 \times 56} = 21.5\ cm^2$

Use 11 - 16 m.m Ø, ($A = 22.11\ cm^2$)

Bend 5 Øs

perimeter of one bar = $\pi \times 16 = 5.00$ cm

$U = \frac{12500}{6 \times 5 \times 0.87 \times 56} = 6.5$ smaller than 8.08 O.K.

Add 3 - 16 m.m. Ø on top of columns to assist in carrying the negative moment.

Bending of bars

1st bar $\frac{1}{11}$ x 100 = 9%, at 0.38 x 6.1 = 2.30m.

1st pair $\frac{3}{11}$ x 100 = 27%, at 0.28 x 6.1 = 1.70m.

2nd pair $\frac{5}{11}$ x 100 = 45%, at 0.17 x 6.1 = 1.10m.

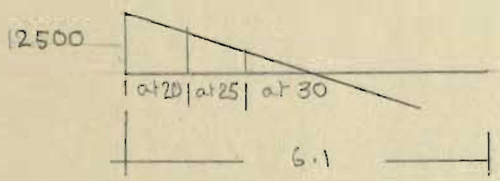
Stirrups

Vc = 4.24 x 40 x 0.87 x 56 = 8300 K.g

12500 - 8300 = 4200 K.g.

Using 6 m.m Ø stirrups, A = 0.28

$$S = \frac{2 \times 0.28 \times 1410 \times 0.87 \times 56}{4200} = 9.1 \text{ cm.}$$



Use s = 9 cm for 1 m, then 20 cm then 30 cm.

DESIGN OF BEAM B14 AND B18   Span 3.8m.

Wall: $1 \times 0.30 \times 7 \times 2000 = 4190$

$1 \times 0.40 \times 0.6 \times 2500 = 600$

4790 K.g./m

$$M = \frac{4790 \times 3.8^2}{12} = 5750 \text{ K.g.m}$$

$$V = 4790 \times \frac{3.8}{2} = 9100 \text{ K.g.}$$

$$As = \frac{575000}{1200 \times 0.87 \times 56} = 9.8 \text{ cm}^2$$

Use 8 - 14 m.m Ø = 12.32 cm^2

bend 2 Ø at:

$\frac{2}{8} \times 100 = 25\%$ at $0.3 \times 3.8 = 1.14$m.

perimeter of one bar = 4.4 cm.

$$U = \frac{9100}{6 \times 4.4 \times 0.87 \times 56} = 7.08. \quad \text{O.K.}$$

Add 3 - 14 m.m Ø on top of beam.

$Vc = 4.24 \times 40 \times 0.87 \times 56 = 8300$

$9100 - 8300 = 800$ K.g.

Using 6 m.m Ø stirrups

$$S = \frac{2 \times 0.28 \times 1410 \times 0.87 \times 56}{800} = 48 \text{ cm.}$$

Start with 20 and end with 40.

DESIGN OF BEAMS B15 - B17   Span 5.4m.

Wall:1 x 0.30 x 7.8 x 2000 = 4700

w of beam 0.40 x 0.70 x 2500 = 700

700 + 4700 = 5400 K.g. per m.

$$M = \frac{5400 \times 5.4^2}{10} = 15,700 \text{ K.g.-m}$$

$M = 10.9\ bd^2$

$1570000 = 10.9 \times 0.40\ d^2$

$d^2 = 3620$, d = 60, use 64

$$V = \frac{5400 \times 5.4}{2} = 14,600 \text{ K.g.}$$

$$As = \frac{1570000}{1200 \times 0.87 \times 60} = 25 \text{ cm}^2$$

13 - 16 m.m Ø = 26.13 cm^2

bend 5 Øs

$$U = \frac{14600}{8 \times 5 \times 0.87 \times 60} = 7 \text{ smaller than } 7.08, \quad O.K.$$

1st bar $\frac{1}{13}$ = 7.7% at 0.41 x 5.4 = 2.2m.

1st pair $\frac{3}{13}$ = 23.1% at 0.30 x 5.4 = 1.62m.

2nd pair $\frac{5}{13}$ = 38.5% at 0.22 x 5.4 = 1.18m.

Add 3 Ø on top.

Vc = 4.24 x 40 x 0.87 x 60 = 8600 K.g.

14600 - 8600 = 6000 K.g.

Using 8 m.m Ø stirrups

$$S = \frac{2 \times 0.50 \times 1410 \times 0.87 \times 60}{6000} = 12 \text{ cm}$$

then at 20 cm then at 30 cm.

DESIGN OF BEAM B16 Span 12.2m.

Wall: 1 x 0.30 x 8 x 2000 = 4800

w = 0.50 x 1.50 x 2500 = 1870

6670 K.g per m.

$M = \frac{6670 \times 12.2^2}{9} = 111,000$ K.g - m.

$= 111,000,00$ K.g - cm.

$11100000 = 10.9 \times 50 \times d^2$

$d^2 = 20200$

d = 142, i.e. 150 cm.

$As = \frac{11,100000}{1200 \times 0.87 \times 142} = 75\ cm^2$

Use 15 - 26 m.m Ø ($A = 80\ cm^2$)

perimeter of one bar = 8.3 cm

$V = 6670 \times \frac{12.2}{2} = 40500$

Bend 7 bars

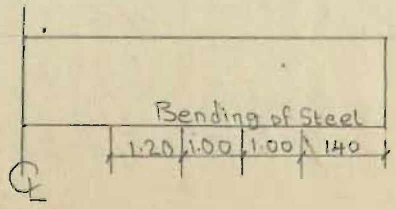
Bending of bars

1st bar $\frac{1}{15} \times 100 = 6.67\%$ at $12.2 \times 0.38 = 4.65$ 4.60

1st pair $\frac{5}{15} \times 100 = 20\%$ at $12.2 \times 0.28 = 3.42$ make it 3.40

2nd pair $\frac{5}{15} \times 100 = 33.35\%$ at $12.2 \times 0.21 = 2.55$ make it 2.40

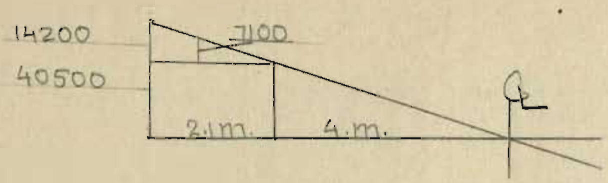
3rd pair $\frac{7}{15} \times 100 = 46.5\%$ at $12.2 \times 0.16 = 1.95$ make it 1.40



Using 8 m.m Ø stirrups

$Av = 0.785\ cm^2$

Vc = 4.24 x 0.87 x 50 x 142 = 26300 K.gs



40500 - 26306 = 14200

$$\frac{26300}{40500} \times 6.1 = 3.95 \text{ m.}$$

$$s = \frac{2 \times 0.5 \times 1410 \times 0.87 \times 142}{14200} = 12.2 \text{ cm.}$$

$$s = \frac{\text{same}}{7100} = 24.4 \text{ cm.}$$

Use 10 cm in 1st meter

20 cm in 2nd meter

30 cm in rest.

DESIGN OF BEAM B21   Span 12.2m

Wall = 1 x 0.12 x 5.8 x 2000 = 1390

w = 0.40 x 1 x 2500 = 1000

2390 K.g per m.

$$M = \frac{2390 \times 12.2^2}{9} = 40,000 \text{ K.g - m}$$

4000,000 K.g - c.m.

$$= 10.9 \times 40d^2$$

$$\therefore d^2 = 9200 , d = 96 , + 6 = 1.02 \text{ m.}$$

$$As = \frac{4000000}{1200 \times 0.87 \times 94} = 41 \text{ cm}^2$$

Use 10 - 24 m.m ∅ , (A = 45.24 cm^2)

perimeter of one bar = 3.14 x 2.4 = 7.5 cm.

$$V = 2390 \times \frac{12.2}{2} = 14,600 \text{ K.g}$$

$$U = \frac{14600}{5 \times 7.5 \times 0.87 \times 94} = 4.8 \text{ smaller than } 7.08$$

Bending of bars

1st bar $\frac{1}{10}$ x 100 = 10% at 12.2 x .34 = 4.15 make it 4.00m.

1st pair $\frac{3}{10}$ x 100 = 30% at 12.2 x .24 = 2.93 make it 2.80m.

2nd pair $\frac{5}{10}$ x 100 = 50% at 12.2 x .14 = 1.17 make it 1.10m.

Vc = 4.24 x 40 x 0.87 x 94 = 13800

Using 8 m.m ∅ stirrups

$$S = \frac{2 \times 0.5 \times 1410 \times 0.87 \times 94}{800} = 14.5 \text{ cm.}$$

Start with 14 cm spacing till 1/3 then use 30 cm.

CHAPTER TWO

Auditorium

Roof Beams , Slabs

G. Beams

Columns and Footing in the chapter on

Columns

AUDITORIUM SLAB

DESIGN OF SECTION 1

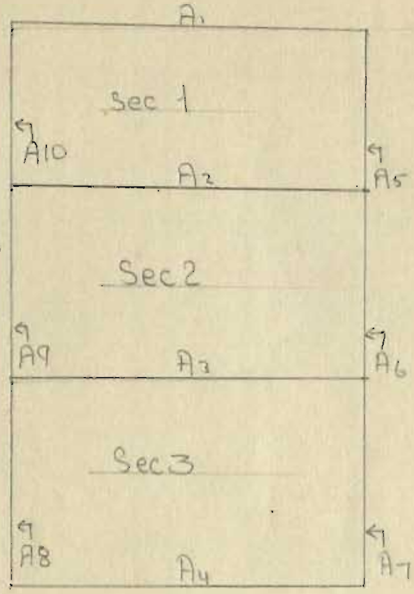
Span 5.4

It is as Chapel section one.

15 cm slab

use 12 m.m ∅ at 8 cm

use 3 intermediate hidden beams to take care of vibrations



For longitudinal reinforcement use 6 - 8 m.m ∅ per m.

DESIGN OF SECTION 2   Span 6.5m   assume 17 c.m slab.

D.L. = 425

L.L. = 200

Total L. = 625 K.g per m^2

$$M = \frac{625 \times 6.5^2}{12} = 2180 \text{ K.G.-m}$$

$$d^2 = 218000 \div 1090 = 200$$

d = 14.2 c.m.   say 14.5 , + 2.5 = 17 cm.

$$As = \frac{218000}{1200 \times 0.87 \times 14.5} = 14.5 \text{ cm}^2 \text{ per m.}$$

A of 1 - 12 m.m ∅ = 1.13 $c.m^2$

14.5 ÷ 1.13 = 12.8 bars per m.

S = 100 ÷ 12.8 = 7.8 cm say 7.5 cm.

Same longitudinal reinforcement and hidden beams as section one.

<u>DESIGN OF SECTION 3</u>   Span 7 m.   assume 19 c.m slab.

D.L. = 0.19 x 1 x 2500 = 475

L.L. = 200

Total = 675 K.G per m^2

$$M = \frac{675 \times 7^2}{12} = 3300 \text{ K.G.-m}$$

$d^2 = 330000 \div 1090 = 305,$

d = 17.3, + 1.7 = 19 c.m.

$$As = \frac{330000}{1200 \times 0.87 \times 17.3} = 18.3 \text{ cm}^2 \text{ per m}$$

A of 1 - 14 m.m Ø = 1.54

18.3 ÷ 1.54 = 11.9 bars per m.

S = 100 ÷ 11.9 = 8 c.m.

Same longitudinal reinforcement as before.

DESIGN OF BEAMS A1, A2, A3, A4

take A3, Span 12.m.

w = 675 x 6.75 = 4550 K.g per m.

assume

0.45 x 0.8x2500= 900 " " " "

Total 5450 " " " "

$$V = \frac{12}{2} \times 5450 = 32700$$

b'd = 32700 ÷ 9.2 = 3550

b' = 45 d = 79.2 say 80

$$M = \frac{5450 \times 12^2}{8} = 98000$$

$$As = \frac{9800000}{1200 \times 0.87 \times 94} = 100 \text{ cm}^2$$

Use 23 - 24 m.m ∅ = 104 $c.m^2$.

perimeter of one bar = 3.14 x 24 = 7.5 cm.

bend 12 bars

$$U = \frac{32,700}{9 \times 7.5 \times 0.87 \times 94} = 6 \quad \text{O.K.}$$

Bending

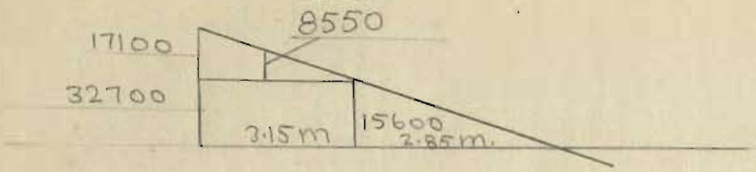
1st 3 bars $\frac{3}{23} \times 100 = 13\%$ at 12 x 0.31 = 3.72 make it 3.5m.

2nd " " $\frac{6}{23} \times 100 = 26\%$ at 12 x 0.24 = 2.87 make it 2.57m.

3rd " " $\frac{9}{23} \times 100 = 39\%$ at 12 x 0.18 = 2.16 make it 2.00m.

4th " " $\frac{12}{23} \times 100 = 52\%$ at 12 x0.13 = 1.56 make it 1.25m.

Vc = 4.24 x 45 x 0.87 x 94 = 15600 Kg.



$\frac{15600}{32700} = \frac{x}{6}$, x = 2.85m

at end using 10 m.m Ø stirrups

$s = \frac{2 \times 0.785 \times 1410 \times 0.87 \times 94}{17100}$ = 10.6 say 10 cm.

$S = \frac{2 \times 0.785 \times 1410 \times 0.87 \times 94}{8550}$ = 21 cm. say 20 c.m.

DESIGN OF BEAMS A6 - A7 - A8 - A9

taking A8 Span 7 m.

7 x 0.707 = 4.949 m.

$$w = \frac{4.95^2}{2} \times \frac{625}{7} = 1590 \text{ K.g per m.}$$

assume

0.30 x 0.30 x 2500= 225 " " " "

Total 1815 " " " "

$$V = 1815 \times \frac{7}{2} = 6350 \text{ K.g.}$$

b'd = 6350 ÷ 9.2 = 690

b' = 30 d = 23 cm. use 30 below slab

$$M = \frac{1815 \times 7^2}{10} = 8900 \text{ K.g}$$

$$As = \frac{890000}{1200 \times 0.87 \times 45} = 19 \text{ c.m}^2$$

$$U = \frac{6350}{5 \times 5 \times 0.87 \times 45} = 6.5 \quad \text{O.K.}$$

Bending

1st bar $\frac{1}{10}$ x 100 = 10% at 7 x 0.38 = 2.85 make it 2.75m.

1st pair $\frac{3}{10}$ x 100 = 30% at 7 x 0.25 = 1.75 make it 1.50m.

2nd pair $\frac{5}{10}$ x 100 = 50% at 7 x 0.18 = 1.26 make it 1.10m.

Vc = 4.24 x 0.87 x 30 x 45 = 4960

Using 6 m.m Ø stirrups

$$S = \frac{2 \times 0.28 \times 1410 \times 0.87 \times 45}{6350 - 4960} = 22 \text{ cm.}$$

Start with 20 c.m. spacing and at the middle 25 c.m.

<u>DESIGN OF BEAMS A4 - A5</u> Use the same section of concrete

$As = 19 \times (\frac{5.5}{7})^2 = 11.8$ c.m^2

Use 6 - 16 m.m Ø (A = 12.06 cm^2)

Bend 3 bars

1st bar 16% , = 0.32 x 5.5 = 1.76 m.

1st pair 50% , = 0.18 x 5.5 = 1.00 m.

GROUND BEAMS AUDITORIUM

DESIGN OF BEAMS A11 - A15

Span 12.m.

Wall = 1 x 0.30 x 6 x 2000 = 3600

w = 1 x 0.40x0.40x2500 = 1400

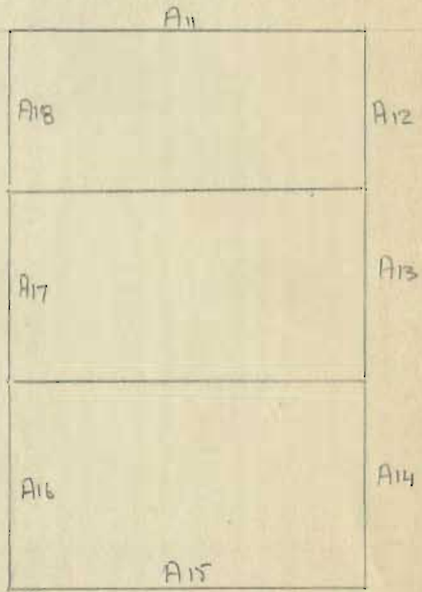
Total 5000 Kg per m

$M = \frac{5000 \times 12^2}{9} = 80000$ K.g - m

$= 8000000$ K.g.c.m.

$= 10.9\ bd^2$

$d^2 = 18500$, d = 136, + 6 = 142 c.m.



$As = \frac{8000000}{1200 \times 0.87 \times 136} = 56\ c.m^2$

use 11 - 26 m.m ∅ , ($A = 58.7\ cm^2$)

V = 5000 x 6 = 30000 K.g.

Bend 5 bars

Perimeter of one bar = 3.14 x 2.6 = 7.5 c.m.

$U = \frac{30000}{6 \times 7.5 \times 0.87 \times 136} = 5.65$ O.K.

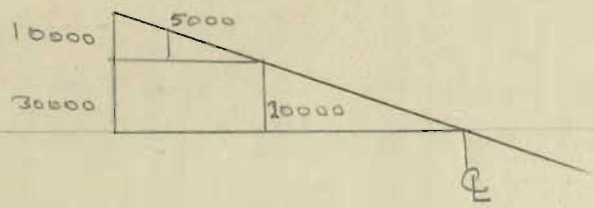
Bending:

1st bar $\frac{1}{11}$ x 100 = 9%, at 12 x 0.34 = 4.10 make it 4.00m.

1st pair $\frac{3}{11}$ x 100 = 27%, at 12 x 0.24 = 2.88 make it 2.70m.

2nd pair $\frac{5}{11}$ x 100 = 45%, at 12 x 0.16 = 1.92 make it 1.40m.

Vc = 4.24 x 40 x 0.87 x 136 = 20000 K.g.



Using 6 m.m Ø stirrups

$$S = \frac{2 \times 0.28 \times 1410 \times 0.87 \times 136}{10000} = 9.4$$

Spacings

9 c.m. for the 1st meter

18 c.m. for the 2nd meter

30 c.m. for the rest.

DESIGN OF A12 - and A18 The same as:

B15 - and B17 in Chapel.

DESIGN OF BEAMS A13 - A14 - A15 -A17

Span 6.9 m.

Wall = 1 x 0.3 x 6 x 2000 = 3600

w = 0.40 x 0.76 x 2500 = 760

4360 K.g per m.

$M = \frac{4360 \times 6.9^2}{10} = 20700$ K.g.m.

$= 10.9 \times 40\ d^2$

$\therefore d^2 = 4760$, d = 69 c.m.

$As = \frac{2070000}{1200 \times 0.87 \times 69} = 29\ cm^2$

Use 10 - 20 m.m ∅ = 31.42 cm^2

$V = 4360 \times \frac{6.9}{2} = 15000$, perm. of one bar = 6.28 cm.

Bond is safe

Bending:

1st pair $\frac{2}{10} \times 100 = 20\%$ at 6.9 x 0.3 = 2.07 m.

2nd pair $\frac{4}{10} \times 100 = 40\%$ at 6.9 x 0.21 = 1.45 m.

Vc = 4.24 x 40 x 0.87 x 69 = 10200 K.g.

Using 6 m.m ∅ stirrups

$S = \frac{2 \times 0.28 \times 1410 \times 0.87 \times 69}{(15000 - 10200)} = 9.85$ c.m. say 9 c.m.

Start with 9 c.m. spacing for 1st meter

20 c.m. spacing for 2nd meter

30 c.m. spacing for rest.

C H A P T ERR T H R E E

Administration Division

1. Slabs: Types: ordinary simple slabs.

A - Top roof slabs:-

1 - roof of bedrooms:- 1, 2, 4, 5, 6, 7.

Assume 12 c.m. slab.

$$\left(\frac{4.2}{3.5} - 0.5\right) \times 650 = 455$$

Short direction:-

$$M = \frac{455 \times 3.5^2}{12} = 465.5 \text{ K.G.-m}$$

$$= 46550 \text{ K.G-c.m.}$$

$$As = \frac{46550}{1200 \times 0.87 \times 9} = 5 \text{ cm}^2 \text{ per m.}$$

Use 10 m.m Ø

$$S = 100 \times \frac{0.785}{5} = 15.7 \quad \text{say } 15 \text{ c.m.}$$

Long direction:-

$$M = \frac{(650 - 455) \times 4.2^2}{12} = 286.00 \text{ K.G - m.}$$

$$As = \frac{28600}{1200 \times 0.87 \times 8} = 3.44$$

Use 8 m.m Ø

$$S = 100 \times \frac{0.5}{3.44} = 14.5 \text{ cm.}$$

2. Roof of bedroom No 3

$$\left(\frac{4.8}{4.2} - 0.5\right) X\ 650 = 425 \text{ K.G per } m^2$$

$$650 - 425 = 225 \text{ K.G. per } m^2$$

Short direction:-

$$\frac{425 \times 4.2^2}{12} = 620 \text{ K.G - m.}$$
$$= 62000 \text{ K.G. - cm.}$$

$$As = \frac{62000}{2100 \times 0.87 \times 9} = 6.6 \text{ c.m}^2$$

Use 10 m.m Ø

$$S = 100 \times \frac{0.785}{6.6} = 11.9 \text{ c.m.} \quad \text{say } 11 \text{ c.m.}$$

Long direction:-

$$M = \frac{225 \times 4.8^2}{12} = 432 \text{ K.G. - m.}$$
$$= 43200 \text{ K.G. - c.m.}$$

$$As = \frac{432000}{1200 \times 0.87 \times 8} = 5.15 \text{ c.m}^2$$

Use 8 m.m Ø = 0.5 cm^2

$$S = 100 \times \frac{0.5}{5.15} = 9.75 \quad \text{say } 9 \text{ c.m.}$$

3. <u>Roof of Toilets amd baths in second floor which is No 8:-</u>

Dimensions 4 x 4.2

<u>Short direction:-</u>

$$M = \frac{325 \times 16}{12} = 432$$

$$As = \frac{43200}{1200 \times 0.87 \times 9} = 4.62 \ c.m^2$$

1 - 10 m.m ⌀ = 0.79

$$S = 100 \times \frac{0.79}{4.62} = 17.1 \quad \text{say } 17 \ c.m.$$

<u>Long direction:-</u>

$$S = 17 \times \frac{8}{9} = 15 \ c.m.$$

4. <u>Roof of corridor and private toilet in second floor:-</u>

$l = 2m$

$$M = \frac{650 \times 4}{12} = 216.7 \text{ K.G - m.}$$

$$As = \frac{21670}{1200 \times 0.87 \times 9} = 2.32 \text{ square c.m/m}$$

Use 5 - 8 m.m ∅/m = 2.5 sq.c.m./m

and 5 - 8 m.m ∅/m in long direction.

4. The top roof of stair case being the floor of the reservoire, its design is under the design of the reservoire.

5. <u>DESIGN OF ROOFS 9 AND 10:-</u> In one direction.

Assume 20 c.m. slab.

D.L. = 0.20 x 2500 = 500

L.L. = = 450

Tile and sand = 150

1100

$$M = \frac{1100 \times 36}{12} = 3300 = 10.9 \times 100\ d^2$$

$$\therefore d^2 = 302\ ,\quad d = 17.5\ ,\ \text{O.K.}$$

$$As = \frac{330000}{1200 \times 0.87 \times 17.5} = 18.1 \text{ sq.c.m/m}$$

1 - 16 m.m ∅ = 2.01

$$S = 100 \times \frac{2.01}{18.1} = 11 \text{ c.m.}$$

6. DESIGN OF ROOF 11 TOP FLOOR:-

Dimensions 5.5 x 6.5

Short direction:-

$$675 \left(\frac{6.5}{5.5} - 0.5\right) = 464$$

$$M = \frac{464 \times 5.5^2}{12} = 116000 \text{ K.G - cm}$$

$$= 10.9 \times 100\, d^2$$

$d^2 = 107$, $d = 10.3$ c.m. say $10.5 + 2.5 = 13$

$$As = \frac{116000}{1200 \times 0.87 \times 10.5} = 10.6$$

1 - 12 m.m ∅ = 1.13

$$S = 100 \times \frac{1.13}{10.6} = 10 \text{ c.m.}$$

Long direction:

$$M = \frac{211 \times 6.5^2}{12} = 74200$$

$$As = \frac{74200}{1200 \times 0.87 \times 9.5} = 7.5 \text{ c.m.}^2$$

1 - 10 m.m ∅ = 0.785 sq. c.m.

$$S = 100 \times \frac{0.785}{7.5} = 10 \text{ c.m.}$$

B - First floor roof slabs is the same as top roof slabs.

C - DESIGN OF THE FLOOR SLABS OF THE FIRST FLOOR BEYOND THE RETAINING WALL:-

1. Floor of Teachers' Toilet and private toilet the same as Toilets in second floor.
2. Floor of corridor and part of principals office beyond retaining wall the same as corridors floor in second story.
3. Floor of principals living room the same as floor of Toilets and bath in second floor.
4. Floor of kitchen and stairs first floor the same as floor of bedrooms 1, 2, 3, second floor.

FLOOR OF TEACHERS' DINING in one direction: Span 3.5

$$M = \frac{65.0 \times 3.5^2}{10} = 792 \text{ K.G -m.}$$

$$= 79200 \text{ K.G.-c.m.}$$

$$As = \frac{79200}{1200 \times 0.87 \times 9} = 8.45 \text{ c.m}^2$$

A of 1 - 12 m.m Ø = 1.13 cm^2

$$S = 100 \times \frac{1.13}{8.45} = 13.4 \text{ c.m.}$$ make it 12 c.m.

and use 5 - 8 m.m Ø/m. in the long direction.

TEACHERS' CONFERENCE:-

5.9 x 4.2

$$w\left(\frac{1}{1'} - \frac{1}{2}\right) = \left(\frac{5.9}{4.2} - \frac{1}{2}\right) \times 650 = 590$$

w carried by long side = 650 - 590 = 60 $K.G/m^2$

Short direction:-

$$M = \frac{590 \times 4.2^2}{10} = 1040 \text{ K.G - m.}$$

$$= 104000 \text{ K.G. - c.m.}$$

$$As = \frac{104000}{1200 \times 0.87 \times 9} = 11.2 \text{ cm}^2$$

A of 1 - 12 m.m ∅ = 1.13

$$100 \times \frac{1.13}{11.2} = 10.5 \text{ make it 10 c.m.}$$

Long direction:-

$$M = \frac{60 \times 5.9^2}{10} = 208 \text{ K.G. - m.}$$

$$= 20800 \text{ K.G - c.m.}$$

$$As = \frac{208000}{1200 \times 0.87 \times 9} = 2.22 \text{ c.m.}^2$$

1 - 8 m.m ∅ = 0.5

$$100 \times \frac{0.5}{2.22} = 22.5 \text{ c.m. make it 20 c.m.}$$

DESIGN OF ROOF BEAMS

ROOF OF TOP FLOOR:- Exterior Beams:-

Beams C1 and C3 Span 5.9

w from wall = 0.30 x 1 x 3.20 x 2500 = 2420 K.G/m.

slab $= \left(\frac{4.2}{2}\right)^2 \times \frac{650}{5.9} = 965$ K.G/m.

965 + 2420 = 3385 K.G./m.

$M = \frac{3385 \times 6^2}{2} = 1210000$

$+ \frac{9090 \times 3.5 \times 2.5}{6} = 1330000$

2540000 K.G - c.m.

$V = 3385 \times \frac{6}{2} = 10155$

$9090 \times \frac{3.5}{6} = 5300$

15455 K.G.

15455 = 9.2 x 40 d

d = 42.4 c.m. Use 40 x 60 , d = 66

$As = \frac{2540000}{2100 \times 0.87 \times 66} = 38$ c.m^2

Bend 5 Ø

1st bar = 10%, 6 x 0.36 = 2.12 make it 2.00

1st pair = 30%, 6 x 0.25 = 1.50 make it 1.40

2nd pair = 50%, 6 x 0.18 = 1.06 make it 0.80

Check for shear at Col:-

V = 15435 K.G.

$Ac = \frac{15435}{10.6} = 1460$ c.m^2

Ac available = 60 x 130 = 7800 c.m^2

It is safe yet use bars anchored at both ends.

DESIGN OF BEAMS C15 - C4 Span 5.m.

w from wall = 0.3 x 1 x 3.2 x 2500 = 2420 K.G/m.

5 x 0.707 = 3.5

$\frac{3.55^2}{2} \times \frac{650}{5} = 820$ = 820

Total w = 3240 K.G/m.

M from w = $\frac{3240 \times 5^2}{10}$ = 810000 K.G - c.m.

3200 x 1.5 = 478000 K.G - c.m.

Total M =1,288000 K.G - c.m.

V = 2420 x $\frac{5}{2}$ = 6050

+ 3200

9250 K.G.

9250 = 9.2 x 40d

d = 25.2 make d = 50

As = $\frac{1288000}{1200 \times 0.87 \times 50}$ = 25 c.m^2

Use 10 - 18 m.m ∅ (25.45 c.m^2)

6 m.m ∅ stirrups. Spacing 10 c.m all through

DESIGN OF BEAM C2 Span 10.5 m.

$$w \text{ from wall} = 0.3 \times 3.2 \times 2500 = 2420 \text{ K.G/m}$$

$$\text{from slab } \frac{4.5^2}{2} \times \frac{650}{6.3} = 1020$$

$$4.2 \times 0.7 = 2.97$$

$$\frac{2.97}{2} \times \frac{650}{4.2} = 685$$

1705 K.G/m

2420

Total 4125 K.G/m.

$$M \text{ from } w = \frac{4125 \times 10.5^2}{10} = 4540000 \text{ K.G. c.m.}$$

$$M \text{ from Conc.L.} = 1450 \times 6.3 = 915000$$

Total M 5455000

$$V = 4125 \times \frac{10.5}{2} = 21500$$

2180

23680 K.G.

$$23680 = 9.2 \times 40d$$

$d = 64.5$ Use $d = 90$

$$As = \frac{5455000}{1200 \times 0.87 \times 90} = 60.5 \text{ c.m}^2$$

Use 10 - 28 m.m Ø/m.

Bend 5

$$\text{Bond} = \frac{23680}{5 \times 8.8 \times 0.87 \times 90} = 6.8 \text{ smaller than } 7.08 \quad \text{O.K.}$$

Check for shear with Col.

$V = 23680$

$$23680 \div 10.6 = 2240 \text{ c.m}^2$$

$$\text{Ac available} = 86 \times 40 = 3440 \text{ c.m}^2$$

O.K.

Continuation of Exterior Roof Beams:

1. C15, C1, C3 and C2 for floor and roof of second floor have the same design as C1, C3 and C2 all ready presented in the design of floor of first floor.
2. C5 and C6 in roof and floor of second floor have the same design as C4 in roof of basement.
3. Let us leave staircase and its beams till later.
4. Design of C4 in second floor roof and floor.

Span 7.m.

$3.3 \times 0.71 = 2.34$

$w = \frac{2.34^2}{2} \times \frac{650}{3.30} = 540$ K.G/m.

w from wall $= 2420$ K.G/m

2960 K.G/m.

$14600 - 4800 - 2960x = 0$

$x = 3.32$

$M = 14600 \times 3.32 = 48500$ K.G - m.

$4800 \times 1.02 = 4900$ K.G - m.

$2960 \times \frac{3.32^2}{2} = 16200$ K.G - m.

27400 K.G - m.

Assume wt of beam $= 0.40 \times 0.60 \times 2500 = 600$ K.G/m.

$M = 2100 \times 3.32 = 7000$

$600 \times \frac{3.32^2}{2} = 3200$

3800

Total $M = 27400 + 3800 = 31200$ K.G - m.

because it is not simply supported

actual $M = 3120000 \times 0.8 = 2500000$ K.G - c.m.

$V = 15820 + 2100 = 17920$

$17920 = 9.2 \times 40d$

$d = 48.5$ c.m.

$$As = \frac{2500000}{1200 \times 0.87 \times 67} = 36$$

Use 12 - 22 m.m ∅ (A = 45.64)

Bend 5 ∅s

$$U = \frac{17920}{7 \times 6.9 \times 0.87 \times 67} = 6.42 \text{ smaller than } 7.08$$

Bending:

1st bar 8.35%, $7 \times 0.38 = 2.65$

1st pair 25.05%, $7 \times 0.27 = 1.89$

2nd pair 41.75%, $7 \times 0.21 = 1.46$

Interior Roof Beams:-

1. Span 4.2 m.

C12, C13, C17, C18, C14.

$$w = (2.97)^2 \times \frac{650}{4.2} = \begin{array}{r} 900 \\ \underline{1220} \\ 2120 \end{array} \text{ K.G/m.}$$

$$M = \frac{2120 \times 4.2^2}{10} = 375000 \text{ K.G - cm.}$$

$$V = 2120 \times \frac{4.2}{2} = 4550$$

$$4550 = 9.2 \times 25d$$

d = 19.8 very small

Use 25 x 30

$$As = \frac{375000}{1200 \times 0.87 \times 39} = 9.3 \text{ c.m}^2$$

Use 4 - 18 m.m ∅

Beams Span 2.5, 3.4, 3.5

C11, C9, C8

900

$$w = (2.5)^2 \times \frac{650}{3.5} = \underline{1170}$$

2070 K.G/m.

$$V = 2070 \times \frac{3.5}{2} = 3630 \text{ K.G}$$

$$M = \frac{2070 \times 3.5^2}{10} = 255,000$$

$$3630 = 9.2 \times 25d$$

$$d = 16$$

Use 30 + 9 = 39

$$As = \frac{255000}{1200 \times 0.87 \times 39} = 6.25$$

Use 4 - 16 m.m ∅ ($A = 8.04$ c.m^2)

Beams Span 5m

C7

$$w = (3.5)^2 \times \frac{650}{5} = \frac{\begin{array}{r}900\\1600\end{array}}{2500\ K.G/m.}$$

$V = 2500 \times 2.5 = 6250$ K.g.

$6250 = 9.2 \times 25d$

$d = 27$ c.m say 39 c.m.

$$M = \frac{2500 \times 25}{10} = 625000$$

$$As = \frac{625000}{1200 \times 0.87 \times 39} = 15.5$$

Use 6 - 18 m.m Ø

Beams Span 6.2 m.

C10

$$w = (4.4)^2 \times \frac{650}{6.2} = 2030$$

$$\frac{\quad 900}{2930 \text{ K.G/m}}$$

$V = 2930 \times 3.1 = 9090$

$d = 9090 \div (9.2 \times 25) = 39$

Use 25×40 , $d = 48$

$$M = \frac{2930 \times 6.2^2}{10} = 1130000$$

$$As = \frac{1130000}{1200 \times 0.87 \times 48} = 22.5 \text{ c.m}^2$$

Use 8 - 20 m.m Ø (A = 25.13)

Bend 4 Øs

1st pair = 25% , $6.2 \times 0.3 = 1.86$ m.

2nd pair = 50% , $6.2 \times 0.18 = 1.12$ m.

Slab Beams in Floor and Roof of Second Floor

1. C10 the same above it.
2. C8 the same above it.
3. C11 the same above it.
4. C7 and C20 in second storey same as C10 in Roof of basement.
5. C14 - same above it.
6. C12 - same above it.
7. C21 - same as C7 in Roof of Basement.

DESIGN OF INTERIOR HIDDEN BEAMS IN FLOOR OF SECOND STOREY:

C22, C23, C24, C25.

Span 4.2

$4.2 \times 0.71 = 2.96$

$w = (2.96)^2 \times \frac{650}{4.2} = 1320$ K.G/m.

Wall = $0.12 \times 3.2 \times 1 \times 2500 = 960$ K.G/m

Total $w = 960 + 1320 = 2280$ K.G/m.

$M = \frac{2280 \times 4.2^2}{12} = 400000$

$400000 = 10.9 \times 12d^2$

$d^2 = 3060 \quad d = 55$ c.m.

Use total 60

$As = \frac{400000}{1200 \times 0.87 \times 55} = 7$ c.m^2

Use 5 - 16 m.m ∅

$V = 2280 \times \frac{4.2}{2} = 4800$ K.G

Bend 2 ∅s

$U = \frac{4800}{3 \times 5 \times 0.87 \times 55} = 6.66$ O.K.

DESIGN OF BEAM C-26

w from one way slab = 11,00 K.G/m

w from two way slab = $\left(\frac{6 \times 0.7}{2}\right)^2 \times \frac{1100}{6} = 1620$

w of beam = 0.30 x 0.50 x 2500 = 375 K.G/m

Total w = 1100 + 1620 + 375 = 3095 K.G/m

$M = \frac{3095 \times 36}{12} = 927500$ K.G - c.m.

$= 927500$ K.G - c.m.

$= 9.2\ bd^2 = 9.2 \times 30d^2$

$d^2 = 3400$, $d = 58.2$

$As = \frac{927500}{1200 \times 0.87 \times 58} = 15.3\ c.m^2$

Use 10 - 14 m.m ∅

Bend 4 ∅

$V = 3095 \times \frac{6}{7} = 9275$ K.gs

Bending

1st pair = 20% , 6 x 0.32 = 1.85m.

2nd pair = 40% , 6 x 0.24 = 1.25m.

Vc = 4.24 x 30 x 0.87 x 58 = 6400

Using 6 m.m ∅ stirrups

$S = \frac{2 \times 0.28 \times 1410 \times 0.87 \times 58}{2875} = 13.8$

Start with 10 c.m.

then 15 c.m.

then 20 c.m.

DESIGN OF BEAM C-27

Wall = o.12 x 1 x 3 x 2000 = 720

D.L. = 1.60 x .20 x 7500 = 800

L.L. = = 450

Tile and sand = 150

2120 K.G/m

$$M = \frac{2120 \times 36}{12} = 636000$$

$$= 10.9 \times 160d^2$$

$d^2 = 356$, $d = 18.9$ O.K.

$$As = \frac{636000}{1200 \times 0.87 \times 18} = 34$$

Use 17 - 16 m.m Ø bottom

and 10 - 14 m.m Ø top, they help for compression.

Administration Ground Beams

DESIGN G.BEAM C32   Span 5m.

Wall = 0.30 x 1 x 4.2 x 2500 = 3140

w = 0.40 x 0.50 x 2500 = 500

3640

$$M = \frac{3640 \times 25}{12} = 760000 \text{ K.G - c.m.}$$

$$= 10.9 \times 40d^2$$

$d^2 = 1740$, $d = 41.7$ say 44 + 6 = 50 c.m.

$$As = \frac{760000}{1200 \times 0.87 \times 44} = 16.7 \text{ c.m}^2$$

Use 8 - 16 m.m Ø = 18.1 c.m^2

Bend 4 Ø

1st pair = 25% , 5 x 0.3 = 1.5m.

2nd pair = 50% , 5 x 0.21 = 1.0m.

DESIGN OF BEAMS C34 - 35 Span 1.5m.

$$\text{Wall} = 2400 = 2400$$

$$w = 0.40 \times 0.40 \times 2500 = \underline{400}$$

$$2800 \text{ K.G/m.}$$

$$M = \frac{2800 \times 1.5^2}{12} = 52500 \text{ K.G - c.m.}$$

$$= 10.9 \times 40d^2$$

$d^2 = 121$, $d = 11$ c.m.

$$As = \frac{52500}{1200 \times 0.87 \times 36} = 1.4 \text{ c.m}^2$$

Use 3 - 12 m.m ∅

DESIGN OF BEAM C37, 36, 39, 38   Span 4.2

Wall = 0.12 x 1 x 3 x 2500 = 900

w = 0.25 x 0.35 x 2500 = 217

1117

$$M = \frac{117 \times 4.2^2}{12} = 164000$$

$$= 10.9 \times 25d^2$$

$d^2 = 605$, $d = 24.5$

Use 25 x 30 c.m.

$$As = \frac{164000}{1200 \times 0.87 \times 25} = 6.3 \text{ c.m}^2$$

Use 4 - 14 m.m Ø

2 - 12 m.m Ø

DESIGN OF BEAM C - 40   Span 5m.

Wall = = 2400

$w = o.40 \times 0.50 \times 2500 = \underline{500}$

2900 K.G/m

M uniform load = 2900 x 25 = 600000 K.G - c.m.

M concentrated load

$$= \frac{2340 \times 5}{4} = 280000$$

Total M = 280000 + 600000 = 880000 K.G - c.m.

$= 10.9 \times 40d^2$

$d^2 = 2030$, d = 45, + 5 = 50.

$$As = \frac{880000}{1200 \times 0.87 \times 45} = 18.8 \ c.m^2$$

Use 11 - 16 m.m ∅ = 22.11

Bend 5 ∅s

$$U = \frac{8420}{6 \times 5.05 \times 0.87 \times 45} \quad O.K.$$

Bending

1st bar, 9%, 5 x 0.4 = 2m. use 1.90m.

1st pair, 27%, 5 x 0.29 = 1.45 use 1.40m.

2nd pair, 45%, 5 x 0.22 = 1.10m use 0.90m.

For stirrups use 6 m.m ∅ all through.

DESIGN OF BEAM C - 41 Span 5m.

Wall = 0.12 x 3.2 x 2500 = 960

w = 0.30 x 0.45 x 2500 = 338

1298 K.G/m.

$$M = \frac{1298 \times 5^2}{12} = 270000$$

$$+ \quad 280000$$

550000 K.G c.m.

$$550000 = 10.9 \times 0.30d^2$$

$$d^2 = 1690, \quad d = 41, + 4 = 45$$

$$As = \frac{550000}{1200 \times 0.87 \times 41} = 12.9$$

Use 8 - 16 m.m Ø

Bend 3 Ø

$$U = \frac{5420}{5 \times 5.65 \times 0.87 \times 41} = 6.08 \quad O.K.$$

Bending

1st bar 12.5%, 5 x 0.35 = 1.75m.

1st pair 37.5%, 5 x 0.25 = 1.25m.

Use 6 m.m Ø stirrups all through

DESIGN OF BEAM C - 42   Span 6.5m.

$w = 0.40 \times 0.60 \times 2500 = 600$

$Wall = \qquad = 960$

1560

$M = \frac{1560 \times 6.5^2}{12} = 550000$ K.G - c.m.

$3600 \times 2.2 = 792000$

Total $M = 792000 + 550000 = 1342000$

$= 10.9 \times 40d^2$

$d^2 = 3100$, $d = 55, + 5 = 60$

$As = \frac{1342000}{1200 \times 0.87 \times 55} = 23.4$ c.m^2

Use 10 - 18 m.m ∅ $A = 25.45\ cm^2$

Bend 5 ∅s

$V = 3600 + 1560 \times 3.25 = 8650$ K.G

$U = \frac{8650}{5 \times 5.65 \times 0.87 \times 55} = 6.5$ O.K.

$Vc = 4.24 \times 40 \times 0.87 \times 55 = 8100$

$V = 8650$

Use 6 m.m ∅ stirrups at 10 c.m.

Bending

1st bar, 10%, 6.5 x 0.4 = 2.60 make it 2.50m.

1st pair, 30%, 6.5 x 0.28 = 1.80 make it 1.70m.

2nd pair, 50%, 6.5 x 0.21 = 1.40 make it 1.00m.

DESIGN OF MAIN STAIR CASES

Slab

L.L. = 350 $K.G/m^2$

$$D.L. = 0.21 \times 1 \times \frac{36}{32} \times 2500 = 590$$

Tiles and sand = 150

Total W = 150 + 590 + 350 = 1090 $K.G/m^2$

$$M = \frac{1090 \times 6.5^2}{10} = 4620 \text{ K.G - m}$$

$$= 462000 \text{ K.G - c.m.}$$

Assume that 2/3 of the moment is carried by beam action and 1/3 by cantilever.

$$M = 462000 \times \frac{2}{3} = 310000$$

$$= 10.9 \times 100d^2$$

$d^2 = 284$ d = 16.9 c.m.

d is actually 8 + 10 = 18 c.m. O.K.

$$As = \frac{310000}{1200 \times 0.87 \times 18} = 16.7$$

Use 14 m.m Ø   A = 1.54

$$s = 100 \times \frac{1.54}{16.7} = 9.25 \text{ c.m. say 9 c.m.}$$

Therefore use 14 m.m Ø at 9 c.m.

and 5 - 8 m.m Ø/m transversally

Cantilever action

w taken by Cantilever

$$1090 \times \frac{1}{3} = 364 \text{ K.G/m}^2$$

$$M = \frac{364 \times 4}{2} = 72800 \text{ K.G - c.m.}$$

$$= 10.9 \times 100\ d^2$$

d is smaller than 16.9 O.K.

$$As = \frac{72800}{1200 \times 0.87 \times 18} = 3.87 \text{ c.m}^2/\text{m}$$

3.87 ÷ 3 steps = 1.29 $c.m^2$/step.

Use 1 - 14 m.m ∅/ step.

GROUND BEAMS OF MAIN STAIR CASES.

C-28 - C-30 Span 6.5m.

Wall = 0.30 x 1 x 3.2 x 2500 = 2400

w = 0.40 x 0.70 x 2500 = 700

From stair 364 x(2 x 1) = 728

3828 K.G/m.

$$M = \frac{3828 \times 6.5^2}{10} = 1610000 \text{ K.G - c.m.}$$

$$= 10.9 \times 40\, d^2$$

$d^2 = 3700$

d = 60.5 say 64, + 6 = 70 O.K.

$$As = \frac{1610000}{1200 \times 0.87 \times 64} = 24.3$$

Use 10 - 18 m.m ∅

Bend 5 ∅

1st bar = 10%, 6.5 x 0.38 = 2.47m.

1st pair = 30% 6.5 x 0.28 = 1.82m.

2nd pair = 50% 6.5 x 0.18 = 1.17m.

ROOF BEAMS OF MAIN STAIR CASES.

C-28 which is at first landing and at first floor level.

Span 4.2m.

for one (m) width (w) from stair = 2/3 x 1090 = 728

$w = 728 \times \frac{6.5}{2} =$ =2370

Assume .30 x 0.30 x 2500 = 225

Wall 0.12 x 1 x 1.6 x 2500 = 480

Total w = 3075

$M = \frac{3075 \times 4.2^2}{10} = 540000$ K.G - c.m.

V = 3075 x 2.1 = 6450

= 9.2 x 30d

d = 24 c.m.

Actual d = 45

$As = \frac{540000}{1200 \times 0.87 \times 45} = 11.60$

Use 4 - 20 m.m ϕ

BEAM C30 at FIRST FLOOR LEVEL. Span 4.2

w from stair = 2370

w from floor slab =

$\frac{1 \times 5.4}{2} \times 1100 = 2970$

$0.30 \times 0.30 \times 2500 = 250$

5590 K.G/m

$M = \frac{5590 \times 4.2^2}{10} = 985000$

$V = 5590 \times 2.1 = 11800$

$= 9.2 \times 30d$

d = 43, 45

Use 30 x 30 c.m.

$As = \frac{985000}{1200 \times 0.87 \times 45} = 21 \text{ c.m}^2$

Use 12 - 16 m.m ∅

Bend 6 ∅s

1st pair 16%, 0.32 x 4.2 = 1.34 make it 1.30m.

2nd pair 32%, 0.24 x 4.2 = 1.00 make it 0.90m.

3rd pair 50%, 0.17 x 4.2 = 0.72 make it 0.50 m.

ROOF BEAMS C - 29 - C 31 OF MAIN STAIR CASES. Span 6.5

$$Slab = 1100 \times \frac{4.6^2}{2} \times \frac{1}{6.5} = 1790 \text{ K.G/m.}$$

Wall = 0.12 x 3.2 x 2500 = 960

Beam = 0.30 x 0.60 x 2500 = 450

3200 K.G/m.

$$V = 3200 \times \frac{6.5}{2} = 10400$$

$$= 9.2 \times 30 \times d$$

d = 38 use d = 74 c.m.

$$M = \frac{3200 \times 6.5^2}{10} = 1360000$$

$$As = \frac{1360000}{1200 \times 0.87 \times 74} = 17.6$$

Use 10 - 16 m.m Ø

Bend 5 Øs

$$U = \frac{10400}{5 \times 5.02 \times 0.87 \times .74} = 6.5$$

Bending

1st bar 10% at 240m.

1st pair 30% at 160m.

2nd pair 50% at 0.85m.

C H A P T E R   F O U R

Class-Room Division.

SLABS - RIBBED FLOORS

Using 50 x 25 x 20 c.m. blocks

One way floor = - Wt of 62 c.m strip

5 blocks = 5 x 17		= 85 K.G.
6 c.m concrete covering	= 0.06 x 0.62 x 2500	= 92.5 K.G.
Ribs	0.12 x 0.20 x 1 x 2500	= 60.0 K.G.
L.L.		=450.0 K.G.
Sand and tiles		=150.0 K.G.
	Total w	= 837.5 K.G.

Two way floor:-

Blocks 18 x 17 =		= 307 K.G.
Concrete covering	= 1.98 x 1.68 x 0.06 x 2500	= 500 K.G.
Concrete ribs	=(4 x 1.98 + 4 x 1.20 x 0.20x2500	= 640 K.G.
L.L.	= 450 x 1.68 x 1.98	=1500 K.G.
Sand and tiles	= 150 x 1.68 x 1.98	= 500K.G.
	Total w	= 3447 K.G.

$$w = \frac{3447}{1.68 \times 1.98} = 1040 \text{ K.G/m}^2 \text{ say } 1100$$

DESIGN OF 4 m - CORRIDOR

$$M = \frac{837.5 \times 16}{10} = 134000 \text{ K.G - c.m.}$$

$$V = 837.5 \times 2 = 1675 \text{ K.G.}$$

$$= 9.2 \times 12\, d$$

$d = 15.2$ c.m smaller than 23.5 O.K.

$$As = \frac{134000}{1200 \times 0.87 \times 23.5} = 5.5 \text{ cm}^2$$

Use 1 - 18 m.m Ø)
1 - 20 m.m Ø) A = 5.68

Check for v

$$v = \frac{1675}{12 \times 0.87 \times 23.5} = 8.35 \text{ smaller than } 10.6 \text{ O.K.}$$

$$\text{Bond } U = \frac{1675}{11.93 \times 0.87 \times 23.5} = 6.8 \text{ smaller than } 7.08 \text{ O.K.}$$

Use 6 m.m Ø above blocks

and 6 m.m Ø stirrups at 20 c.m.

DESIGN OF ROOFS 12 6 x 8 meters

$w = 1100 \times 0.62 = 680$

is taken by short side $= \left(\frac{8}{6} - \frac{1}{2}\right) \times 680 = 565$

Short direction:-

$$M = \frac{565 \times 36}{12} = 168900$$

$$As = \frac{168900}{1200 \times 0.87 \times 23.5} = 6.9 \text{ cm}^2$$

Use 2 - 22 m.m ∅ $A = 7.6 \text{ cm}^2$

$V = 565 \times 3 = 1695$

$$v = \frac{1695}{12 \times 0.87 \times 23.5} = 6.9$$ smaller than 10.6 O.K.

$$U = \frac{1695}{2 \times 6.9 \times 0.87 \times 23.5} = 6.8$$ smaller than 7.08 O.K.

Long direction:-

$$M = \frac{110 \times 64}{10} = 70000$$

$$As = \frac{70000}{1200 \times 0.87 \times 23.5} = 2.85 \text{ c.m}^2$$

Use 2 - 18 m.m ∅ ($A = 5.09 \text{ cm}^2$)

DESIGN OF ROOF 17 7 x 8 m.

Short side:-

$$w = \left(\frac{8}{7} - \frac{1}{2}\right) \times 680 = 625$$

$$M = \frac{625 \times 49}{10} = 305000$$

$$As = \frac{305000}{1200 \times 0.87 \times 23.5} = 12.2 \text{ cm}^2$$

Use 2 - 28 m.m ∅

$$V = 625 \times 3.5 = 2180$$

$$v = \frac{2180}{12 \times 0.87 \times 23.5} = 8.9 \text{ smaller than } 10.6 \quad \text{O.K.}$$

$$U = \frac{2180}{17.6 \times 0.87 \times 23.5} = 6.05 \text{ smaller than } 7.08 \quad \text{O.K.}$$

Long direction:-

$$M = \frac{55 \times 64}{1200 \times 0.87 \times 23.5} = 0.144 \text{ cm}^2$$

Use 2 - 12 m.m ∅

DESIGN OF ROOF 15   9 x 9 meters.

$$\frac{680}{2} = 340 \text{ in both directions}$$

$$M = \frac{340 \times 81}{10} = 2750$$

$$As = \frac{275000}{1200 \times 0.87 \times 23.5} = 11.3 \text{ cm}^2$$

Use 2 - 28 m.m Ø

Check for shear

$$V = 340 \times 4.5 = 1185$$

$$v = \frac{1185}{12 \times 0.87 \times 23.5} = 4.85 \text{ smaller than } 10.6$$

$$U = \frac{1185}{17.6 \times 0.87 \times 23.5} = 3.3 \text{ O.K.}$$

DESIGN OF ROOFS 6 x 6

w = 340 in both directions

$$M = \frac{340 \times 36}{10} = 1220$$

$$As = \frac{122000}{1200 \times 0.87 \times 23.5} = 5.0\ cm^2$$

Use 2 - 18 m.m ∅

Shear is safe

$$Bond\ U = \frac{1020}{2 \times 565 \times 0.87 \times 23.5} = 4.4 \quad O.K.$$

ROOF BEAMS

DESIGN OF BEAMS D - 1 Span 6.

w from wall = 0.12 x 3.2 x 2500 = 960 K.G/m.

w from slab = $\frac{4.61^2}{2} \times \frac{1100}{6.5}$ = 1800 K.G/m.

1800

Total w = 4560

$M = \frac{4560 \times 36}{10} = 1920000$ K.G c.m.

V = 4560 x 3 = 14800

= 9.2 x 30 d

d = 54 c.m make it 55 c.m.

$As = \frac{1920000}{1200 \times 0.87 \times 55} = 33.8\ cm^2$

Use 12 - 20 m.m ∅

Bend 5 ∅ U = O.K.

Bending

1st bar at 2.28 m.

1st pair at 1.62 m.

2nd pair at 1.26 m.

Use 6 m.m ∅ stirrups at 10, 15, 20 c.m.

DESIGN OF ROOF BEAMS D - 2 Span 8m.

0.12 x 3.2 x 2500 = 960 K.G/m.

$5.7^2 \times \frac{1100}{8}$ = 4500

Total 5460 K.G/m.

$M = \frac{5460 \times 64}{10} = 3500000$ K.G - c.m.

V = 5460 x 4 = 21700

= 40 x 9.2d.

d = 59 make it 62 c.m.

$As = \frac{3500000}{1200 \times 0.87 \times 62} = 54.2\ c.m^2$

Use 18 - 20 m.m ø

Bend 9 ø U = safe.

1st bar at 3.35m.

1st pair at 2.56m.

2nd pair at 2.07m.

3rd pair at 1.68m.

4th pair at 1.36m.

Use 6 m.m ø at 10, 15, 20 c.m.

DESIGN OF BEAM D - 3 $l = 1.6m.$

from wall: $0.12 \times 3.2 \times 2500 = 960$ K.G/m.

from slab: $1.6 \times 0.71 = 1.1$

$$\frac{1.1^2}{2} \times \frac{1100}{1.6} = 415 \text{ K.G/m.}$$

Total w = 1375 K.G/m.

$$M = \frac{1500 \times 1.6^2}{10} = 38300 \text{ K.G - c.m.}$$

$$V = \frac{1500 \times 1.6}{2} = 1200 \text{ K.G.}$$

$$d = \frac{1200}{9.2 \times 30} = 4.35$$

Keep tha same section as D1 for architectural purposes.

<u>DESIGN OF BEAM D - 4</u>

$0.30 \times 3.2 \times 2500 = 2400$

from slab $= \underline{1800}$

Total $w = 4200$

$$M = \frac{4200 \times 36}{10} = 1{,}770{,}000 \text{ K.G - c.m.}$$

$V = 4200 \times 3 = 13700$

$d = 37.2$ make it 60

$A_s = 28.3$

Use 10 - 20 m.m Ø

Bend 4 Ø U = O.K.

Bending of bars:

1st pair at 1.8m.

2nd pair at 1.26m.

Use 6 m.m Ø stirrups at 10, 15, 20 c.m.

DESIGN OF BEAM D - 5   l = 9.m.

w from wall = 2400

w from slab = 2500

Total w = 4900 K.G/m.

M = 405-000 K.G - c.m.

V = 5000 x 4.5 = 22500

d = 61.5 make it 62 c.m.

As = 62.5 $c.m^2$

Use 12 - 22 m.m Ø, A = 45.62

and 6 - 20 m.m Ø, A = 18.85

Total A = 64.47

Bend 9 Ø   U = O.K.

1st bar at 3.78m.

1st pair at 2.88m.

2nd pair at 2.34m.

3rd pair at 1.89m.

4th pair at 1.53m.

Use 6 m.m Ø stirrups at 6, 11, 20 c.m.

DESIGN OF BEAM D - 6   Span = 8m.

w from wall = 960 K.G/m.

w of beam = 548 K.G/m.

Total w =1500 K.G/m.

M = 960000 K.G c.m.

d = 47 say 49 c.m.

As = 18.9 $c.m^2$

Use 10 - 16 m.m Ø , A = 20.1

Bend 5 Ø   U is safe.

Bending of bars:

1st bar at 3.00m.

1st pair at 2.00m.

2nd pair at 1.36m.

Use 6 m.m Ø stirrups at 10, 15, 20 c.m.

DESIGN OF BEAM D - 7

0.40 x 0.50 beam.

w from wall = 960 K.G/m.

w from beam = 500 K.G/m.

Total w =1460 K.G/m.

$$M = \frac{1460 \times 36}{10} = 615{,}000 \text{ K.G c.m.}$$

$d^2 = 1410$

d = 37.5 c.m. say 44.c.m.

$As = 13.4\ cm^2$

Use 10 - 14 m.m ∅, $A = 15.39\ cm^2$

Bend 5 ∅ U is safe.

Bending of bars:

1st bar at 3.00m.

1st pair at 2.00m.

2nd pair at 1.36m.

Use 6 m.m ∅ stirrups at 10, 15, 20 c.m.

DESIGN OF BEAM D - 8 Span 3 m.

Assuming 0.3 x 0.25 beam.

w from wall = 960 K.G/m.

w from beam = 188 K.G/m.

w total = 1148 K.G/m.

$$M = \frac{1148 \times 9}{10} = 103000 \text{ K.G - c.m.}$$

$d^2 = 380$, $d = 19.5$ say 24 c.m.

As = 4.1 cm^2

Use 6 - 14 m.m ∅, A = 9.24 c.m.

Bend 3 ∅ U is safe.

Bending is at one fifth of span.

Use 6 m.m stirrups at 10, 15, 20 c.m.

<u>DESIGN OF BEAM D - 9</u>

Assume 0.40 x 0.60 beam.

w from wall = 2400 K.G/m.

w from beam = 600 K.G/m.

w total = 3000 K.G/m.

M = 1270000 K.G - c.m.

d^2 = 2900, d = 54

As = 22.5 $c.m^2$

Use 12 - 16 m.m ∅, A = 24.12

Bend 6 ∅ U is safe.

Bending of bars:

1st pair at 1.92m.

2nd pair at 1.44m.

3rd pair at 1.02m.

Use 6 m.m ∅ stirrups at 10, 15, 20 c.m.

DESIGN OF BEAM D - 11 Span 9m.

Assume 0.40 x 0.85 beam.

w from wall = 2400 K.G/m.

w from beam = 850 K.G/m.

w total = 3250 K.G/m.

$$M = \frac{3250 \times 81}{10} = 2,650,000 \text{ K.G c.m.}$$

$$d^2 = \frac{2650000}{10.9 \times 40} = 6050$$

d = 77.5 make it 80 c.m.

$$As = \frac{2650000}{10.9 \times 0.87 \times 80} = 31.3 \text{ c.m}^2$$

Use 10 - 20 m.m Ø , $A = 31.42 \text{ c.m}^2$

Bend 5 Ø

$$U = \frac{14600}{5 \times 6.3 \times 0.87 \times 80} = 6.65 \text{ smaller than } 7.08$$

Vc = 4.24 x 40 x 0.87 x 80 = 12100

$$s = \frac{2 \times 0.28 \times 1410 \times 0.87 \times 80}{14600 - 12100} = 21.7$$

Use 6 m.m Ø at 10, 15, 20 c.m.

Bending

1st bar 10%, 9 x 0.38 = 3.42 make it 3.30m.

1st pair 30%, 9 x 0.25 = 2.25 make it 2.20m.

2nd pair 50%, 9 x 0.17 = 1.53 make it 1.40m.

C H A P T E R   F I V E

Columns Footings

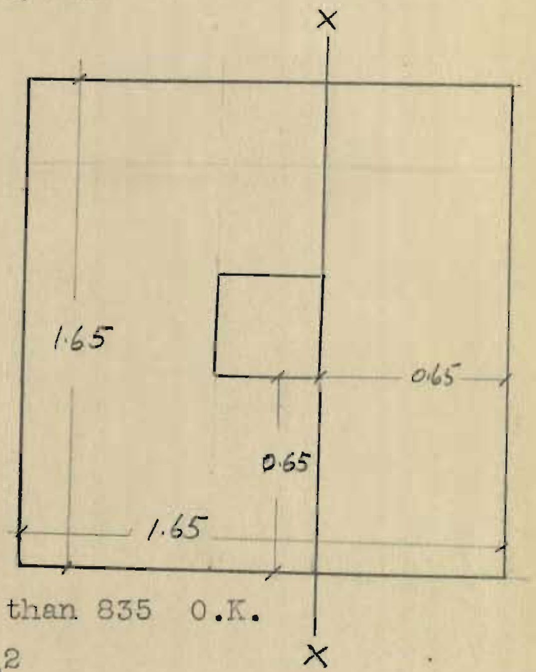
and

Retaining Wall

DESIGN OF COLUMNS 1 - 2 - 5 - 14 - 9 - 12

for 6 bags of cement/m^3 $Fc = 60$ K.G/$c.m^2$

Col:-



$Ac = \frac{12500}{60} = 208$ $c.m^2$

Assume .35 x .35

$\frac{h}{d} = \frac{750}{35} = 21.4$

P(1.3 - 0.03 x 21.4)

P = 32700/0.66 = 5000

$\therefore Ac = \frac{50000}{60} = 835$

.35 x .35 = 1220 greater than 835 O.K.

$As = \frac{1}{100} \times 1220 = 12.2$ $c.m^2$

Use 8 - 14 m.m Ø, and use

6 m.m Ø stirrups at 20 c.m.

Lower part of Col.

$Ac = \frac{67740}{60} = 1124$ $c.m^2$

Use as before 35 x 35

FOOTING:-

Assume $W = 2.5\ K.G/c.m^2$

$$A = \frac{67740}{2.5} = 27200\ c.m^2$$

Use 1.65 x 1.65 c.m.

$$Mx - x = \frac{2.5 \times 65^2 \times 165}{2}$$

$$= 870000$$

$$d \text{ by shear} = \frac{67,700}{15 \times 4 \times 35} = 32.2\ c.m$$

$$+\ 10\ \ c.m$$

$$42\ \ c.m.$$

$$\text{steel} = \frac{0.85 \times 870000}{1200 \times 0.87 \times 32} = 22.2$$

Use 16 - 14 m.m ∅ , A = 24.63.

The same section of steel for the other direction. Using 10 c.m. spacing, and Bending 4 ∅ of each direction.

DESIGN OF COLs 3 - 16 - 4 - 15

Use 35 x 35

$As = 0.01 \times 1220 = 12.2\ cm^2$

Use 8 - 14 m.m Ø

Lower part of col is the same

35 x 35

FOOTING:-

$$A = \frac{45300}{2.5} = 18100$$

Use 135 x 135

$$M = \frac{135 \times 50 \times 50 \times 2.5}{2} = 418,000$$

$$As = \frac{418000 \times 0.85}{1200 \times 0.87 \times 22} = 15.3\ c.m^2$$

$$d = \frac{45300}{4 \times 35 \times 15} = 21.6\ ,\ + 10 = 32\ c.m.$$

Use 10 - 14 m.m Ø , $A = 15.39\ c.m^2$

Spacing 14 c.m.

DESIGN OF COLs 10 - 11

Col:

$$P\left(1.30 - 0.03 \times \frac{620}{35}\right)$$

Assume 35 x 35

$$P = \frac{77700}{0.77} = 100100 \text{ K.Gs}$$

$$Ac = \frac{100100}{60} = 1690$$

then use 41 x 41

$As = 17.7$ $c.m^2$

Use 10 - 16 m.m ∅ , $A = 20.1$ $c.m^2$

FOOTING:-

$$A = \frac{77700}{2.3} = 31,000$$

Use 180 x 180 = 32200

$$w = \frac{77,700}{32,200} = 2.4$$

$$M = \frac{2.4 \times 180 \times 69^2}{2} = 1,030,000$$

$$d = \frac{77700}{4 \times 42 \times 15} = 30.7 \text{ , } + 10 = 41$$

$$As = \frac{0.85 \times 1,030,000}{1200 \times 0.87 \times 31} = 27.2$$

Use 14 - 16 m.m ∅, spacing 13 c.m. Bend 4 ∅

DESIGN OF COLs. 6 - 7 - 8 - 13

In basement: $Ac = \frac{142040}{60} = 2420$

Use $50 \times 50 = 2500$ c.m^2

$As = 2500 \times 0.5/100 = 12.5$ c.m^2

Use 8 - 14 m.m Ø , $A = 12.32$ cm^2

Use 6 m.m Ø stirrups at 20 c.m.

w of Col approx. = $0.5 \times 0.5 \times 10 \times 2500 = 6250$ K.Gs

$6250 + 142040 = 148290$ K.Gs.

FOOTING:-

$$\frac{148290}{2.5} = 59200$$

Use $245 \times 245 = 60000$

$$w = \frac{148290}{60000} = 2.48.$$

$$Mx - x = \frac{2.48 \times 245 \times (97.5)^2 \times 0.8}{2}$$

$= 2,320,000$ K.G c.m.

$$d = \frac{148290}{4 \times 45 \times 15} = 55$$

Use $55 + 10 = 65$

$$As = \frac{2320000}{1200 \times 0.87 \times 55} = 40.5 \text{ c.m}^2$$

Use 20 - 16 m.m Ø. Bend 6 Øs.

Same Col in first floor

$$Ac = \frac{92690}{60} = 1550$$

Use 40×40

$$As = 1600 \times \frac{-5}{100} = 8 \text{ c.m}^2$$

Use 4 - 16 m.m Ø

Col. in second floor

$$Ac = \frac{52750}{60} = 880$$

Use 30 x 30

$$As = 9000 \times \frac{.5}{100} = 4.5 \ c.m^2$$

Use 4 - 12 m.m Ø , $A = 4.52 \ c.m^2$

DESIGN OF COLs 17 - 30

Col.in first floor

$$Ac = \frac{85680}{60} = 1430$$

Use 40 x 40

wt of Col = 0.4 x 0.4 x 10 x 2500 = 4000 K.G.

$$As = \frac{0.5}{100} \times 1600 = 8 \text{ c.m}^2$$

Use 4 - 16 m.m Ø

Use 6 m.m Ø stirrups at 20 c.m.

FOOTING:-

$$\frac{89680}{2.5} = 35800$$

Use 190 x 190 = 36000

$$\text{Actual } w = \frac{89680}{36000} = 2.5$$

$$M = 2.5 \times 1.90 \times \left(\frac{75}{2}\right)^2 \times 0.8 = 1055000 \text{ K.G c.m.}$$

89680 = d x 15 x 160

d = 37.2   48

$$As = \frac{1055000}{1200 \times 0.87 \times 38} = 27.4 \text{ c.m}^2$$

Use 18 - 14 m.m Ø 27.71 c.m^2 and bend 4 Øs upward.

Col. in second floor

$$R = \frac{2}{3} \times 89680 = 59500$$

$$Ac = \frac{59500}{60} = 1020 \text{ c.m}^2$$

$$As = \frac{0.5}{100} \times 1020 = 5.10$$

Use 4 - 14 m.m Ø

DESIGN OF COLs. 18 - 20, 21, 21, 26, 29.

Col.

$$Ac = \frac{77250}{60} = 1290$$

Use 36 x 36

w of 10 m. Col. = 0.129 x 10 x 2500 = 3220

77250 + 3220 = 80470 K.Gs

$$Ac = \frac{80470}{60} = 1340$$

Use 38 x 38

$$As = \frac{1450}{100} \times 0.5 = 7.25 \text{ c.m}^2$$

Use - 4 - 16 m.m Ø

Use 6 m.m Ø stirrups at 20 c.m.

FOOTING:-

$$\frac{8070}{2.5} = 32200$$

Use 180 x 180

$$d = \frac{80470}{4 \times 38 \times 15} = 35.5 \quad \text{say } 36 + 10 = 46$$

$$M = 2.5 \times \frac{71^2}{2} \times 180 \times 0.8 = 910000$$

$$As = \frac{910000}{1200 \times 0.87 \times 36} = 24.2$$

Use 16 - 14 m.m Ø , bend 4 Øs.

Col. in second floor

$$80470 \times \frac{2}{3} = 53800$$

$$Ac = \frac{53800}{60} = 900 \text{ , use } 30 \times 30$$

$$As = \frac{900}{100} \times 0.5 = 4.5 \text{ c.m}^2$$

Use 4 - 12 m.m Ø , A = 4.52

and 6 m.m Ø stirrups at 20 c.m.

DESIGN OF COLs. 19, 23, 25, 27, 28, 24

Col:-

$$Ac = 70040 \div 60 = 1170$$

Use 35 x 35 = 1230

$$As = \frac{1230}{100} \times 0.5 = 6.15 \text{ c.m}^2$$

Use 6 - 14 m.m ∅

FOOTINGS:-

$$\frac{70040}{2.5} = 28000, \text{ use } 170 \times 170$$

$$\text{Actual } w = \frac{70040}{29000} = 2.42$$

$$d = \frac{40040}{4 \times 35 \times 15} = 35.5 \text{ , } + 10 = 45$$

$$M = 2.42 \times \left(\frac{67.5}{2}\right)^2 \times 170 \times 0.8 = 750000$$

$$As = \frac{750000}{1200 \times 0.87 \times 35} = 20.5$$

Use 14 - 14 m.m ∅ Bend 4 ∅

Second floor Col.

$$R = 70040 \times \frac{2}{3} = 46500$$

$$Ac = \frac{46500}{60} = 780$$

Use 30 x 30 = 900

$$As = \frac{900}{100} \times 0.5 = 4.5 \text{ c.m}^2$$

Use 4 - 12 m.m ∅

and 6 m.m ∅ stirrups at 20 c.m.

DESIGN OF COLs (GROUPS) A B 31, 33, 39, 45, 49, 53, 57, 16, 48, 44, 38, 52, 66, 69, 35, 37, 34, 40, 46, 50, 54, 58, 62, 63, 64, 59, 55, 51, 47, 43, 41.

Col:-

$$Ac = \frac{123,150}{60} = 2050 \ c.m^2$$

Use 45 x 45 $c.m^2$

$$As = 45 \times 45 = \frac{0.5}{100} = 10.22 \ c.m^2$$

Use 8 - 14 m.m ∅

and 6 m.m ∅ at 20 c.m.

wt of Col. = 0.45 x 0.45 x 10 x 2500 = 5000

P = 123150

5000

128150 K.Gs.

FOOTING:-

$$\frac{128150}{2.5} = 51200$$

Use 225 x 225

$$M = 2.5 \times 225 \times \left(\frac{90}{2}\right)^2 \times 0.8 = 1830000$$

$$d = \frac{128150}{4 \times 45 \times 15} = 48, + 10 \quad \text{say } 60 \text{ c.m.}$$

$$As = \frac{1830000}{1200 \times 0.87 \times 50} = 35 \ c.m^2$$

Use 18 - 16 m.m ∅

DESIGN OF COLs. GROUP C, 56, 60, 65.

Cols:-

$$Ac = \frac{109000}{60} = 18200$$

Use 43 x 43 = 1850

$$As = 1850 \times \frac{0.5}{100} = 9.25 \text{ c.m}^2$$

Use 6 - 14 m.m Ø

and 6 m.m Ø stirrups at 20 c.m.

FOOTING:-

$$\frac{114000}{2.5} = 45500$$

Use 215 x 215

$$M = 2.5 \times 215 \times \frac{86^2}{2} \times 0.8 = 158000$$

$$d = \frac{114000}{4 \times 43 \times 15} = 44.5, + 10 \quad \text{say } 55$$

$$As = \frac{1580000}{1200 \times 0.87 \times 45} = 33.4$$

Use 17 - 16 m.m Ø = 34.17 c.m^2

DESIGN OF COLs. GROUP D, 68, 67

Col:-

$$Ac = \frac{100000}{60} = 1668 \text{ c.m}^2$$

Use 42 x 42 c.m. 1760

$$As = \frac{1760}{100} \times 0.5 = 8.8 \text{ c.m}^2$$

Use 6 - 14 m.m Ø

wt of Col. = 0.42 x 0.42 x 10 x 2500 = 4400

FOOTING:-

$$\frac{104000}{2.5} = 41600$$

Use 210 x 210 = 44000

$$\text{Actual } w = \frac{104000}{44000} = 2.36$$

$$M = 2.36 \times 210 \times \left(\frac{84}{2}\right)^2 = 1750000$$

$$d = \frac{104000}{4 \times 42 \times 15} = 41.3, + 10 = 52$$

$$As = \frac{1750000}{1200 \times 0.87 \times 42} = 32 \text{ c.m}^2$$

Use 16 - 16 m.m Ø Bend 4 Øs.

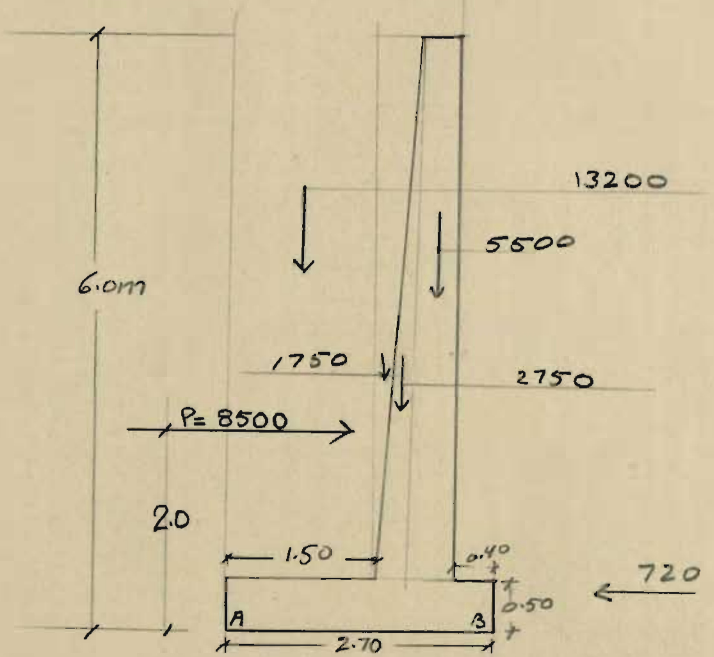
DESIGN OF RETAINING WALL

Clear Height = 4.25

and about = 1.75 m below groung

Total height = 6.00 m.

w = 2.5 K.G/c.m^2



$w = 100$ lb/cu-ft $= \frac{100}{2.24} (3.3)^3 = 1600$ K.G/m^3

$$P = \frac{1}{2} wh^2 \frac{1 - \sin \phi}{1 + \sin \phi} = \frac{1}{2} \times 1600 \times 6^2 \frac{0.456}{1.544}$$

= 8500 K.Gs.

$0.40 \times 5.50 \times 2500 = 5500$

$\frac{0.40 \times 5.5}{2} \times 2500 = 2750$

$0.5 \times 27 \times 2500 = 3370$

$1.5 \times 5.5 \times 1600 = 13200$

$\frac{0.4 \times 5.5}{2} \times 1600 = 1750$

Overturning Moment =

$8500 \times 2 = 17000$

$720 \times \frac{1.75}{3} = 420$

17420 K.G - m.

Stabilizing Moments

$5500 \times 0.6 = 3300$

$2750 \times 0.93 = 2560$

$3370 \times 1.35 = 4540$

$1750 \times 1.06 = 1850$

$13200 \times 1.95 = 25700$

37950

160

38110 K.G - m.

$F.S. = \frac{38110}{17420} = 2.2$

Total Vertical Forces = 05500

02750

00800

01750

13200

03370

27370 K.G.

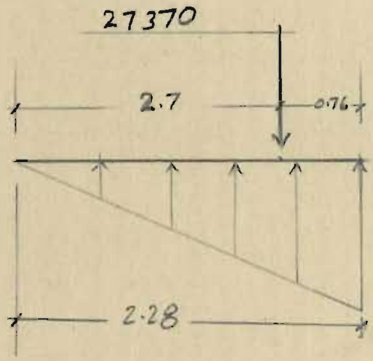
Coeff of friction = 0.5

27370 x 0.5 = 13685

$\frac{13685}{7780}$ = F.S. = 1.75 O.K.

Resultant M = 38110

$$\begin{array}{r} 38110 \\ \underline{17420} \\ 20690 \end{array}$$



$\frac{20690}{27370}$ = 0.76 m.

0.73 x 3 = 2.28 m

$\frac{w}{2}$ x 228 x 100 = 27370

w = $\frac{27370}{11400}$ = 2.4

Take section x - x

P = $\frac{1}{2}$ x 1600 x 5.5^2 x $\frac{456}{1.544}$

= 7150

7150 x 1.83 = 13100

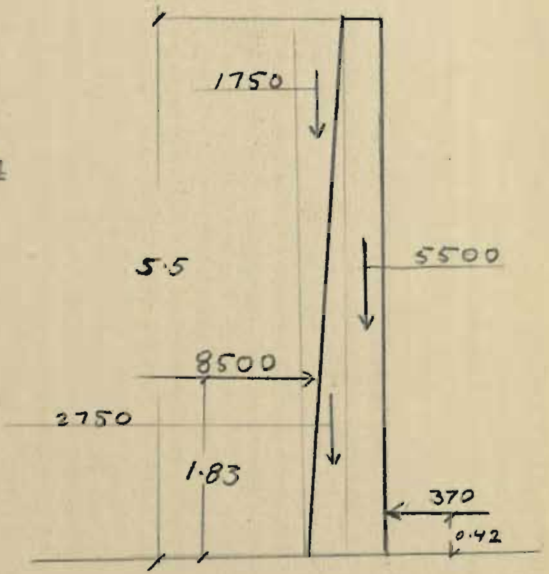
1750 x 0.67 = 01170

5500 x 0.20 = 01100

2750 x 0.53 = 01460

370 x 0.42 = 00155

3885



P on other side = $\frac{1}{2}$ x 1600 x 1.25^2 x $\frac{0.456}{1.544}$ = 370 K.Gs.

Resultant M = 13100

$$\frac{3885}{.9215\ K.G - m.}$$

$= 921500\ K.G\ c.m.$

$= 10.2\ d^2 \times 100\ ,\ d^2 = 905$

$d = 30\ c.m.$

Actual d is 80 - 10 = 70 c.m.

$$As = \frac{921,500}{1200 \times 0.87 \times 70} = 12.7$$

$1 - 14\ m.m\ \varnothing = 1.54\ c.m^2$

$$S = 100 \times \frac{1.54}{12.7} = 12.2\ c.m \quad \text{say } 10$$

Take a section at 3.5 m. lower than top.

$$\frac{3.5}{5.5} \times 0.40 = 0.25$$

$(0.4 \times 3.5)\ 2500 = 3500$

$$\frac{0.25 \times 3.5}{2} \times 2500 = 1100$$

$$\frac{0.25 \times 3.5}{2} \times 1600 = 700$$

$$P = \frac{1}{2} \times 1600 \times 3.5^2 \times \frac{0.456}{1.544}$$

$= 2860$

ΣM_s : 3500 x 0.2 = 700 K.G.- m.

1100(0.40+0.08) = 530 K.G - m.

700(0.40+0.16) = 390 K.G - m.

1620 K.G - m.

2860 x 1.17 = 3330

$\Sigma M = 3330 - 1620 = 1710$

$171000 = 10.2 \times 100\ d^2$

$d^2 = 168$, $d = 13$ c.m.

We have 55 c.m.

$$As = \frac{171000}{1200 \times 0.87 \times 55} = 2.9\ c.m^2$$

Use 5 - 10 m.m ∅/m all through.

Design of Cantilever at left side of base.: -

$$\frac{1.13 \times 100}{2} \times 108 = 6100$$

$w = 5.50 \times 1 \times 1.5 \times 1600 = 13200$

$2.28 - 1.20 = 1.08$ m.

$1.50 - 1.08 = 0.42$

$$P = \frac{1.08}{2.28} \times 2.4 = 1.13\ K.G/c.m^2$$

$$M = \left(\frac{1.13 \times 100}{2}\right) \times \frac{108^2}{2} = 33000$$

$13200 \times 75 = 985000$

$985000 - 33000 = 952000$ K.G - c.m.

$= 10.2 \times 10\ d^2$

$d^2 = 935$, $d = 30.6$

$$As = \frac{952000}{1200 \times 0.87 \times 31} = 29.5\ c.m^2/m.$$

Use 10 - 20 m.m ∅/m.

Check d for shear

$F = 13200 - 6100 = 7100$ K.G.

$7100 = 100 \times d \times 15$

$d = 4.75$ smaller than 31

Design of the short Cantilever:-

$$M = 2.4 \times 100 \times \frac{40^2}{2} = 192000$$

$$800 \times 20 \qquad = \underline{16000}$$

$$176000 = 10.2 \times 100\ d^2$$

$d = 13.1$ smaller than 31

$$As = \frac{176000}{1200 \times 0.87 \times 31}$$

Use 5 - 12 m.m Ø/m.

By shear

$$F = 2.4 \times 40 \times 100 = 9600$$

$$\underline{800}$$

$$8800$$

$$d = \frac{8800}{100 \times 15} = 6 \text{ c.m.}$$

C H A P T E R   S I X

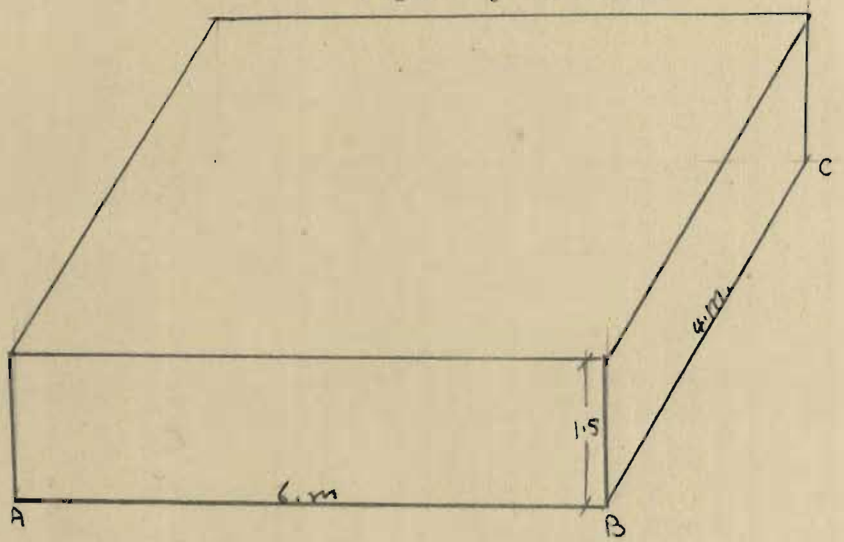
Water Tank

Above the Main Staircase of

the Administration Division.

DESIGN OF THE TANK: Dimensions 4 x 6 x 1.5

Capacity 36 cu.m.



Design part AB and consider that hoop steel carries all the tension.

Max P = 1.5 x 1 = 1.5 tons/m^2

Force on face BC at a height of 0.5 m =

1.5 x 4 x 0.5 = 3 Tons = 3000 K.Gs.

3000 ÷ 2 = 1500 K.G

$As = \frac{3000}{1200} = 2.5\ c.m^2$

Design face BC

P = 1.5 Tons/m^2

F = 1.5 (6 x 0.5) = 4.5

4.5 ÷ 2 = 2.25 Tons.

$\frac{2250}{1200} = 1.88\ c.m^2$

$$M = \frac{2000 \times 16}{10} = 320000 \text{ K.G - c.m.}$$
$$= 10.2 \times 100\, d^2$$

$$d^2 = 314\ ,\ d = 17.7$$
$$\underline{2.3}$$
$$20.0 \text{ c.m.} \quad \text{O.K.}$$

$$As = \frac{320000}{1200 \times 0.87 \times 17.5} = 17.5 \text{ c.m}^2$$

Use 1 - 14 m.m ∅ at 8 c.m. and

use 5 - 8 m.m ∅/m in the long direction.

<u>Beams in Long Direction:</u>

$$w = \frac{4 \times 1 \times 2000}{2} = 4000 \text{ K.G/m}$$

Tank wall = 0.25 x 1.5 x 2500 = 935 K.G/m

Weight of beam = 0.40 x 0.80 x 2500 = 800 K.G/m

Total w = 4000 + 935 + 800 = 5735 K.G/m.

$$M = \frac{5735 \times 36}{10} = 2,070,000 \text{ K.G - c.m.}$$
$$= 10.2 \times 40\, d^2$$

d = 71 + 9 = 80 c.m. O.K.

V = 5735 x 3 = 17205 K.G.

$$As = \frac{2070000}{1200 \times 0.87 \times 71} = 28.1 \text{ c.m}^2$$

Use 8 - 22 m.m ∅ bend 4 ∅

$$U = \frac{17205}{4 \times 6.9 \times 0.87 \times 71} = 6 \quad \text{O.K.}$$

<u>Bending:</u>

1st pair at 1.60 m.

2nd pair at 0.90 m.

Use 6 m.m ∅ at 7, 15, 25 c.m.

Use Transversal beams reinforced with 4 - 14 m.m ∅

Changes on Columns of the Staircase, 23, 24, 25, 26.

Total load = 103,180 K.Gs.

Col:

$$A = \frac{103180}{60} = 1720$$

Use 45 x 45

As = 10.1 $c.m^2$

Use 8 - 14 m.m ∅

Wt of Col = 0.45 x 0.45 x 10 x 2500 = 5050

FOOTING:

$$\frac{108230}{2.5} = 43300$$

Use 210 x 210

$$M = 2.5 \times 210 \times \frac{(82.5)^2}{2} \times 0.8$$

$$= 1,420,000 \text{ K.G - c.m.}$$

$$d = \frac{108230}{4 \times 45 \times 15} = 40.5 \text{ c.m.} \quad \text{say 50 c.m.}$$

$$As = \frac{1420000}{1200 \times 0.87 \times 40.5} = 33.5 \text{ c.m}^2$$

Use 17 - 16 m.m ∅

Bond 5 ∅

Reduce section to 35 x 35 in the first floor

As = 6.15 $c.m^2$

Use 6 - 12 m.m ∅

APPENDIX A

When one considers the plan of the Chapel he will realize that Beams B1, B2, B3 and B4 are independent beams. The roof Beams on the sides are only carrying themselves, thus to design them as continuous beams is useless. The only structural element that could be designed as continuous for the purpose of economy is the Slab of the Chapel.

I am going to show in the following analysis that in this special case (The Chapel), the design I produced is as economical.

CHAPEL SLAB ANALYSIS BY THE PRINCIPAL OF HARDY CROSS

A	B		C		D	
100	41	59	63	37	100	Distribution Factor/Fixed end Moment
-1400	-1400	-690	-690	-2030	-2030	
+1400	+ 290	-420	-845	+ 495	+2030	
-145	-700	+422	+210	-1015	-247	
+145	+460	-660	-770	+ 455	+247	
-230	- 72	+385	+330	-123	-227	
+230	+187	-270	-285	+168	+227	
-93	-115	+142	+135	-113	-84	
+93	+105	-152	-156	+ 92	+84	
-52	-46	+78	+76	-42	-46	
+52	+51	-73	-74	+44	+46	
-25	-26	+37	+36	-23	-22	
+25	+26	-37	-37	+22	+22	
-13	-12	+18	+18	-11	-11	
+13	+12	-18	-18	+11	+11	
-6	-6	+9	+9	-5	-5	
+6	+6	-9	-9	+5	+5	
-3	-3	+4	+4	-2	-2	
+3	+3	-4	-4	+2	+2	
-1	-1	+2	+2	-1	-1	
+1	+1	-2	-2	+1	+1	
0	0	-1	+1	0	0	
0	0	+1	-1	0	0	
0	1240		2070		0	

$$Mb = -575 \times 5.4 \times 2.7 + 5.4\ Ra = -1240$$

$$Ra = \frac{8400 - 1240}{5.4} = \frac{7160}{5.4} = 1330 \text{ K.gs.}$$

$$Mc = -2070 = 9.2 \times 1330 + 3.8\ Rb - 9.2 \times 575 \times 4.6$$

$$Rb = \frac{24800 - 12200 - 2070}{3.8} = 2760 \text{ K.G}$$

$$Mc = -2070 = 6.5\ Rd - 575 \times \frac{6.5^2}{2}$$

$$Rd = \frac{12170 - 2070}{6.5} = \frac{10100}{6.5} = 1560$$

$$Mb = -1240 = 1560 \times 10.3 + 3.8\ Rc - 575 \times 16.3^2 = -1240$$

$$Rc = \frac{30500 - 16000 - 1240}{3.8} = \frac{13260}{3.8} = 3500$$

$$Ra + Rb + Rc + Rd = 9150$$

$$575 \times 15.7 = 9050 \quad \text{O.K.}$$

$$Mc = \frac{1}{2} fckjbd^2 = Kbd^2 = 10.9bd^2$$

$$2070 = 10.9 \times 1 \times d^2$$

$$d^2 = \frac{2070}{10.9} \quad , \; d = \left(\frac{2070}{10.9}\right)^{\frac{1}{2}} = 13.8$$

Just in agreement with the d I have chosen in my design. Moreover the reaction just calculated agrees with the end shear I have used in the design of the roof beams in question

APPENDIX B

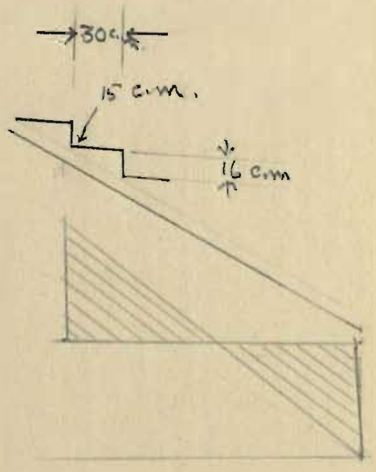
This part consists of the structural design needed to alter the architectural design of the main staircase into a hanging one.

This structural analysis divides the staircase into 4 structural units as shown in the attached drawings. The units are:-

1. Stairs
2. Cantilevered landing
3. Cantilever beam
4. Principal beam.

DESIGN OF 1. STAIRS:

Loads:-



Concrete: 0.15 x 2500 = 375 K.G/m^2

Stairs : 0.08 x 2500 = 200 K.G/m^2

Marble : 0.05 x 2000 = 100 K.G/m^2

Live load: 500 K.G/m^2

Total w = 1175 K.G/m^2

$$M = \frac{1175 \times 4.2^2}{10} = 2065 \text{ K.G.m.}$$

d = 13.5

As = 15.2

Use 10 - 14 m.m ∅/m.

DESIGN 2 - CANTILEVERED LANDING:-

load = 1000 K.G/m^2

$$M = \frac{1000 \times 1.5^2}{2} = 1120 \text{ K.G - m.}$$

d = 18 c.m.

As = 5.8 c.m^2

Use 8 - 10 m.m Ø/m.

<u>DESIGN 3 - CANTILEVER BEAM:</u>

$1.5 \times 1000 = 1500$ K.G/m

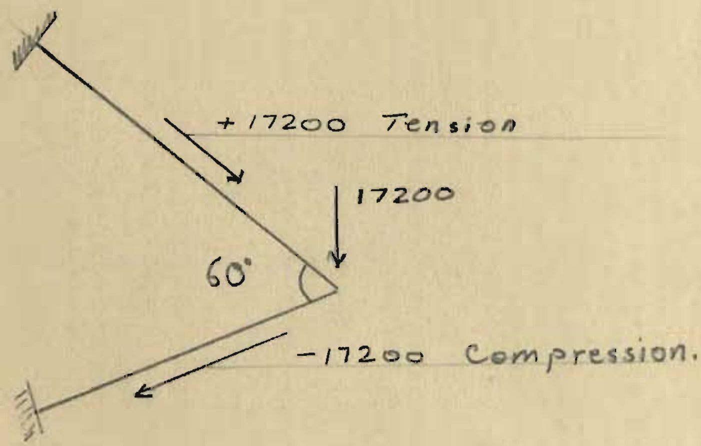
$\frac{4.2 \times 1175}{2} = 2500$ K.G/m

wt of beam = 300 K.G/m.

Total = 4300 K.G/m.

$$M = \frac{4300 \times 4}{2} = 8600 \text{ K.G - m.}$$

This beam should be designed for compression



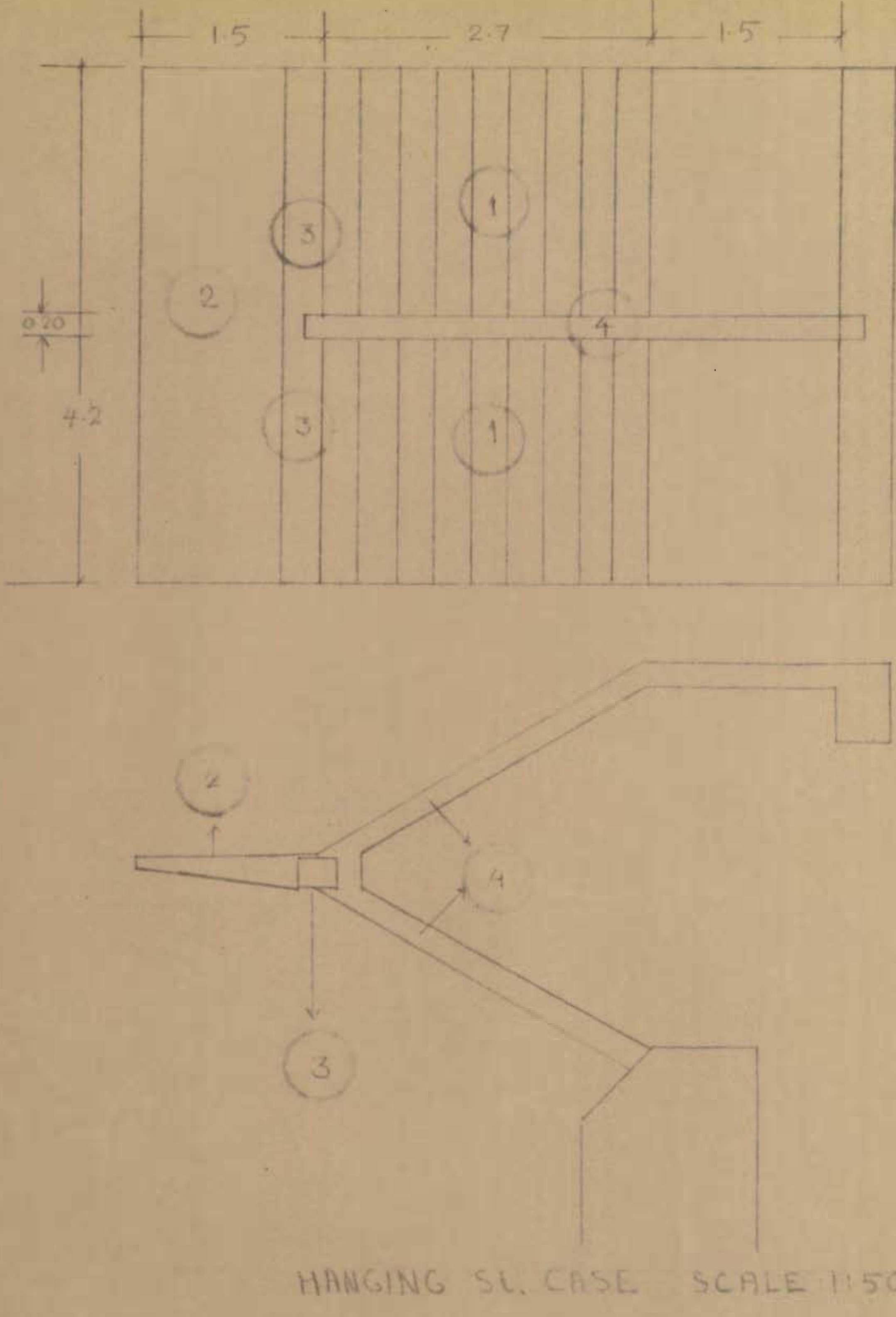
Steel of Compression beam = 20 x 40 x 0.01 = 8 c.m.^{2}

Use 6 - 14 m.m Ø

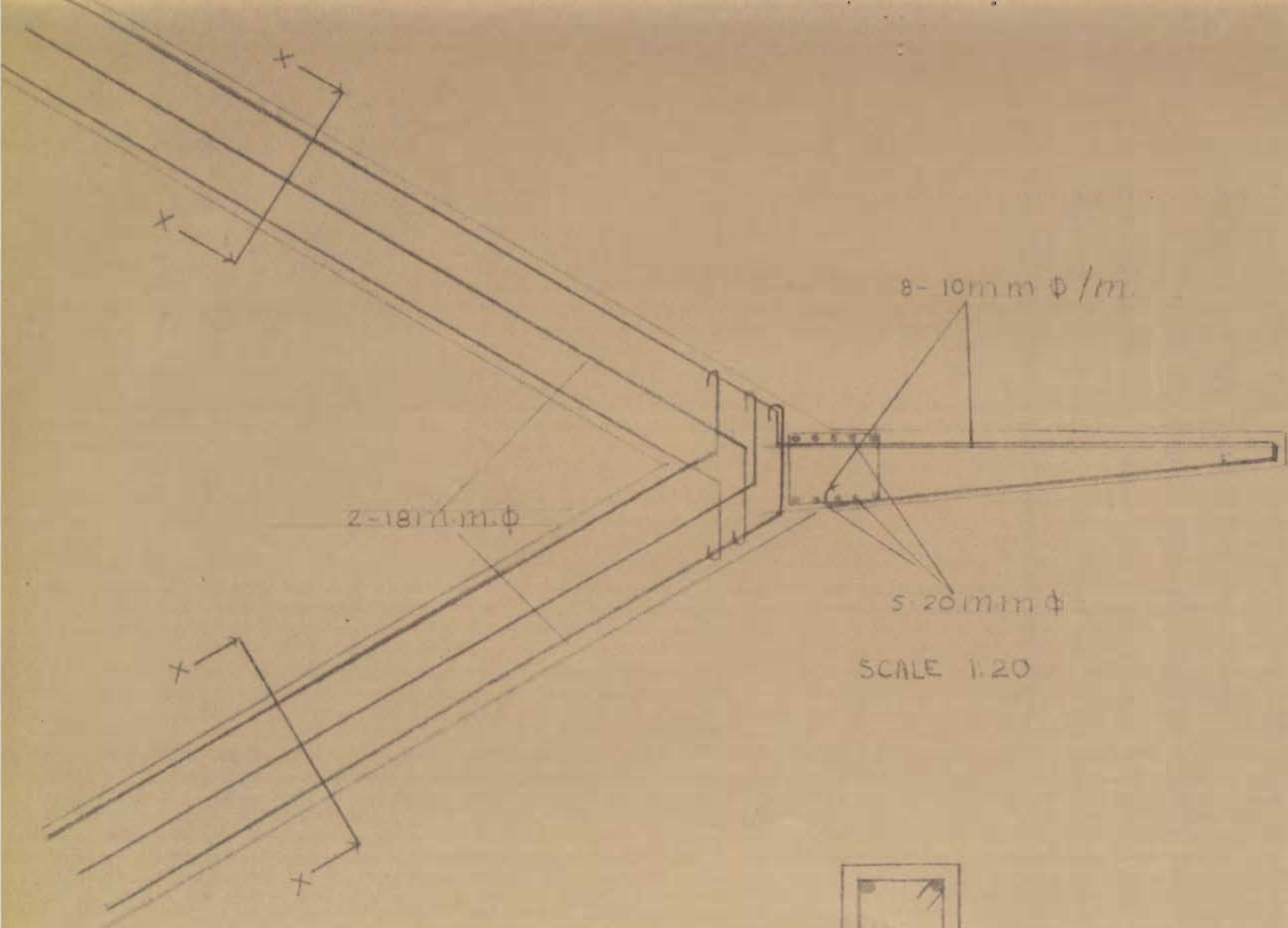
Steel of Tension beam = 17200 ÷ 1200 = 14.3 c.m^2

Use 6 - 18 m.m Ø/m.

<u>Caution:</u> This principal beam is indipendent of the stair unit, so in construction, and for construction purposes, sufficient insulation should be provided.



HANGING ST. CASE SCALE 1:50



SCALE 1:20

SECTION X-X SCALE 1:10

APPENDIX C

For the sake of more practice and research on this subject, and for the purpose of takling the problem more scientifically, this Index was added.

It is mainly the application of the Hardy-Cross Principal on two frames that have been chosen from the main body of the thesis. The first is from the Auditorium and the second is from the Classroom devision.

THE FIRST FRAME:

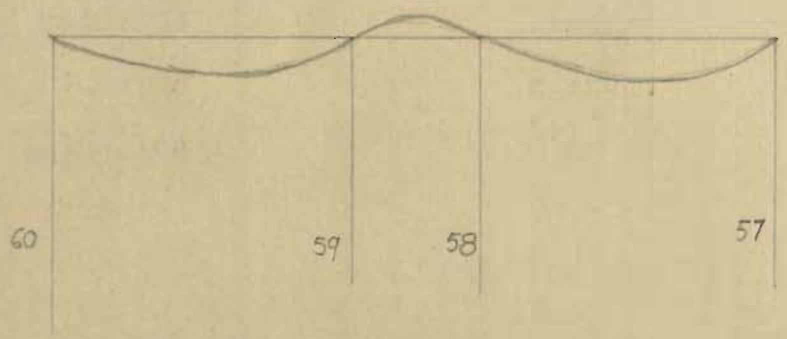
Roof Beam (A-3) of the Auditorium is resting on Columns (9) and (12).

The intention of the following analysis is to have these Columns carry part of the moment on the beam. The distribution analysis is attached herewith.

THE SECOND FRAME:

Two R. Beams (D-2) are resting on Columns (57), (58) (59) and (60). Assuming a third beam between (58) and (59) having the same section of the other beams and resting on Columns (58) and (59).

On loading the slabs carried by those beams the worst condition occurs when the classrooms on both sides of the corridor are fully loaded with students and the corridor has no live load at all. The bending occuring takes the following shape:

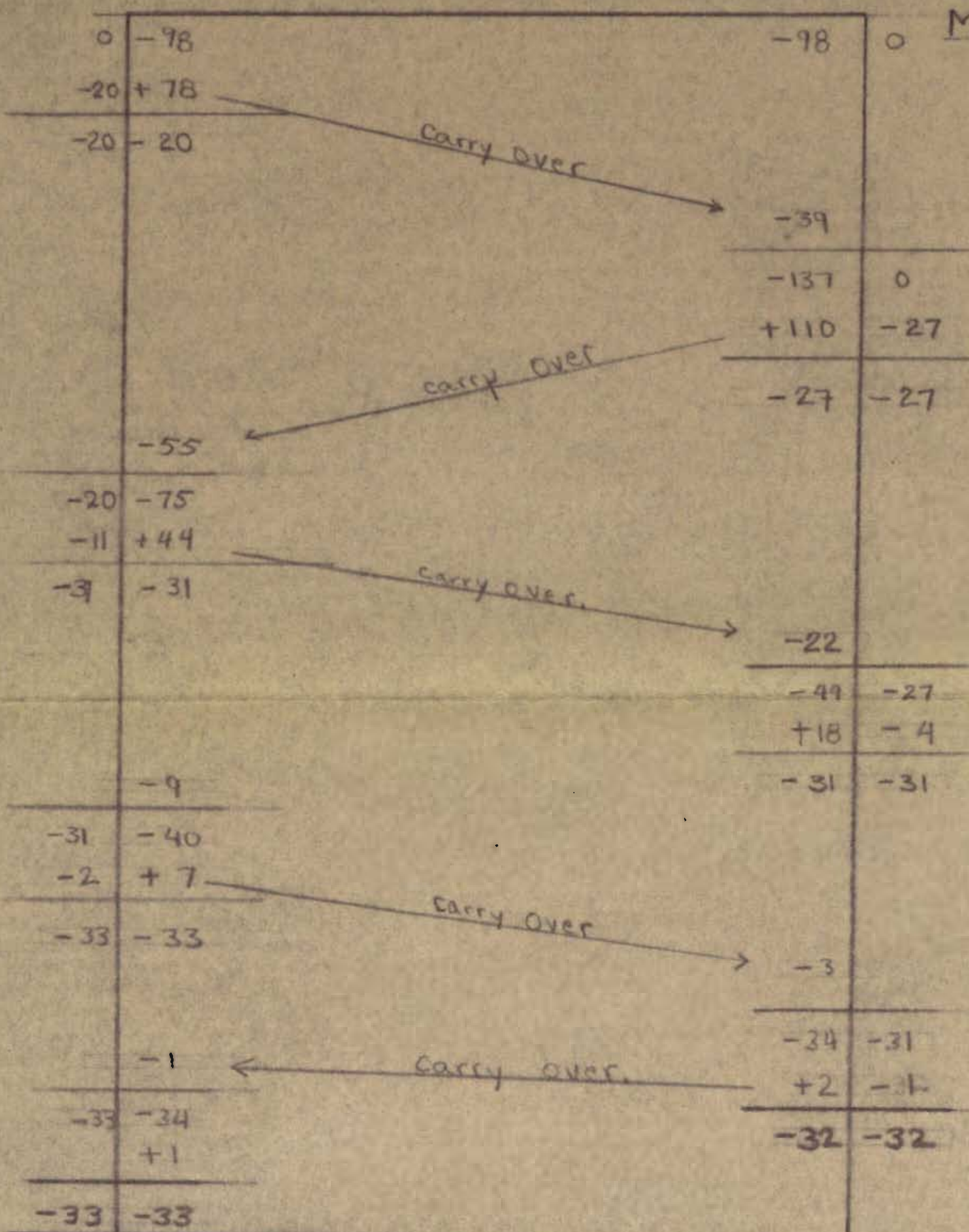


The moment distribution for this second frame is attached.

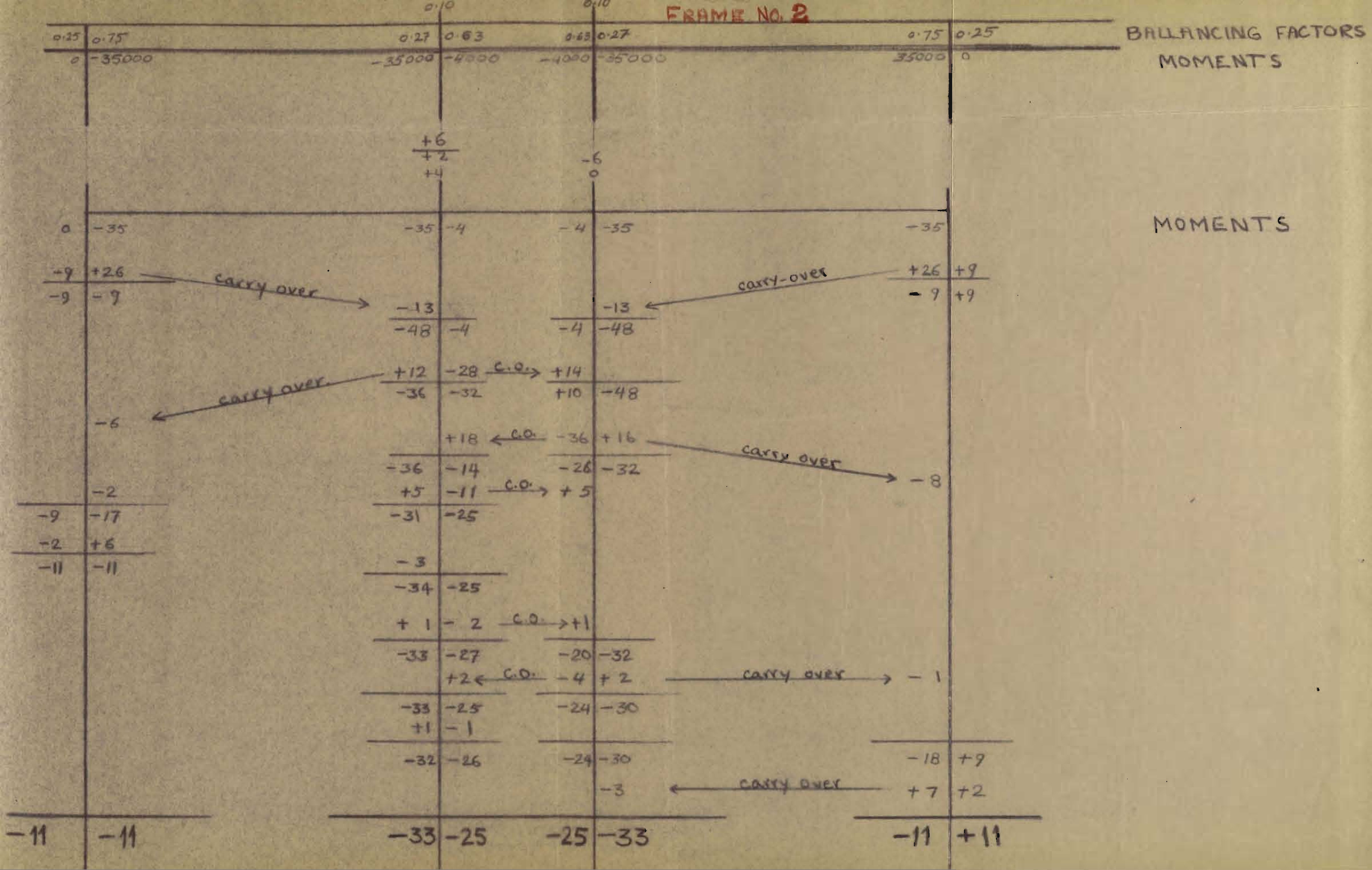
1st Frame

BALLANCING FACTORS

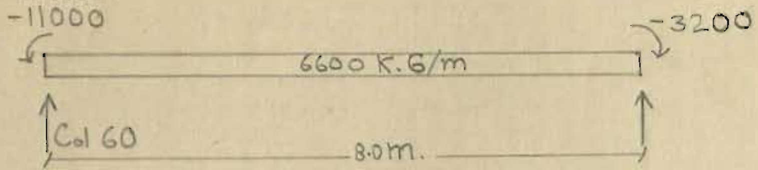
MOMENTS.



FRAME No. 2


BALLANCING FACTORS
MOMENTS
0.25 0.75
0.27 0.63
0.63 0.27
0.75 0.25
0.10
0.10
0 -35000
-35000 -4000
-4000 -35000
35000 0
MOMENTS
0 -35
-35 -4
-4 -35
-35
carry over
carry-over
c.o.
carry over
-11 -11
-33 -25
-25 -33
-11 +11

Beam (D-2)



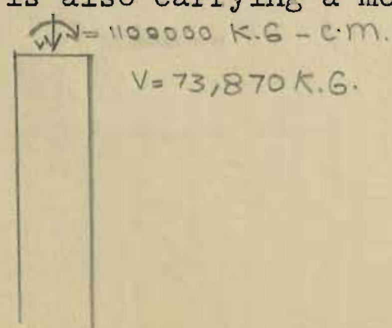
The sum of moments about B are:

	- 32,000
Ra x 8=	- 8 Ra
6600 x 32 =	+ 210,000
	+ 11,000
Therefore Ra =	23,700

<u>Vertical Loads on Column (60)</u>

G. Beams	=	1 - 06000	K.G.
		2 - 09000	K.G.
		3 - 09000	K.G.
R. Beams	=	1 - 12600	K.G.
		2 - 12600	K.G.
Wt. of Col.	=	00970	K.G.
Reaction from Frame	=	23700	K.G.
Total	=	73870	K.G.

The Column is also carrying a moment of 11000 K.G - m.



Now $\frac{Ma}{2I} + \frac{P}{a^2} = fc$

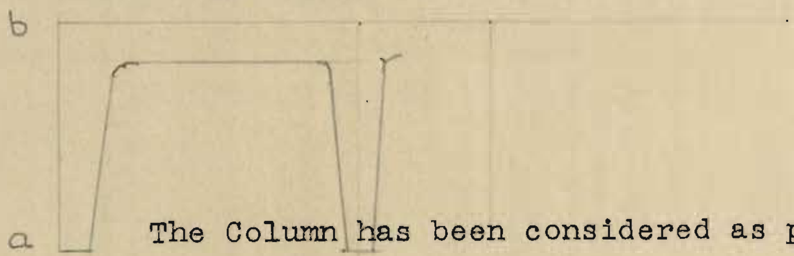
$$\frac{1100000 \times 40}{2 \times 920000} + \frac{93870}{40 \times 65} = 53 \text{ smaller than } 56$$

Permisable fc being 56 ; this section is O.K.

I have calculated one Column only as an example.

Because the Column is taking over part of the moment it will increase in section as shown.

The following architectural treatment should be useful in reducing the total volume of the Columns.



The Column has been considered as pin point so the moment will increase directly with the height, beginning at (a) with zero. Therefore, the section of the column at the base is determined only, by the vertical load on the column, it carries no moment. The higher we go from (a) to (b) the moment increases until it reaches maximum at (b), as shown in the figure.

The same principal and treatment has been applied for the Lecture Hall of the Engineering Building.

T H E   E N D