MANUAL
ON
PLANNING & DESIGN
OF
REINFORCED CONCRETE
MULTISTOREYED
BUILDINGS



CENTRAL DESIGNS
ORGANISATION

CENTRAL PUBLIC
WORKS DEPARTMENT

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MANUAL ON PLANNING & DESIGN OF REINFORCED CONCRETE MULTISTOREYED BUILDINGS

Volume II

FOREWORD

I am glad to know that the first volume of the Manual on "Planning & Design of Multistoreyed buildings" dealing with the aspects of planning and structural analysis has been very well-received by the design engineers. This volume dealing with the design and detailing aspects will make the manual a comprehensive handbook. I am particularly happy that the Computer Cell of the Central Designs Organisation has been able to evolve in a very short time computer programmes required for evolving various design charts and tables which, I am sure, will add to the usefulness of this Manual.

I would like all the Engineers in C. P. W. D. connected with design of buildidgs to make full use of this manual.

NEW DELHI
the 12th May, 1977

V. R. VAISH
Engineer-in-Chief

PREFACE

The first volume of the manual on 'Planning & Design of Multistoreyed Buildings with preliminary design and structural analysis was published last year. This volume deals with the design and detailing aspects.

Structural design is an innovating process. Though the codes and textbooks deal with basic principles, there are many gaps which have to be filled by the structural designer himself based on his experince and ingenuity. The CPWD have designed and constructed a number of multi-storeyed buildings all over the country and the expertise accumulated by the department in the course of years is sought to be reflected in the manual which outlines the procedure of design of various structural elements, illustrating the same with the worked-out examples. During the course of evolving structural designs of various buildings, a number of design aids (tables and charts) have also been prepared. These have been added at appropriate places in the manual.

The Computer Cell of the Central Designs Organsation has done valuable work in evolving programmes for structural analysis and design. Some of the tables and charts have been prepared on the basis of the computer programme evolved by this cell. These are calculated to meet the requirments of the practical designer.

Specimen drawings appended to this Manual indicate the various structural details that are to be exhibited on working drawings.

It is hoped that this Manual, with its worked out examples, design charts, tables and specimen drawings would serve as a practical handbook for structural designers and fill a long-felt need. S/Shri SC Gupta, PK Ratho, Deepak Narayan, Bapu Somasekhara, SS Juneja, Jose Kurian, RC Grover, Engineers of the CDO have enthusiastically participated in the preparation of this Manual. My thanks are due to all of them. My special thanks are to Shri PK Ratho for helping me in drafting the Manual and to Shri Jose Kurian for preparing the computer programmes and analysis in such a short time.

Any Suggestions for improvement of the Manual would be gratefully received.

NEW DELHI

T. S. VEDAGIRI
Chief Engineer (Designs)

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10. Rectangular portion high tensile steel reinforcement of opposite edges $\frac{d_1}{d} = 0.80$

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12. Do. 0.90

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14. Rectangular section high tensile steel reinforcement $\frac{As}{4}$ on each face $\frac{d_1}{d} = 0.80$

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PLATES

1. Details of footings reinforcement . . . . . .

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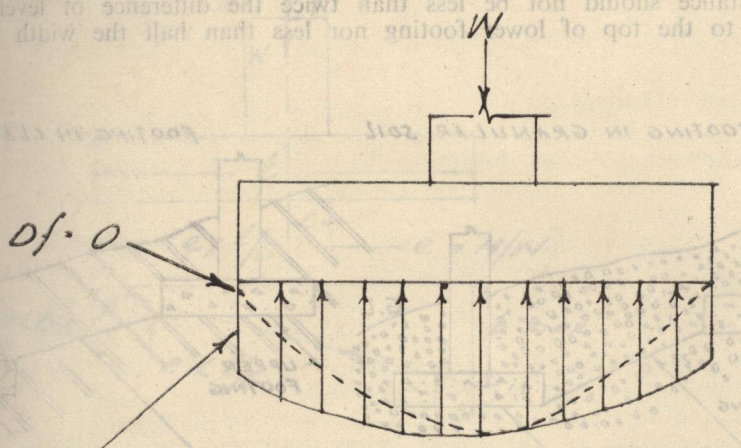
3. Details of slab reinforcement . . . . . .

4. Details of beam reinforcement . . . . . .

CHAPTER 6
DESIGN OF FOUNDATIONS

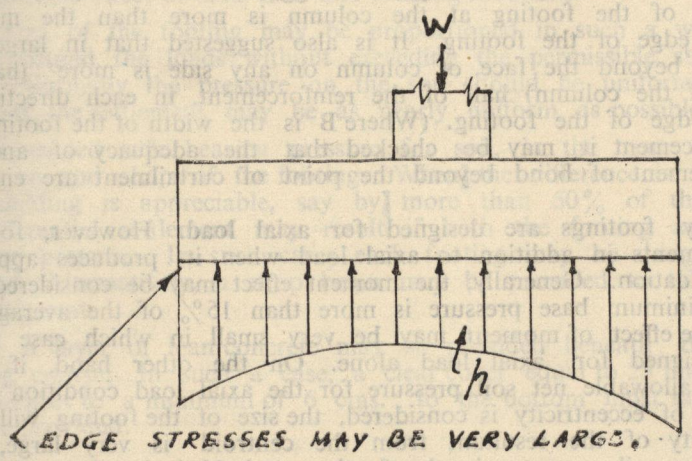
6·1 Shallow foundations

In foundation, the load from a column or a wall is spread over a large area by providing a footing so that the pressure on the soil is limited to the recommended safe bearing pressure for the soil. The actual stress distribution on the soil depends on the properties of the soil beneath the footing and the rigidity of the footing. The stress distribution underneath a flexible or rigid footing on cohesionless material is as shown in Fig. 6·1. The stress distribution due to a rigid footing resting on a soil possessing both cohesion and angle of internal friction is as shown in Fig. 6·2. For design purposes, the pressure due to a concentric load is assumed to be uniformly distributed over the entire area of the footing if the load is applied at the centre of the gravity of the bearing area. In order to avoid tilting of footings, the pressure under the footing should, as far as possible, be symmetrically distributed. This principle is equally applicable for combined footings and footings with strap beams.



EDGE STRESS DEPENDS ON THE DEPTH OF FOOTING Df
PROBABLE PRESSURE DISTRIBUTION BENEATH A RIGID FOOTING
[ON A LOOSE COHESIONLESS SOIL]

FIG. 6·1



EDGE STRESSES MAY BE VERY LARGE.
(b) GENERALLY FOR COHESIVE SOIL.

FIG. 6.2

6·1·1 Foundations of column footings may be designed to have adequate stiffness. Thus it is suggested that the design of the column footings based on elastic theory may be adopted. It is desirable that the length of the footing beyond the face of the column is limited to three times the depth of the footing. Where, it is not possible to adhere to this limit and instead a thin and long footing is proposed, the assumption of uniform bearing pressure may be verified with reference to the sub-grade reaction of the soils. Alternatively the design may be based on the theory of beams on elastic foundation.

6·1·2 The recommended safe bearing capacity is based on the shear failure criteria or the settlement consideration, whichever gives the lower value. In both the cases the recommended value is for the net increase in vertical pressure which may be permitted on the soil at the particular reduced level and location.

6·1·3 When two footings are in close proximity, the effect of interference may be examined. In addition, when either of the footings is at a higher level it is to be ensured that for granular soils, the clear distance between the two footings is atleast equal to twice the difference of the foundation levels and is not less than half the width of the footing. In clayey soils the clear distance should not be less than twice the difference of level from the bottom of higher footing to the top of lower footing nor less than half the width of footing.

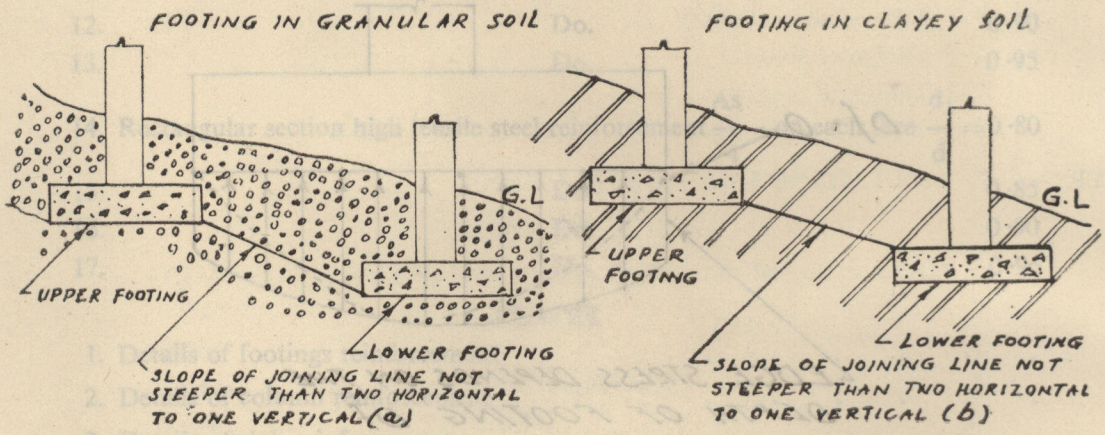


FIG. 6.3

6·2 Spread Footings :

6·2·1 Due to the resulting economy, it is suggested that sloped footings may be provided where the thickness of the footing at the column is more than the minimum thickness to be provided at the edge of the footing. It is also suggested that in large sized footings, (i.e. when the projection beyond the face of column on any side is more than twice the effective depth at the face of the column) half of the reinforcement, in each direction, may be curtailed at 0·1B from the edge of the footing. (Where B is the width of the footing). However, before curtailing the reinforcement it may be checked that the adequacy of anchor length for the bar and the requirement of bond beyond the point of curtailment are ensured.

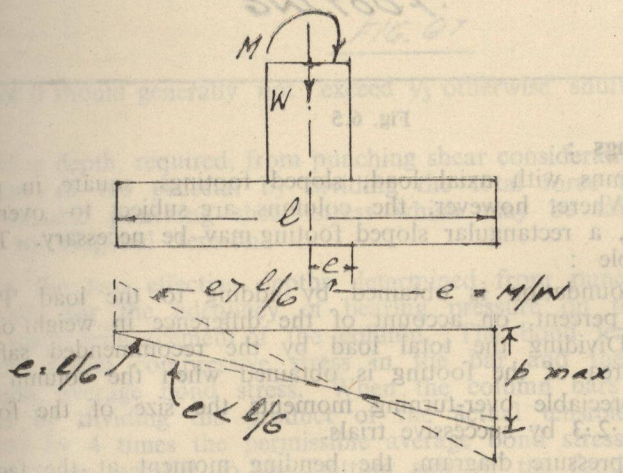
6·2·2 Normally, footings are designed for axial load. However, footings may have to be designed for moments in addition to axial load when it produces appreciable variation of pressure on the foundation. Generally, the moment effect may be considered when the difference of maximum and minimum base pressure is more than 15% of the average. If this difference is less than 15% the effect of moment may be very small in which case it may be neglected and the footing designed for axial load alone. On the other hand, if the maximum base pressure exceeds the allowable net soil pressure for the axial load condition alone by more than 25% when the effect of eccentricity is considered, the size of the footing will need reproportioning. If the eccentricity of the resultant from the centroid is very large, negative pressures may result. Since the soil cannot grip the footing to hold it in place, the area of the footing, which is in tension, is considered ineffective, and the pressure distribution recalculated. (Refer example).

6·2·3 For footings with moments or eccentricity about both axes.

$$p = \frac{W}{A} \pm \frac{M_y X}{I_y} \pm \frac{M_x Y}{I_x}$$

where

p = the bearing pressure at the point under calculation.
W = vertical concentric load on footing.
A = base area of footing.
I_x = moment of inertia of footing about X axis.
I_y = moment of inertia of footing about Y axis.
X = distance from Y axis to the point at which the stress is calculated.
Y = distance from X axis to the point at which the stress is calculated.



COLUMN CENTRALLY LOCATED ON FOOTING BUT WITH OVERTURNING MOMENT

FIG. 6.4

$p_{max} \leq 1.25 \times$ permissible bearing pressure on foundation for concentric load and with no moments.

The permissible pressure may be exceeded at the edges of the footing by 25 per cent when footings are designed for the combined effect of axial load and bending moments.

6·2·4 The base area of the footing may be proportioned in such a way that the footing area is sufficient to withstand the loads without exceeding the permissible stresses and permissible net soil pressure. Normally the pressure on the soil should be uniform under all footings of the building so that the settlement may be as nearly uniform as possible.

6·2·5 The recommended safe bearing pressure for soil, in the case of spread footings, is dependent on the size and shape of the footing. When the difference in the width of adjacent footings in a building is appreciable, say by more than 50% of the width of smaller footing, significant differential settlement may result if both the footing are designed for the same allowable bearing pressure. In such cases each footing may be designed for the appropriate bearing capacity. Alternatively, a strap beam may be provided to take up the moments due to differential settlement.

6·2·6 Generally, a layer of lean concrete may be provided beneath the structural footing to serve as a levelling course. In such a case, a clear cover equal to 25 mm plus the diameter of the reinforcement subject to a minimum of 6 cms. to the bottom most layer of reinforcement of the footing, is to be provided.

6.3 **Pedestal :**

Every column may be provided with a pedestal of minimum 15 cms. depth and a projection alround of 15 cms beyond the column. The height of pedestal may be calculated from

the requirement of vertical bond length for the main reinforcement of the column as per para 6·4 (v).

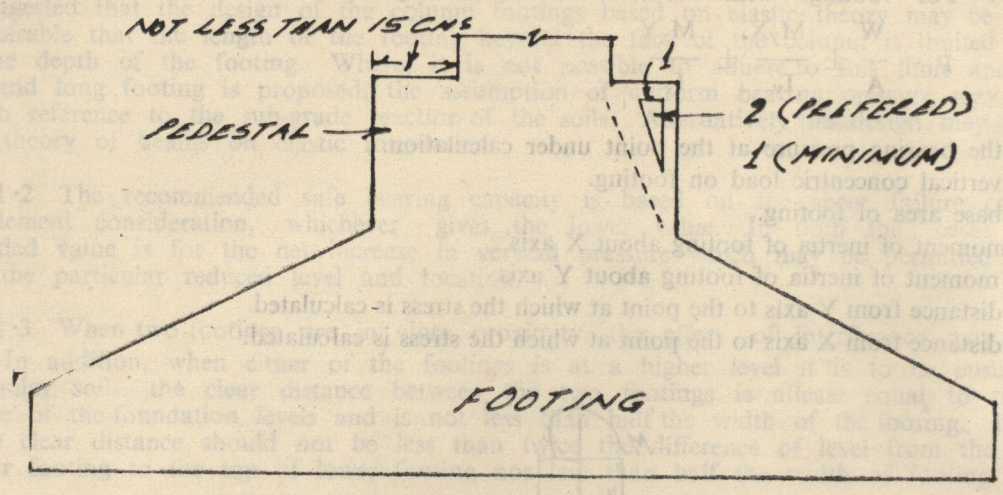


Fig. 6.5

6·4 Design of Isolated Footings :

For normal sized columns with axial load, sloped footings, square in plan, is obviously the most suitable choice. Where, however, the columns are subject to over-turning moments or site conditions so require, a rectangular sloped footing may be necessary. The following procedure of design is applicable :

(i) The load on the foundation is obtained by adding to the load 'P' on the column a certain percentage, say 5 percent, on account of the difference in weight of the footing and equivalent mass of earth. Dividing the total load by the recommended safe bearing pressure on the soil, the required area of the footing is obtained when the column is axially loaded.

For columns with appreciable over-turning moments the size of the footing is obtained from the equation in para 6·2·3 by successive trials.

(ii) After drawing the pressure diagram, the bending moment at the face of the pedestal is computed as the moment of the forces acting over the entire area of the footing ABCD as shown in Fig. 6·6.

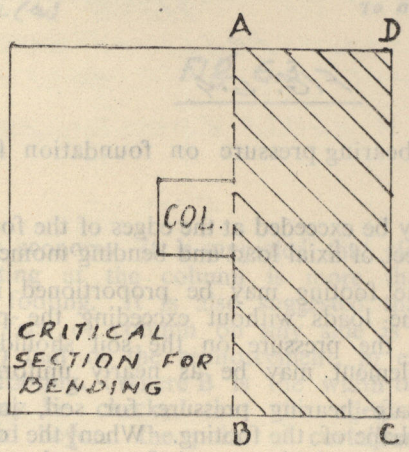


Fig. 6.6

(iii) The required effective depth 'd' is calculated from the formula $M=Rbd^2$.

Where 'b' is the effective width of section under bending compression which may be taken as equal to the sum of the width of the column/pedestal plus effective depth of the footing at the face of the column/pedestal.

M =Bending moment on the section under consideration.

R $=1/2 x f_c x k x j$.

f_c =Permissible stress in compression in concrete in bending.

k =Ratio of depth of neutral axis to effective depth.

j =Ratio of lever arm to effective depth (calculated with reference to the triangular distribution of stress over the trapezoidal area in compression).

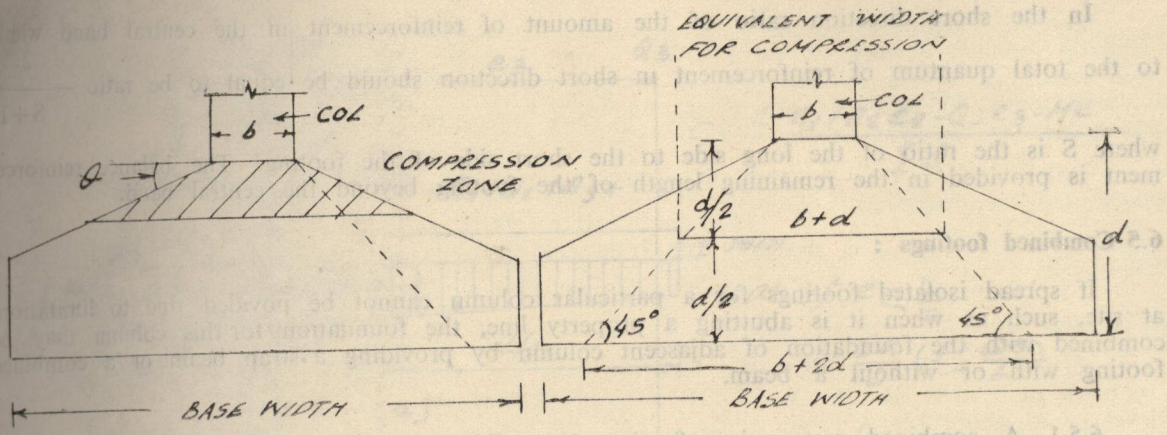


FIG. 6.7

The tangent of θ should generally not exceed ⅓ otherwise shuttering for the top slope will be required.

(iv) The effective depth, required, from punching shear consideration, is determined at a distance d/2 from the periphery of the pedestal by dividing the total force beyond the periphery at this point by the permissible punching shear stress which may be taken as twice the permissible shear stress due to diagonal tension.

(v) Higher of the two effective depths determined from punching and bending consideration is then checked for the adequacy of bearing pressure on the footing and the adequacy of bond for the main reinforcement of the column. The bond length, is obtained by dividing the product of the actual compressive stress in the bar and the diameter of the bar by 5 times the permissible average bond stress. When the column bars are under tension the bond length is obtained by dividing the product of the actual tensible stress in the bar and the diameter of the bar by 4 times the permissible average bond stress. To provide this length the bars are bent and taken parallel to the bottom side of the footing. In order to avoid punching of the concrete due to high compressive stress of the bar at the bent, it is suggested that the vertical bond length should atleast be equal to the product of the actual compressive stress in the bar and the diameter of the bar divided by 5 times the permissible local bond stress. Where the requisite depth of footing on local bond stress consideration is not provided the height of the pedestal may be increased to cover the additional bond length.

(vi) The effective depth required from the consideration of shear as a measure of diagonal tension is checked at a vertical section located at a distance from the face of the column equal to the effective depth of the footing.

$$q = \frac{V}{(b+2d)\times j\times d_1}$$

where q=shear stress at the section due to the shear force V.

(vii) On the basis of the finally adopted effective depth, the area of the reinforcement required is calculated from the formula :

$$As = \frac{M}{F_s x j x d}$$

Where A_s=Area of reinforcement.

F_s=Permissible stress in steel reinforcement.

$j\times d_1$=Lever arm

The reinforcement thus provided is checked for the bond stress by considering the critical section for shear at the face of the column/pedestal, the shear force being equal to the total reaction considered for bending moment purpose.

(viii) In two way reinforced rectangular footing the reinforcement in the long direction is distributed uniformly across the full width of the footing. In the short direction the total reinforcement required is provided in three bands. The spacing is closer in the central band width which is equal to the width of the footing.

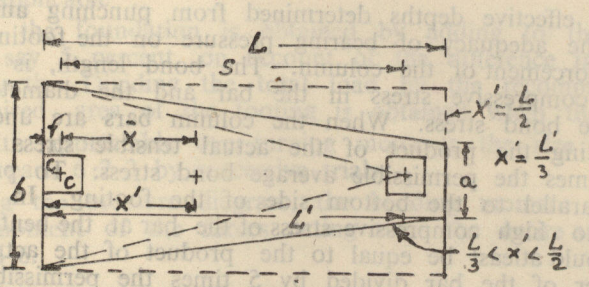
In the short direction ratio of the amount of reinforcement in the central band width to the total quantum of reinforcement in short direction should be equal to be ratio $\frac{2}{S+1}$

where S is the ratio of the long side to the short side of the footing. The balance reinforcement is provided in the remaining length of the footing beyond the central band.

6.5 Combined footings :

If spread isolated footings for a particular column cannot be povided due to limitations at site, such as when it is abutting a property line, the foundation for this column may be combined with the foundation of adjascent column by providing a strap beam or a combined footing with or without a beam.

6.5.1 A combined rectangular footing is possible when the resultant soil pressure lies at or beyond the mid-point between the columns measured from the column abutting property line. If however the resultant lies within the limit of L/2 and L/3 where L is distance between the columns) then a trapezoidal combined footing may be provided. A strap footing may be used where the distance between columns is so great that a combined or trapezoidal footing becomes quite narrow resulting in high bending moments.



CONDITIONS FOR VALID TRAPEZOIDAL FOOTING SOLUTION

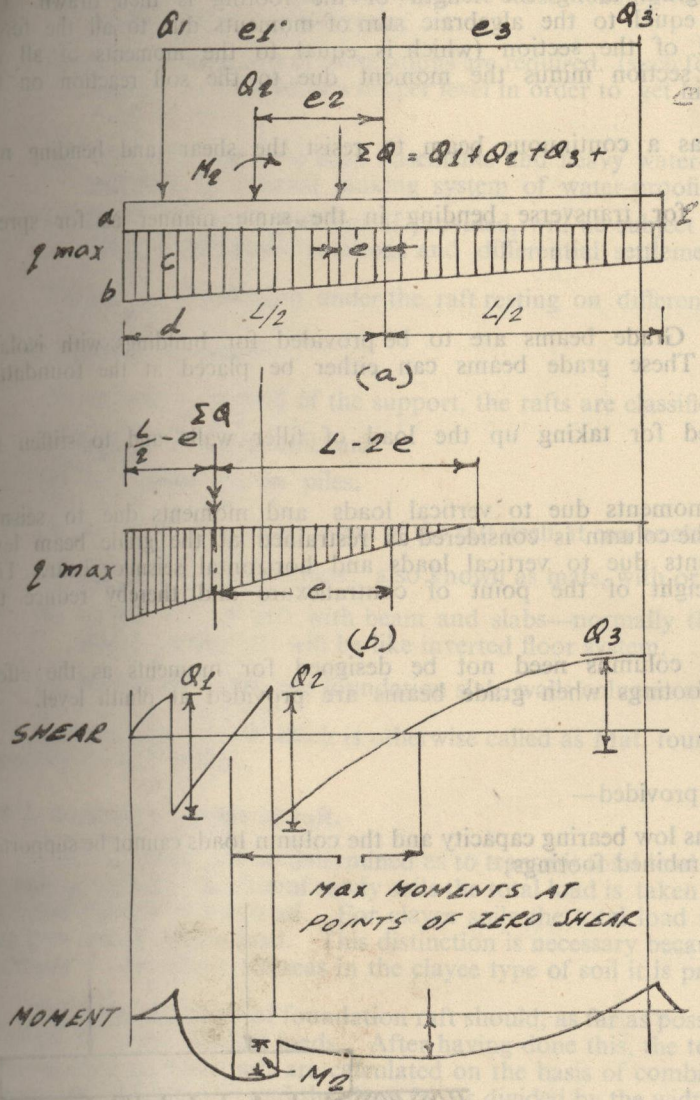
FIG. 6.8

6.5.2 A combined footing is also necessary when the footings of two adjascent columns overlap. In this case if the property line limit does not exist, the length and breadth of combined footing is so chosen as to have minimum design moment for the footing. When the distance between the columns is not considerable, (say 6 times the thickness of the footing) a footing without beam is suggested. If however the distance is considerable, a single beam of uniform depth connecting the two columns and extending from one end of the footing to the other, may be provided. In this case the size of the footing may be so chosen as to make the design moment in the span between the columns equal to the higher of the moments due to two cantilevers. When the distance between the columns is considerable the maximum heaving moment in the central portion may be considerably greater than the moments at the faces of the columns. In such a case the central portion may be economically designed as a 'T' beam section with the followihg limitations :

The flange thickness shall be not less than one half the width of the web and the total flange width shall be not more than 4 times the width of the web.

6·6 Design of combined footings by conventional (rigid) method:

The conventional design is based on the assumptions that the footing is infinitely stiff and that the base pressure of the soil has a straightline distribution. For the design of a combined footing supporting several loads, the following procedure is adopted.



$e = \dfrac{Q_1e_1+Q_2e_2-Q_3e_3-M_2}{\Sigma Q}$

(a) if $e \leq \frac{L}{6}$

$q = \dfrac{\Sigma Q}{L}\left(1 \pm \dfrac{6e}{L}\right)$

(b) if $e > \frac{L}{6}$

$q_{max} = \dfrac{2\Sigma Q}{3\left(\frac{L}{2}-e\right)}$

$q_{min} = 0$

NOTE

Pressure q max, q min, shear and moment are shown per linear metre of footing

FIG. 6.9

(i) From the magnitude of loads and positons of columns, the position of the resultant is determined.

(ii) Wherever possible the length and position of combined footings is so adopted as to make the centre of gravity of the combined footing coincide with the line of action of the resultant of the column loads. Where this is not possible on account of restrictions at site, the eccentricity of the resultant load from the centre of gravity of the footing is determined from the formula,

$$e = \frac{Q_1e_1+Q_2e_2-Q_3e_3-M_2}{\Sigma Q}$$

(iii) If the eccentricity is less than or equal to 1/6th of the length of the footing, the maximum and minimum base pressures per metre length are calculated from the formula

$$q = \frac{\Sigma Q}{L}\left(1 \pm \frac{6e}{L}\right)$$

(iv) The shear force diagram along the length of the footing is drawn. The shear at any section is equal to the algebraic sum of all vertical forces to the left or right of the section (which is equal to the difference of the column loads acting downwards and the total reaction due to the soil pressure acting upwards).

(v) The bending moment diagram along the length of the footing is then drawn. The bending moment at any section is equal to the algebraic sum of moments due to all the forces and reactions to the left or right of the section (which is equal to the moments of all the column loads on one side of the section minus the moment due to the soil reaction on the same side of the section).

(vi) The footing is designed as a continuous beam to resist the shear and bending moments thus calculated.

(vii) The footing is designed for transverse bending in the same manner as for spread footings.

6.7 **Grade beams:**

The I.S. Code stipulates that Grade beams are to be provided for buildings with isolated footings in Zones III, IV & V. These grade beams can either be placed at the foundation level or at about plinth level.

The grade beams are designed for taking up the load of filler walls and to stiffen all the columns in two directions.

The beams are designed for moments due to vertical loads and moments due to seismic forces. With this arrangement, the column is considered as restrained at the grade beam level and designed accordingly for moments due to vertical loads and horizontal seismic shear. This has the effect of reducing the height of the point of contraflexure and thereby reduce the moments on columns.

The isolated footings of such columns need not be designed for moments as the effect will be negligible except for deep footings when grade beams are provided at plinth level.

6.8 Raft Foundations

Raft foundations are generally provided—

(a) in locations where the soil has low bearing capacity and the column loads cannot be supported on individual footings or combined footings;

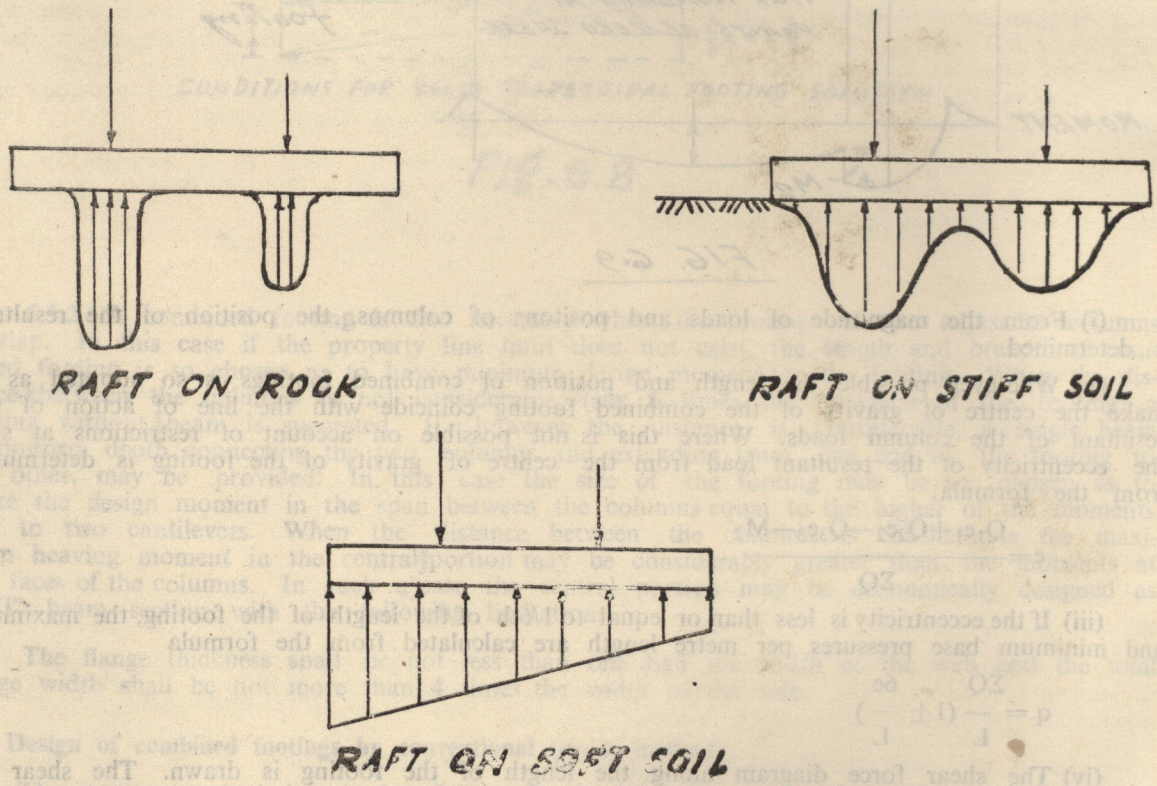


FIG. 6.10

(b) where soil may contain compressible lenses leading to the possibility of differential settlement which is difficult to ascertain;

(c) where floating type of foundations are required. (such foundations are required where the building is to be founded at a deeper level in order to get higher bearing capacity through replacement of soil).

(d) where building have deep basements and heavy water-proofing is required. (A raft is more amenable to external tanking system of water-proofing).

(e) where individual footings, if provided, will be subject to such bending moments which may result in differential rotations and differential settlements of individual footings.

The pressure distribution under the raft resting on different types of soil is indicated in figure 6.10

6·8·1. **TYPES OF RAFT**

On the basis of method of the support, the rafts are classified as—

(a) raft supported on soil; and

(b) raft supported on piles.

On the basis of structural system of the raft itself, it can be classified as—

(a) raft of uniform thickness; also known as mats, with or without pedestals and columns.

(b) ribbed raft, i.e. raft with beam and slabs—normally the beams will be projecting upwards and the ribbed raft will be like inverted floor system.

(c) cellular raft where the foundation slab, walls columns and the floor above act as one unit.

Raft of uniform depth which is otherwise called as Mat. foundation is most popular one, due to its simplicity in construction.

6.8.2. **Analysis and design of raft.**

The sizes of the raft are so determined as to transmit total load on soil without exceeding the permissible bearing capacity. In case of sandy soils the total load is taken as the combination of dead load and maximum estimate of live load. For clayee soils the total load is the combination of dead load and long term average of live load. This distinction is necessary because of the fact that on sandy soils the settlement is immediate whereas in the clayee type of soil it is prolonged one.

The centre of gravity of foundation raft should, as far as possible, be made to coincide with the centre of gravity of the vertical loads. After having done this, the total bending moments and sheer forces in either direction of the raft are calculated on the basis of combined footing approach as per para 6.6. The total bending moment and the sheer forces divided by the width in either direction is the lower bound value per unit width of the raft. The raft is then divided into strips bounded by centre lines of bays in each direction. Each strip thus obtained is then analysed as a combined footing considering the loads on the strip alone and the soil reaction due to these loads without reference to the overall resultant soil pressure. The bending moments and sheer forces so obtained divided by the width of each strip gives the upper bound values per unit width of raft in each strip. These values are bound to be on the higher side as the two-way distribution is not taken into account and also because of the fact that the soil pressure under the strip alone is considered without reference to the overall average soil pressure. These values of bending moments and the sheer forces may have, therefore, to be reduced by about 15 to 25% depending upon the variation of column loads and the spacing.

It is also to be seen that the bending moment at any section is not less than $WL^2/10$ where 'W' is the average overall soil pressure and 'L' is the distance between columns.

The thickness of raft is then decided in such a way that it is stiff and is capable of resisting the highest value of sheer forces without any sheer reinforcement and developing enough resisting moment as a singly reinforced section except in a few sections where double reinforcement may be required.

The reinforcement required for the lower bound value are divided uniformlly throughout the length and width of the raft at the appropriate face. Additional reinforcements is provided in strips to provide for the difference between the lower-bound and the adjusted upper bound values in the concerned strips.

It is also to be seen that on the face of the raft where theoretically no reinforcement is necessary, reinforcement to the extent of half the quantity of steel on the other face is provided. The minimum quantity of steel tobe provided is 0·15% of the gross sectional area of the raft on each face in each direction if mild steel is adopted. This minimum will be 0·12% in case of tor steel.

6.9 Illustrative Examples

EXAMPLE—Redistribution of base pressure when negative pressure is indicated due to effect of axial load and bending moment on a footing.

Data

Column load	100 tonnes
Moment on column	90 tonne metres
Footing size	3·4 m × 3·4 m
Depth of footing	1·5 metres
Thickness of footing at edge	25 cms.
Thickness of footing at face of pedestal	65 cms. (assumed)
Self weight of footing	4·634 × 2400 = 11·12 tonnes
Weight of soil	12·706 ×1920 = 24·396 tonnes
Total load on footing	100 + 11·12 + 24·396 = 135·516
	Say 135·52 tonnes

$$\frac{P}{A} = \frac{135 \cdot 52}{3 \cdot 4 \times 3 \cdot 4} = 11 \cdot 72 \text{ T/Sqm.}$$

$$Z = \frac{1}{6} \times 3 \cdot 4 \times 3 \cdot 4^2 = 6 \cdot 55$$

$$\frac{M}{Z} = \frac{90}{6 \cdot 55} = 13 \cdot 74 \text{ t/sqm.}$$

$$\frac{P}{A} \pm \frac{M}{Z} = 11 \cdot 72 \pm 13 \cdot 74 = 25 \cdot 46 \text{ T/Sqm.}$$

or (−) 2·02 T/Sqm.

$$\text{eccentricity @ } \frac{M}{P} = \frac{90}{135 \cdot 52} = 0 \cdot 664 \text{ m.}$$

$$\frac{L}{2} - e = \frac{L_1}{3} \qquad L_1 = 3\left(\frac{L}{2} - e\right)$$

$$= 3\left(\frac{3 \cdot 4}{2} - 0 \cdot 66\right) = 3 \cdot 12 \text{ m.}$$

$$\text{Total } P = \tfrac{1}{2} P_1 L_1 b$$

$$P_1 = \frac{2P}{L_1 b} = \frac{2 \times 135 \cdot 52}{3 \cdot 12 \times 3 \cdot 4} = 25 \cdot 55 \text{ t/sqm.}$$

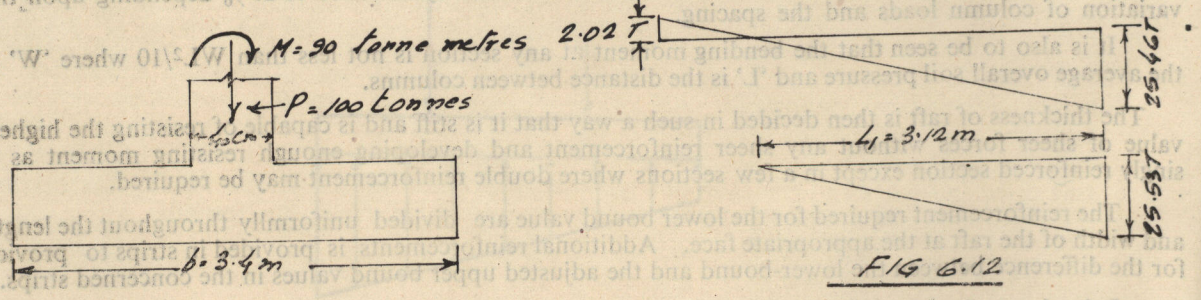


FIG 6.12

EXAMPLE

DESIGN OF AN ISOLATED SQUARE FOOTING

Data

Column load	100 tonnes
Column size	40 x 40 cms.
Permissible compressive stress in concrete 'C' 'c'	50 kg/cm²
Permissible tensile stress in steel 't'	2300 kg/cm²
Allowable soil pressure (net)	9 tonnes/Sqm.

$$m = \frac{2800}{3 \times 50} = 18 \cdot 67 \text{ Say } 19$$

$$\frac{m \times c}{t} = \frac{n}{d-n}$$

$$\frac{19 \times 50}{2300} = \frac{n}{d-n}$$

$$0 \cdot 413\, d = 1 \cdot 413\, n$$

$$n = \frac{0 \cdot 413\, d}{1 \cdot 413} = 0 \cdot 29\, d$$

$$jd = d - \frac{n}{3} = d\left\{1 - \frac{0 \cdot 29)}{3}\right\} = 0 \cdot 903\, d$$

$$MR = \frac{1}{2}\, n.c.b.\left\{d - \frac{n}{3}\right\}$$ [M.R. Moment of resistance]

$$= \frac{1}{2} \times 0 \cdot 29d \times 50 \times b \times 0 \cdot 903\, d$$

$$= 6 \cdot 5468\, b\, d^2 = Q\, b\, d^2$$

$$Q = 6.5468 \simeq 6.55$$

FOOTING SIZE

Column load	100 tonnes
Add 5%	5 tonnes
Total load	105 tonnes

Area of footing required $= \frac{105}{9} = 11 \cdot 67$ Sqm.

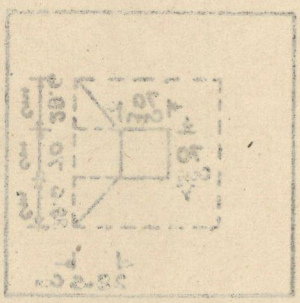
Assuming square footing is to be provided side of the square footing $= \sqrt{11 \cdot 67}$

Say 3.4 metres.

Net upward soil pressure $= \frac{100 \times 1000}{3 \cdot 4 \times 3 \cdot 4} = 8650$ kg/sqm.

Pedestal size = 70 x 70 cms.

Height of pedestal = 15 cms.



Depth from bending moment consideration

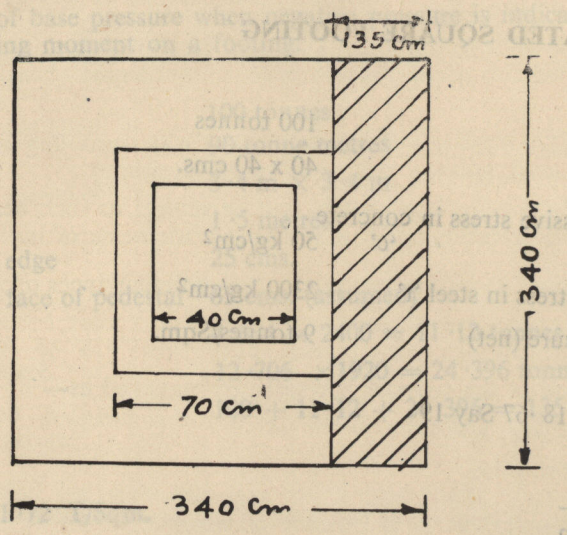


FIG. 6.13

Max. bending moment at the edge of the pedestal

$$M = 8 \cdot 65 \times \frac{1 \cdot 35 \times 1 \cdot 35 \times 3 \cdot 4}{2}$$

$$= 26 \cdot 80 \text{ tonne metre}$$

$$\text{Av-width of compression} = \frac{(70 + 2d) + 70}{2} = 70 + d$$

$$d = \sqrt{\frac{M}{\Phi b}}$$

$$d = \sqrt{\frac{26 \cdot 8 \times 10^5}{6 \cdot 55 \times (70 + d)}}$$

Or $d^2 \quad (70 + d) = 4 \cdot 092 \times 10^5$

$$d = 56 \cdot 81 \text{ cms.}$$

Overall depth $= 56 \cdot 81 + 6 + 1 \cdot 2 + 0 \cdot 6 = 64 \cdot 61$

Say 65 cms.

effective depth $= 65 - 7 \cdot 8 = 57 \cdot 2$ cms.

Check for punching at a distance d/2 from face of pedestal

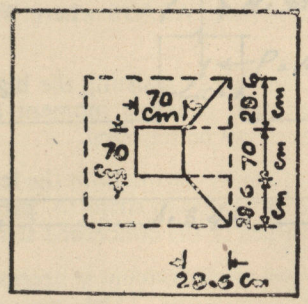


FIG. 6.14

Punching shear $= 100{,}000$

$$- 8650 \times \frac{127 \cdot 2 \times 127 \cdot 2}{100 \times 100}$$

$$= 100{,}000 - 14000$$

$$= 86000 \text{ kg.}$$

Perimeter of footing resisting punching shear $= 4 \times (70 + 28 \cdot 6 + 28 \cdot 6) = 508 \cdot 8$ cms.

Effective depth at a distance d/2 from pedestal face

$$= 25 + \frac{(65-25)}{1\cdot35} \times \left\{1\cdot35 \times \frac{0\cdot572}{2}\right\} - 6 - 1\cdot2 - 0\cdot6$$

$= 48\cdot72$ cms.

Punching shear stress $= \dfrac{86000}{508\cdot8 \times 48\cdot72} = 3\cdot47$ kg./cm^2 < 10 kg./cm^2

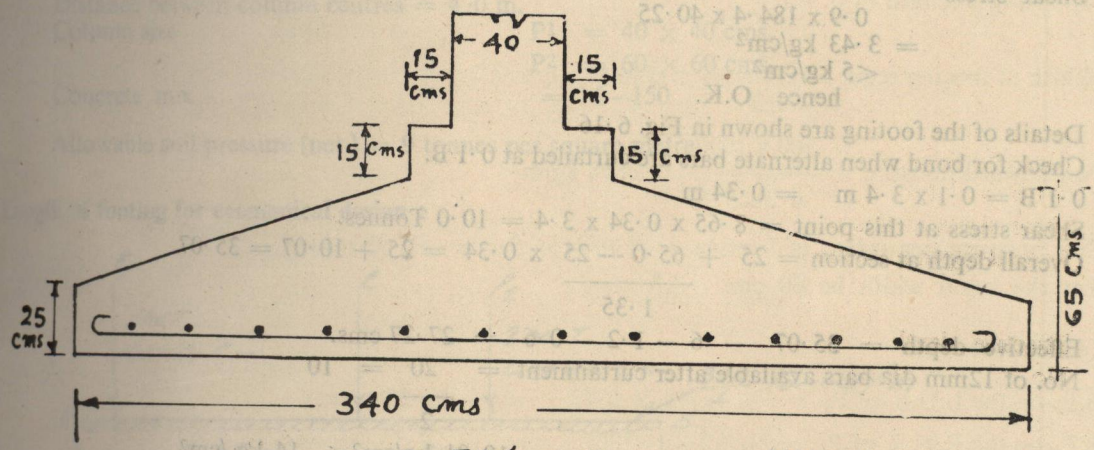


FIG. 6.15

Effective depth to the centre of upper layer of bars

$= 65 - 6 - 1\cdot2 - 0\cdot6 = 57\cdot2$ cms.

$A_t = \dfrac{26\cdot8 \times 10^5}{2300 \times 0\cdot9 \times 57\cdot2} = 22\cdot63$ cm^2

No. of 12 mm ϕ tor bars $= \dfrac{22\cdot63}{1\cdot13} = 20\cdot03$

$\simeq 20$ Nos.

Same steel may be provided in perpendicular direction.

Check for bond stress for column bars.

Anchor length required $= \dfrac{1750 \times 1\cdot2}{5 \times 7} = 60$ cms.

Depth available $= 15 + 57\cdot2 = 72\cdot2 > 60$ cms.

Check for bond stress of bars in the footing

S.F. $= 8\cdot65 \times 1\cdot35 \times 3\cdot4$

$= 39\cdot704$ Tonnes

Bond stress $= \dfrac{39\cdot704 \times 10^3}{0\cdot9 \times 57\cdot2 \times 20 \times 3\cdot77}$

$= 10\cdot23$ kg/cm$^2 < 14$ kg/cm^2

Check for diagonal tension

Overall depth at the critical section.

$$= 25 + \frac{(65-25)}{1\cdot35} \times (1\cdot35 - 0\cdot572)$$

$$= 25 + \frac{40}{1\cdot35} \times 0\cdot778$$

$= 25 + 23\cdot05$

$= 48\cdot05$ cms.

eff. depths at critical section

$= 48\cdot05 - 6 - 1\cdot2 - 0\cdot6 = 40\cdot25$ cms.

Shear force at critical section
$= 8 \cdot 65 \times (1 \cdot 35 - 0 \cdot 572) \times 3 \cdot 4 = 22 \cdot 881$ T

eff. width of compression $= 70 + 2d$
$= 70 + 2 \times 57 \cdot 2$
$= 70 + 114 \cdot 4$
$= 184 \cdot 4$ cms.

$$\text{Shear stress} = \frac{22 \cdot 88 \times 1000}{0 \cdot 9 \times 184 \cdot 4 \times 40 \cdot 25}$$
$= 3 \cdot 43$ kg/cm^2
< 5 kg/cm^2
hence O.K.

Details of the footing are shown in Fig. 6 ·16

Check for bond when alternate bars are curtailed at 0 ·1 B.

$0 \cdot 1 B = 0 \cdot 1 \times 3 \cdot 4$ m $= 0 \cdot 34$ m

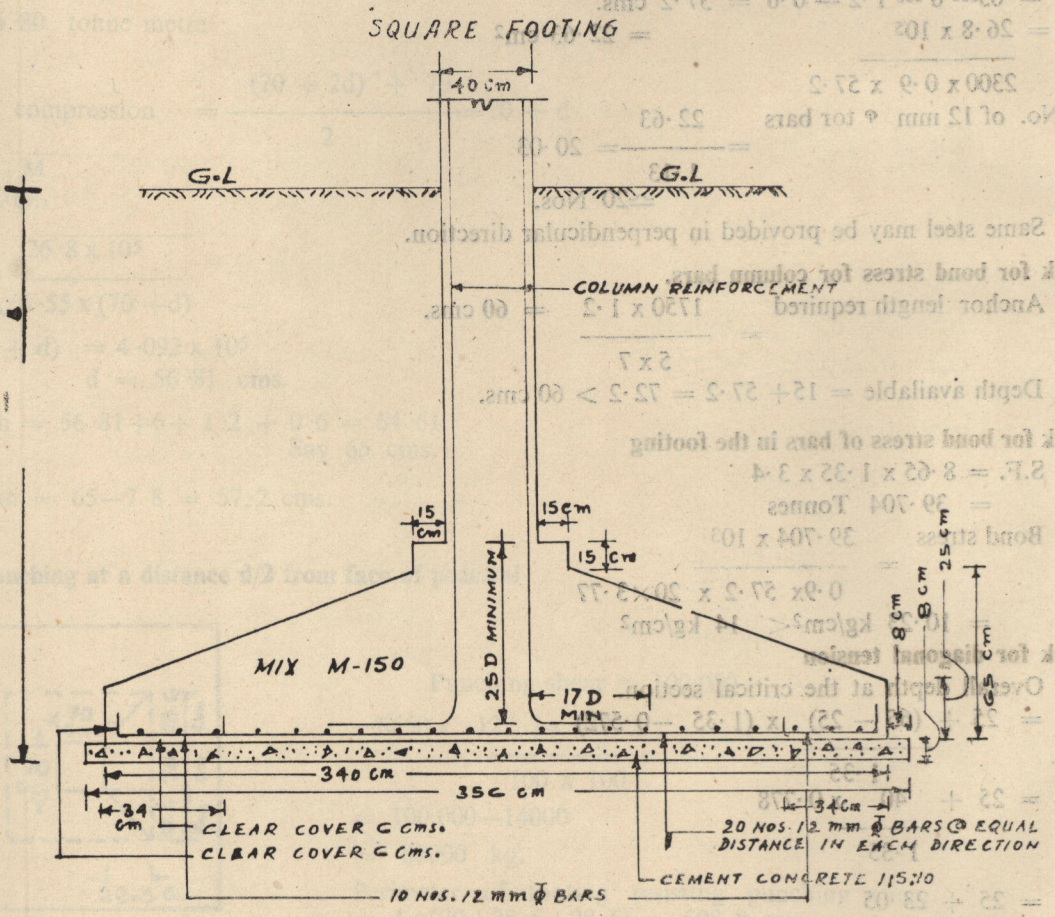
Shear stress at this point $= 8 \cdot 65 \times 0 \cdot 34 \times 3 \cdot 4 = 10 \cdot 0$ Tonnes.

Overall depth at section $= 25 + \frac{65 \cdot 0 - 25}{1 \cdot 35} \times 0 \cdot 34 = 25 + 10 \cdot 07 = 35 \cdot 07$

Effective depth $= 35 \cdot 07 - 6 - 1 \cdot 2 - 0 \cdot 6 = 27 \cdot 27$ cms.

No. of 12mm dia bars available after curtailment $= \frac{20}{2} = 10$

$$\text{Bond stress} = \frac{10 \times 1000}{0 \cdot 9 \times 27 \cdot 27 \times 10 \cdot \times 3 \cdot 77} = 10 \cdot 81 \text{ kg/cm}^2 < 14 \text{ kg/cm}^2$$
Hence Safe



SECTION OF FOOTINGS

FIG. 6.16

EXAMPLE

DESIGN OF A COMBINED FOOTING

Data

Column loads $P_1 = 100$ tonnes
$P_2 = 200$ tonnes.

Distance between column centres = 4·0 m.

Column size $P_1 = 40 \times 40$ cms.
$P_2 = 60 \times 60$ cms.

Concrete mix = M—150

Allowable soil pressure (net) = 9 tonnes per square metre.

Length of footing for economical design :

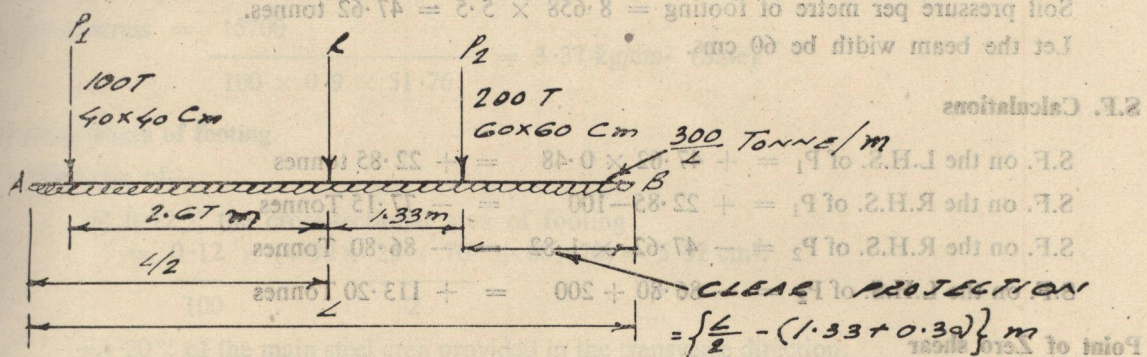


FIG 6·17

Point where S.F. will be zero from A = X metres

$$\frac{300}{L} \times X = 100$$

$$X = \frac{100 \times L}{300} = \frac{L}{3} \text{ metres}$$

Here B.M. will be maximum (hogging).

Equating max. cent. moment with max. hogging moment.

$$\frac{300}{2L}\left(\frac{L}{2} - 1\cdot63\right)^2 = 200\left\{\frac{2L}{3} - \left(\frac{L}{2} - 1\cdot33\right)\right\} - \frac{300}{2L}\left(\frac{2L}{3}\right)^2$$

$$= \frac{300}{2L}\left(\frac{L^2}{4} + 2\cdot66 - 1\cdot63L\right) = 200\left\{\frac{L}{6} + 1\cdot33\right\} - \frac{300}{2L} \times \frac{4L^2}{9}$$

$$= 37\cdot5L + \frac{399}{L} - 244\cdot5 = 33\cdot33L + 266 - 66\cdot67L$$

$$= 70\cdot84L^2 + 399 - 510\cdot5L = 0$$

$$= L^2 - 7\cdot21L + 5\cdot64 = 0$$

$$= (L - 3\cdot6)^2 = 7\cdot32$$

$$= (L - 3\cdot6)^2 = (2\cdot7)^2$$

$$= L - 3\cdot6 = 2\cdot7$$

$$\therefore L = 2\cdot7 + 3\cdot6 = 6\cdot3 \text{ m}$$

Superimposed load on both columns = 300 tonnes
Add 5% = 15 tonnes.

Total 315 Tonnes

$$\text{Area of footing reqd.} = \frac{315}{9} = 35 \text{ Sq. m.}$$

Adopting a length of 6.3m

$$\text{Width of footing reqd.} = \frac{35}{6 \cdot 3} = 5 \cdot 5 \text{ m (Say)}$$

$$\text{Net upward pressure} = \frac{300 \times 1000}{6 \cdot 3 \times 5 \cdot 5} = 8658 \text{ kg/m}^2$$

Soil pressure per metre of footing = $8 \cdot 658 \times 5 \cdot 5 = 47 \cdot 62$ tonnes.
Let the beam width be 60 cms.

S.F. Calculations

S.F. on the L.H.S. of P_1 = $+ 47 \cdot 62 \times 0 \cdot 48$ = $+ 22 \cdot 85$ tonnes
S.F. on the R.H.S. of P_1 = $+ 22 \cdot 85 - 100$ = $- 77 \cdot 15$ Tonnes
S.F. on the R.H.S. of P_2 = $- 47 \cdot 62 \times 1 \cdot 82$ = $- 86 \cdot 80$ Tonnes
S.F. on the L.H.S. of P_2 = $- 86 \cdot 80 + 200$ = $+ 113 \cdot 20$ Tonnes

Point of Zero shear

Let this be at X metres from A.

$$47 \cdot 62 \text{ X} - 100 = 0$$

$$\text{X} = \frac{100}{47 \cdot 62} = 2 \cdot 1 \text{ metres}$$

Design of slab as cantilever

$$\text{Span of Slab} = \frac{5 \cdot 50 - 0 \cdot 60}{2} = \frac{4 \cdot 90}{2} = 2 \cdot 45 \text{ m.}$$

$$\text{Max. B.M. in Cantilever} = \frac{8658 \times (2 \cdot 45)^2}{2}$$

$$= 26000 \text{ kg. m. or } 2600000 \text{ kg. cm.}$$

$$\text{Thickness of slab} = \sqrt{\frac{2600000}{100 \times 6 \cdot 55}} = 63 \text{ cms.}$$

Taking clear cover of 6 cms. & using 12 mm tor bars.

Total depth = $63 + 0 \cdot 6 + 6 \cdot 0$
= $69 \cdot 6$ cms. Say 70 cms.

Actual Effective depth provided = $70 - 6 \cdot 6 = 63 \cdot 4$ cms.

$$A_t = \frac{2600000}{2300 \times 0 \cdot 9 \times 63 \cdot 4} = 19 \cdot 8 \text{ Sq. cms.}$$

$$\text{Spacing of 12 mm tor bars} = \frac{1 \cdot 13 \times 100}{19 \cdot 8} = 5 \cdot 7 \text{ cms. c/c}$$

Say 5·5 cms. c/c at bottom of slab.

Check for bond

Max. S.F. = 8658 x 2·45 = 21200 kg.

Bond stress = $\dfrac{21200 \times 7 \times 5{\cdot}5}{0{\cdot}9 \times 63{\cdot}4 \times 1{\cdot}2 \times 22 \times 100}$ = 5·4 kg/cm² (Safe)

Check for diagonal tension

S.F. = 8658 × (2·45 − 0·634)

= 8658 × 1·816 = 15700 kg.

Let the slab be tapered uniformly to 25 cms. over all at the edges.

Overall depth at 63·4 cms. from beam = $70 - \dfrac{(70-25) \times 63{\cdot}4}{245}$

= 70 − 11.64 = 58.36 cms.

Effective depth = 58·36 − 6·6 = 51·76 cms.

Shear stress = $\dfrac{15700}{100 \times 0{\cdot}9 \times 51{\cdot}76}$ = 3·37 kg/cm² (Safe)

Steel along length of footing

Minimum of—

(i) 0·12% the cross-sectional area of footing

= $\dfrac{0{\cdot}12}{100} \times \left(4{\cdot}90 \times \dfrac{25+70}{2} + 60 \times 70\right)$ = 5·32 cm².

(ii) 20% of the main steel area provided in the transverse direction.

= $0{\cdot}20\left[1{\cdot}13 \times \left(\dfrac{630}{5{\cdot}5} + 1\right)\right]$ = 26·2 cm²

Area of distribution steel along length of footing per m. width

= $\dfrac{26{\cdot}2}{5{\cdot}5}$ = 4·76 cm² /m

Longitudinal beam

Max. sagging moment at the outer face of column P_2

= $\dfrac{47{\cdot}62 \times 1{\cdot}52^2}{2}$ = 55·0 t. m.

Max. sagging moment at inside face of col. P_2

= $47{\cdot}62 \times \dfrac{2{\cdot}12^2}{2} - 200 \times 0{\cdot}30$

= 107 − 60 = 47·0 t.m.

Max. hogging moment between the cols. (at point of Zero shear).

= $47{\cdot}62 \times \dfrac{2{\cdot}1^2}{2} = 100\,(2{\cdot}1 - 0{\cdot}48)$ = 57·0 tonne metre

Bending moment at centre of col. P_1 = $\dfrac{47{\cdot}62 \times 0{\cdot}48^2}{2}$

= 5·5 t. m. sagging.

Depth of beam needed for sagging moments

= $\sqrt{\dfrac{55 \times 100000}{60 \times 6{\cdot}55}}$ = 118·5 cms,

This depth will be quite safe for hogging moments also where the beam acts as a T-beam.

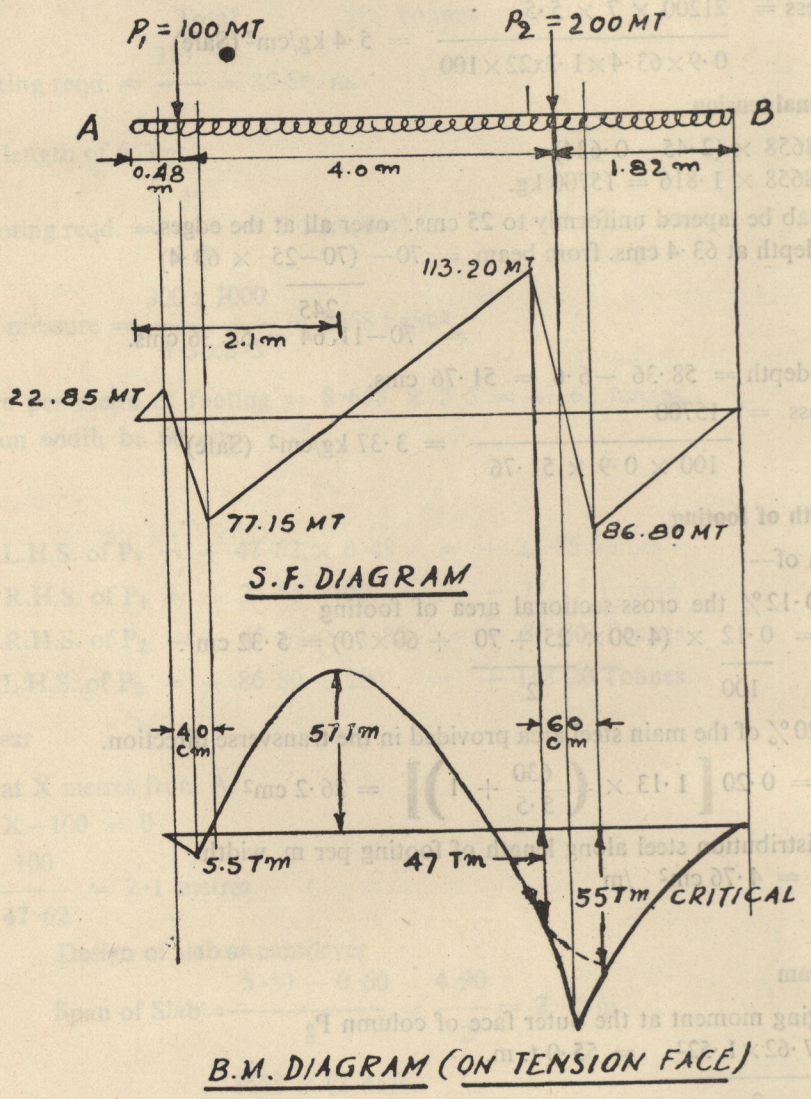


FIG. 6.18

Adopt 118·5 cms. effective depth.

Overall depth = (6·0 + 1·2 + 1·0 + 0·8 + 1·2 + 1·0) + 118·5
= 129·7 Say 130 cms.

Actual effective depth = 130·0—11·2 = 118·8 cms.
Say 119 cms.

Reinforcement for sagging moment = $\frac{55 \times 100000}{2300 \times 0 \cdot 9 \times 119}$ = 22·3 cm²

Min. steel required = $\frac{0 \cdot 2}{100} \times 60 \times 130$ = 15·60 cm²

Provide 21 bars 12 mm tor (area 23·73 cm²)

Reinforcement for hogging moment $= \dfrac{57 \times 100000}{2300 \times 0 \cdot 9 \times 119} = 23 \cdot 1 \text{ cm}^2$

Provide 12 mm tor bars 21 Nos.

Shear force at outside face of Col. $P_2 = 47 \cdot 62 \times 1 \cdot 52$
$= 72 \cdot 50$ Tonnes.

Shear force at inside face of col. $P_2 = 200 - 47 \cdot 62 \times 2 \cdot 12$
$= 200 - 101 = 99 \cdot 0$ tonnes.

Bond stress at inside face of Col. $P_2 = \dfrac{99 \times 1000}{0 \cdot 9 \times 119 \times 21 \times 3 \cdot 77}$
$= 11 \cdot 7 \text{ kg./cm}^2$
(Less than 14 kg/cm² O.K.)

Sagging moment at inside face of col. $P_2 = 47 \cdot 0$ tm.

S.F. at inside face of col. $P_2 = 99 \cdot 0$ Tonnes.

Let point of inflexion be at a distance X from inside face of col. P_2

$$99 \cdot 0 X = 47 + \frac{47 \cdot 62 X^2}{2}$$

or $23 \cdot 81X - 99X + 47 = 0$

or $X^2 - 4 \cdot 16 X + 1 \cdot 97 = 0$

$(X - 2 \cdot 08)^2 = 4 \cdot 33 - 1 \cdot 97 = 2 \cdot 36 = (\pm 1 \cdot 54)^2$

$X - 2 \cdot 08 = \pm 1 \cdot 54$

$X = 2 \cdot 08 - 1 \cdot 54 = 0 \cdot 54$ m.

or $2 \cdot 08 + 1 \cdot 54 = 3 \cdot 62$ m.

For anchorage—Maximum of the two shall be adopted.

(i) $69d = 69 \times 1 \cdot 2 = 82 \cdot 8$ cm or $0 \cdot 828$ m.

(ii) 12 dia. or 30 cm. whichever is more beyond the point of inflexion.

Therefore bottom bars shall extend $0 \cdot 54 + 0 \cdot 30 = 0 \cdot 84$ m from the inside face of col. P_2

S.F. at point of inflexion.

$= 99 \cdot 0 - 47 \cdot 62 \times 0 \cdot 54$

$= 99 \cdot 0 - 25 \cdot 7$

$= 73 \cdot 3$ tonnes.

No. of bars required for safe bond stress at top

$$= \frac{73.3 \times 1000}{0 \cdot 9 \times 119 \times 3 \cdot 14 \times 14 \times 1 \cdot 2} = 13$$

Point of inflexion is quite near the cols.

Hence all the top bars may continue upto the col.

Diagonal tension

Effective depth of beam = 119 cms. = 1 ·19 m

Distance of point of inflexion from $P_2 = 0 \cdot 54$ m

Diagonal tension will be tested at the point of inflexion.

$$\text{Shear stress} = \frac{73 \cdot 3 \times 1000}{60 \times 0 \cdot 9 \times 119}$$

$= 11 \cdot 4 \text{ kg/ cm}^2$ (more)

S.F. in the cantilever at a distance of 119 cms. from face of col. P_2

$= 47 \cdot 62 \times 0 \cdot 33 = 15 \cdot 8$ Tonnes.

$$\text{Shear stress} = \frac{15 \cdot 8 \times 1000}{60 \times 0 \cdot 9 \times 119}$$

$$= 2 \cdot 45 \text{ kg/ cm}^2 \text{ (safe)}$$

S.F. for which the shear stress will be 5 kg/cm^2 $= 60 \times 0 \cdot 9 \times 119 \times 5$

$= 32200$ kg.

$$\text{Spacing of 8 mm } \phi \text{ tor legged stirrups} = \frac{1750 \times 8 \times 0.5 \times 0 \cdot 9 \times 119}{73 \cdot 3 \times 1000}$$

$= 10 \cdot 22$ cms. c/c.

Provide @ 10 cms. c/c.

These may be provided upto 1 ·5 m from inside face of col. P_2 where shear force drops down to

$= 3 \cdot 62 \times 47 \cdot 62 - 200$

$= 27 \cdot 62$ tonnes.

For col. P_1

$$\text{Sagging moment at the centre of col. } P_1 = \frac{47 \cdot 62 \times 0.48^2}{2}$$

$= 5 \cdot 5$ tonne metre

Min. moments

Inside face of col. $P_1 = 47 \cdot 0$ t. m. (Sagging) as the minimum moment to be provided for is

$$= \frac{w l^2}{16} = 47 \cdot 0 \text{ t.m.)}$$

$$\text{Reinforcement for sagging moment} = \frac{47 \cdot 0 \times 100000}{2300 \times 0 \cdot 9 \times 119} = 19 \cdot 1 \text{ cm}^2$$

$$\text{No. of 12 mm } \Phi \text{ tor bars} = \frac{191 \cdot 1}{1 \cdot 13} = 17 \text{ bars.}$$

S.F. at outside face of col. $P_1 = 47 \cdot 62 \times 0 \cdot 28 = 13.4$ tonnes

S.F. at inside face of col.. P_1

$= 100 - 47 \cdot 62 \times 0 \cdot 68$

$= 100 - 32 \cdot 4$

$= 67 \cdot 6$ tonnes

Bond stress at inside face of col. P_1

$$= \frac{67.6 \times 1000}{0 \cdot 9 \times 119 \times 17 \times 3 \cdot 14 \times 1 \cdot 2}$$

$= 9 \cdot 9$ kg/cm^2 (safe)

$69 d = 69 \times 1 \cdot 2 = 82 \cdot 8$ or $0 \cdot 83$ m.

So extend bottom steel by 0 ·84 m from inside face of Col. P_1

From statics, point of inflexion is $3.62 - 3.50 = 0 \cdot 12$m from face of Col. P_1 towards the centre of col.

S.F. at point of inflexion $= 73 \cdot 3$ tonnes.

No. of bars required for safe bond stress at top $= 13$ bars.

All the top bars must continue (upto the end of footing)

Diagonal tension

Shear stress at point of inflexion = 11·4 kg/cm 2.

Shear stress in the cantilever = 2·08 kg/cm² (Safe)

Safe shear force for stress of 5 kg/cm² = 32·2 tonnes.

Spacing of 8 mm Φ tor 8 legged stirrups = 13cm. c/c.

These may be provided upto 0·75m from inside face of Col. P_1 where shear force drops down to
=1·43 × 47·62 −100 = 31·90 tonnes.

In the remaining portion provide nominal stirrups, say 8 mm Φ tor 4 legged @ 30 cms. c/c.

Depth/span ratio

The effective depth = 119 cms.

Cantilever span of beam =1·52 metres

Depth/span ratio $= \frac{119}{152} = 0{\cdot}78 > 0.4$

It is a deep beam. Provide additional reinforcement required $= \frac{0{\cdot}1 \times 60 \times 130}{100} = 7{\cdot}80$ cm²

Using 12 mm Φ tor bars, No. of bars required $= \frac{7{\cdot}80}{1{\cdot}13} = 7$ No.

Provide 12 mm Φ bars 4 Nos. on each vertical face of the beam.

The details of the combined footing are shown in Fig. 6·19

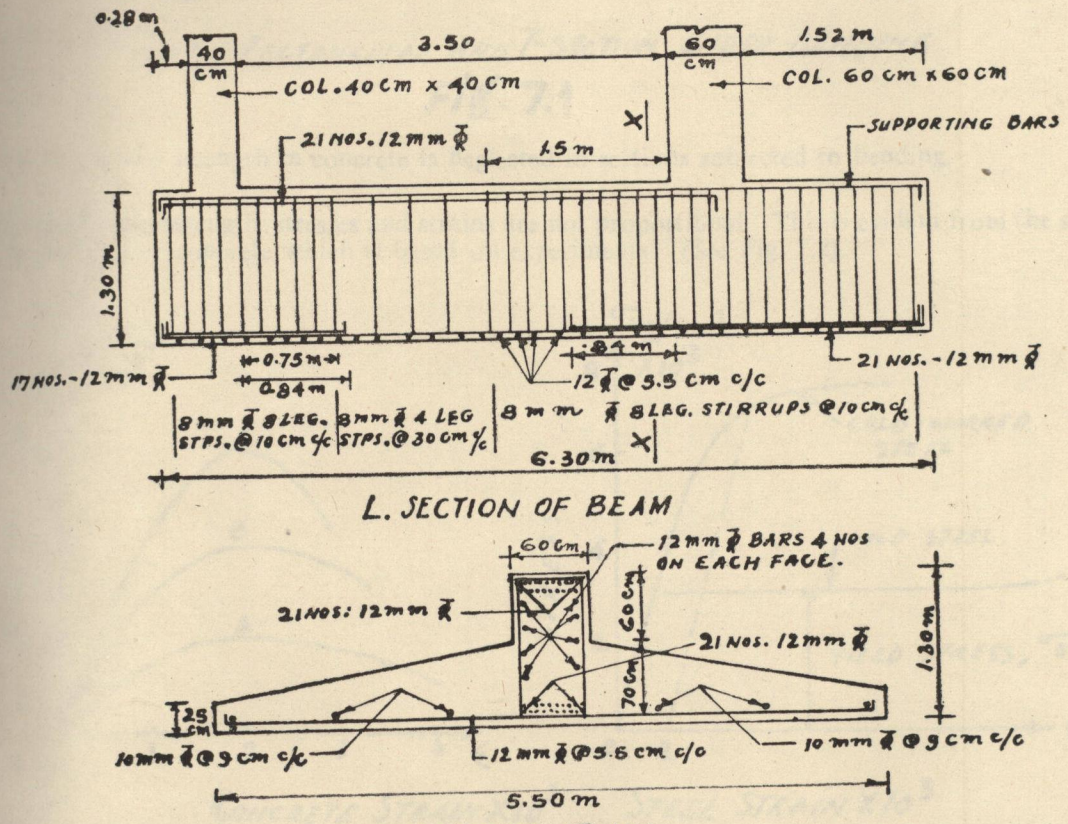


FIG. 6.19

BIBLIOGRAPHY

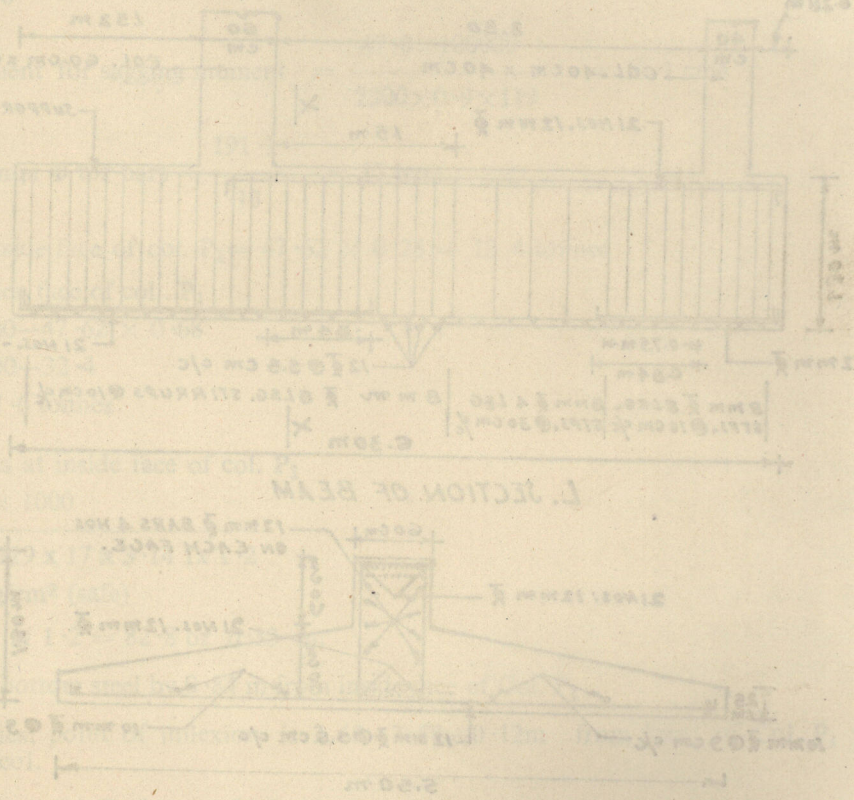
1. JOSEPH BOWELS, "Foundation analysis and Design" Mc Graw-Hill Book Company.
2. CLARENCE W. DUNHAM, "Foundations of Structures" Second Edition, McGraw Hill Book Company, Inc. New York—Toronto—London, 1962.
3. RALPH B. PECK, WALTER E. HANSON AND THOMAS H. THORNBURN, "Foundation Engineering", Second Edition, John Willey & Sons, INC., New York, London, Sydney, Toronto.
4. WAYNE C. TENG, "Foundation Design" Prentice Hall, Inc. 1962.
5. ELWYN E, SEELYE, "Foundations—Design and Practice", John Willey and Sons, Inc., New York, London, Sydney 1966.
6. JAI KRISHNA AND O. P. JAIN, "Plain and Reinforced Concrete", New Chand and Bros., Roorke 1966.
7. WINTER, URQUHART, O'ROURKE, NILSON, "Desin of Concrete Structures", McGraw Hill Company.

REFERENCES

I.S. 1904 . . . . . Code of Practice for structural safety of building foundations.

I.S. 2950 . . . . . Code of Practice for design and construction of Raft foundations.

I.S. 456 . . . . . Code of Practice for Plain and Reinforced concrete.



CHAPTER 7

DESIGN OF SLABS, BEAMS AND COLUMNS

7.1 Ultimate Load Method of Design

The Code of Practice for Plain and Reinforced Concrete I.S. 456-1964 indicates the ultimate load method of design. The design of constituent members of a structure should conform to the provisions in this Code.

The ultimate load method of design of reinforced concrete sections is based on the following assumptions:—

(1) Plane sections, normal to the axis, remain plane after bending. The validity of this assumption has been established from test results. On this account the distribution of strain across the depth of a cross-section follows a straight line. (See Fig. 7.1).

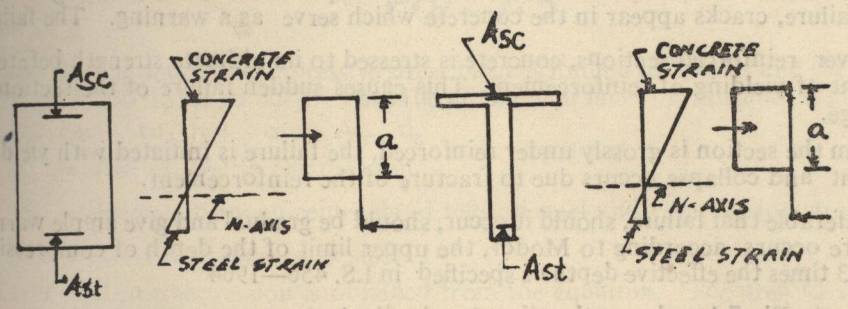


FIG. 1. RECTANGULAR AND T-SECTION UNDER BENDING

Fig. 7.1

(2) The tensile strength in concrete is neglected in sections subjected to bending.

(3) At ultimate strength, stresses and strains are not proportional. This is evident from the stress strain relationship of concrete which is based on experiments. (See Fig. 7.2).

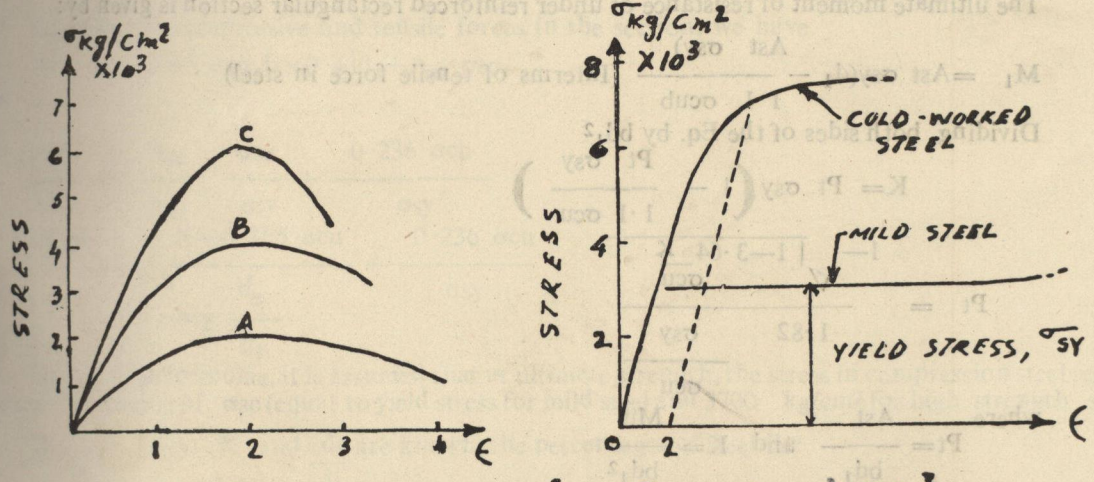


Fig. 7.2

(4) The maximum fibre stress in concrete should not exceed 68% of the cube strength of concrete.

7.1.1 Rectangular sections under flexure with tension reinforcement

From the above considerations the stress diagram can be drawn from the strain diagram and the stress versus strain relationship. (See Fig. 7.1 & 7.2).

I.S. 456-1964 permits that the diagram of compressive concrete stress distribution may be assumed rectangular, trapezoid, parabola or any other shape which results in ultimate strength in reasonable agreement with tests. Accordingly, the compressive stress diagram is replaced as an equivalent rectangle of depth equal to $0 \cdot 43\ xd_1$ and with uniform compressive stress of $0 \cdot 55\ \sigma cu$. where 'd_1 is the effective depth of the member and σcu the cube strength of the concrete. Thus the ultimate moment of resistance Mu for a balanced section $= 0 \cdot 55\ \sigma cu \times b \times 0 \cdot 43\ d_1 \left((d_1 - \frac{0 \cdot 43\ d_1}{2} \right) = 0 \cdot 185 \times \sigma cu\ bd_1^2$..

Equating the ultimate moments of resistance in terms of concrete and steel we obtain an area of tensile steel required for the balanced condition.

Three principal modes of failure of rectangular section with tensile reinforcement are :

(1) With under reinforced section, the failure is initiated by yielding of the reinforcement. Consequently, the neutral axis shifts upwards and the final collapse is due to the crushing of concrete. Before the ultimate failure, cracks appear in the concrete which serve as a warning. The failure is gradual.

(2) In over reinforced sections, concrete is stressed to its ultimate strength before the commencement of yielding of reinforcement. This causes sudden failure of the section at the ultimate stage.

(3) When the section is grossly under reinforced, the failure is initiated with yielding of reinforcement and collapse occurs due to fracture of the reinforcement.

It is preferable that failure, should it occur, should be gradual and give ample warning. To ensure that the failure occurs according to Model, the upper limit of the depth of compression zone is to be limited to 0 ·43 times the effective depth as specified in I.S. 456—1964

Referring to Fig 7.1 and equating the longitudinal forces we get

$0.55\ \sigma cuba = \sigma sy\ Ast$

The lever arm of the resisting moment is given by:

$$jd_1 = d_1 - \frac{a}{2}$$

$$= d_1 - \frac{Ast \quad \sigma sy}{1 \cdot 1 \quad \sigma cub}$$

The ultimate moment of resistance of under reinforced rectangular section is given by:

$$M_1 = Ast\ \sigma sy\left(d_1 - \frac{Ast\ \sigma sy}{1 \cdot 1\ \sigma cub}\right)$$ (Interms of tensile force in steel)

Dividing both sides of the Eq. by bd_1^2

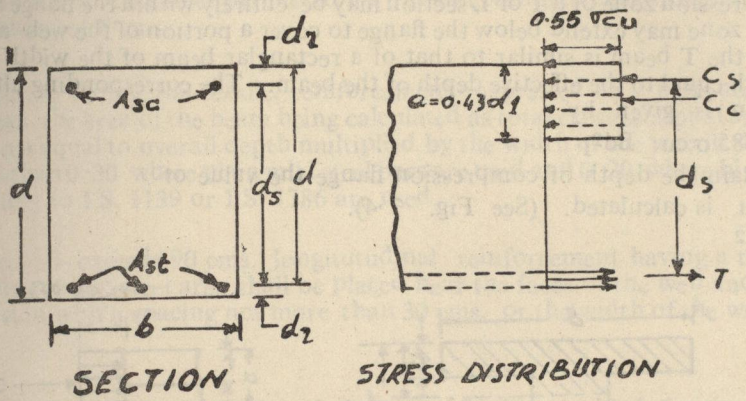
$$K = Pt\ \sigma sy\left(1 - \frac{Pt\ \sigma sy}{1 \cdot 1\ \sigma cu}\right)$$

$$Pt = \frac{1 - \sqrt{1 - 3 \cdot 64 \frac{K}{\sigma cu}}}{1 \cdot 82 \frac{\sigma sy}{\sigma cu}}$$

where $Pt = \frac{Ast}{bd_1}$ and $K = \frac{Mu}{bd_1^2}$

If σcu, σsy and K are known, the percentage of steel Pt can be obtained from the above equation. For mild steel with $\sigma sy = 2600 kg/cm^2$ and for high yield strength deformed bars with $\sigma sy = 4250\ kg/cm^2$, values of Pt have been tabulated against different values of K, for concrete grades M—150 to M 250 in Tables 1 to 6.

7.1.2 Rectangular Section Under flexure with tension and compression Reinforcements :



DOUBLY REINFORCED RECTANGULAR SECTION

Fig. 7.3

In this case also the depth of compressive stress block is limited to 0 ·43 d_1 and the moment resisted by the concrete alone is 0 ·185 σ cu bd^2_1. Thus the ultimate moment of resistance is given by

$M_u = 0 \cdot 185\ \sigma cu\ bd_1^2 + \sigma sc.\ A_{sc}\ d_s$

Where A_{sc} = area of steel in compression.

d_s = distance between centroids of tension steel and compression steel.

σsc = stress in compression steel.

The area of steel in compression is obtained from the equation. The area of steel in tension is obtained by equating the tension forces to the compression in concrete and steel.

From the above equation, we get

$$P_{sc} = \frac{K - 0 \cdot 185\ \sigma cu}{\sigma sc\ \frac{d_s}{d_1}}$$

where $P_{sc} = \frac{A_{sc}}{bd}$

Equating the compressive and tensile forces in the section, we have

$Ast\ \sigma sy = 0 \cdot 55\ \sigma cu\ b \times 0 \cdot 43 d_1 + A_{sc}\ \sigma sc$

or

$$\frac{Ast}{bd_1} = \frac{A_{sc}}{bd_1}\ \frac{\sigma sc}{\sigma sy} + \frac{0 \cdot 236\ \sigma cu}{\sigma sy}$$

Hence

$$pt = \frac{K - 0 \cdot 185\ \sigma cu}{\sigma sy\ \frac{d_s}{d_1}} + \frac{0 \cdot 236\ \sigma cu}{\sigma sy}$$

In the above formulae, it is assumed that at ultimate strength, the stress in compression steel reaches the maximum value of σsc (equal to yield stress for mild steel and 3700 kg/cm² for high strength steel).

If σcu, σsy, K and $\frac{d_s}{d_1}$ are known, the percentages of steel Psc and Pt can be obtained from the above equation. For σsy=2600 kg/cm² and σsy=4250 kg/cm², the values of Pt and Psc have been tabulated against different values of K for various values of $\frac{d_s}{d_1}$ and various concrete mixes in Tables 7, 8 and 9 for mild steel and Tables 10, 11 and 12 for high strength steel.

7.1.3 T or L beams under flexure with tension reinforcement alone

The compression zone of a T or L section may be entirely within the flange thickness or the depth of compression zone may extend below the flange to cover a portion of the web also. In the former case the analysis of the T beam is similar to that of a rectanglar beam of the width equal to the width of flange and depth equal to the effective depth of the beam. The corresponding ultimate resisting moment for the section is given by

$Mu = 0 \cdot 185\ \sigma cu\ Bd^2_1$

To ascertain the depth of compression flange the value of

$K = \frac{Mu}{Bd_1^2}$ is calculated. (See Fig. 7·4).

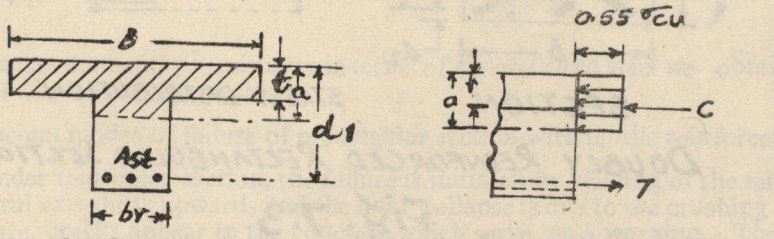


FIG. 4 EFFECTIVE CROSS-SECTION OF T-BEAM WITH a > t

FIG. 7.4

Based on the K value the corresponding depth of compression zone is calculated from table If this depth is less than the thickness of the flange corresponding area of steel reinforcement is obtained from the same tables.

If the depth of the compression zone is more than the flange thickness then the following procedure is adopted. The value of $\frac{Mu}{\sigma cu\ Bd^2}$ is compared with the value of K_t obtained from Table 13

If $\frac{Mu}{\sigma cu\ Bd^2_1}$ is less than or equal to Kt then area of the steel is calculated as follows:-

For the usual range of values of $\frac{t}{d_1}$ and $\frac{b}{B}$ (i.e. for $\frac{t}{d_1}$ greater than 0·15 and values of $\frac{b}{B}$ between 0·10 and 0·35), the tension reinforcement may be obtained as follows without any appreciable error;

$$A_{st} = \frac{Mu}{\sigma sy\ (d_1 - t/2)}$$

(ii) For values of $\frac{t}{d_1} \leqslant 0 \cdot 15$ and $\frac{b}{B} \leqslant 0 \cdot 35$, the tension steel is given by:

$$A_{st} = \frac{Mu}{\sigma sy\ j\ d_1}$$

when $j = \dfrac{\left(1 - \frac{b}{B}\right)\left(\frac{t}{d_1}\right)\left(1 - \frac{1}{2}\frac{t}{d_1}\right) + \left(\frac{b}{B}\right)\left(\frac{a}{d_1}\right)\left(1 - \frac{1}{2}\frac{a}{d_1}\right)}{\left(1 - \frac{b}{B}\right)\left(\frac{t}{d_1}\right) + \left(\frac{b}{B}\right)\left(\frac{(a)}{d_1}\right)}$

7.2. General Requirements

In the analysis of continuous frames c/c distance are adopted in the determination of moments. Moments at faces of support may be used for design of beams and girders.

Isolated beams in which the T form is used only for the purpose of providing additional compression area, shall have a flange thickness not less than one half the width of the web and a total flange width not more than 4 times the width of the web.

In concrete joist floor construction joist ribs shall be atleast 10 cms. wide spaced not more than 75 cms. clear and of a depth not more than 3 1/2 times their minimum width. Ribbed slab construction not meeting the above limitations shall be designed as slabs and beams.

All members subjected to a compression load shall be designed for the eccentricity 'e' corresponding to the maximum moment which can accompany this loading condition but not less than 0 ·05 h for spirally reinforced compression members 0 ·10 h for tied compression members about either principal axis where h is the overall width of member parallel to 'e'. Such eccentricity shall be limited to minimum of 2 ·5 cms and a maximum of 8 cms.

In reinforced concrete beams, the tensile, reinforcement expressed as a percentage of the gross sectional area of the beam, the area of the beam being calculated as total cross sectional area for rectangular beams and as the area equal to overall depth multiplied by the width of the web in the case of T and L beams shall be not less than 0 ·30 where plain mild steel bars are used and 0 ·20 where high yield strength deformed bars conforming to I.S. 1139 or I.S. 1786 are used.

If the depth of the web exceeds 90 cms. longitutudinal reinforcement having a total area atleast equal to 10% of the main tension steel area shall be placed near the faces of the web and distributed in the zone of flexural tension with a spacing not more than 30 cms. or the width of the web whichever is less.

When the specified strength of concrete in columns exceeds that specified for the floor system by more than 40%, transmission of load shall be provided as follows:-

Concrete of the strength specified for the column shall be placed in the floor for an area 4 times the column areas about the column well integrated into the floor concrete and cast monolithically.

Reinorcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the effective depth of the member or 12 bar diameters whichever is greater except at supports of simply supproted beams and at free end of cantilever.

At least 1/3rd the positive moment reinforcement for simply supported members and 1/4th the positive moment reinforcement in continuous members shall extend along the same face of the member into the support and in beams at least 15 cms.

When a flexural member is part of the lateral load resisting system, the positive reinforcement required to be extended into the support as in preceeding paragraph shall be anchored to develop the yield stress in tension at the face of the support.

Atleast 1/3rd the total reinforcement provided for negative moment at the support shall have an embedment length beyond the point of inflexion not less than the effective depth of the member, 12 times the diameter of the bar or 1/16 of the clear span whichever is greater.

Construction joints in floors shall be located near the middle of the spans of slabs, beams or girders, unless a beam intersects a girder at this point, in which case the joints in the girders shall be offset a distance equal to twice the width of the beam. Provision shall be made for transfer of shear and other forces through the construction joints.

7.3 Design of slabs

In a beam and slab type construction of a floor, the slab is continuous over the beams or built into them. For a continuous slab, the effective span is taken equal to the clear span, if the width of the support is not less than 1/12th of the clear span of the slab nor less than 600 mm. If the width of the support is less than the above limit, the effective span is considered as the distance between the centres of the support or the clear distance between the supports plus the effective depth of the slab , whichever is smaller. In the case of end span with one end free and the other continuous the effective span is taken equal to the clear span plus half the effective depth of the slab.

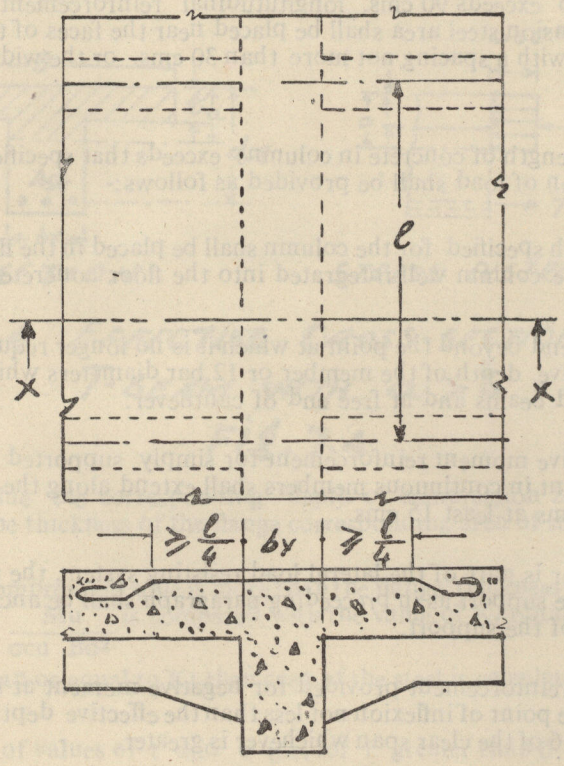
Slabs may be designed to span in the short direction alone when the effective span in the long direction is more than twice the effective span in the short direction. Bending moments at the critical sections are determined for the following arrangements of live load.

(a) Any two adjascent spans loaded and all other spans unloaded.

(b) Alternate spans loaded and all other spans unloaded.

The bending moments in the slab at the various sections are obtained from two cycle moment distribution. However when the L. L is less than the dead load the bending moments and the shear forces may approximately be obtained as in Para 3.2.3. and 3.2.4 of the Manual on Planning and Design of Reinforced Concrete Multistoreyed Buildings Vol. I.

Where the main reinforcement of a slab, which is considered as the flange of a T or L beam is parralel to the beam, transverse reinforcement is to be provided near the top surface of the slab and the quantity of this transverse reinforcement shall be calculated from the moment-co-effiecients for long span as per Annexure 3.3 referred in para 3.2.4.3 of the Manual. This transverse reinforcement is to be provided for a length as shown in the Fig. 7·5.



SECTION XX

Fig. 7.5

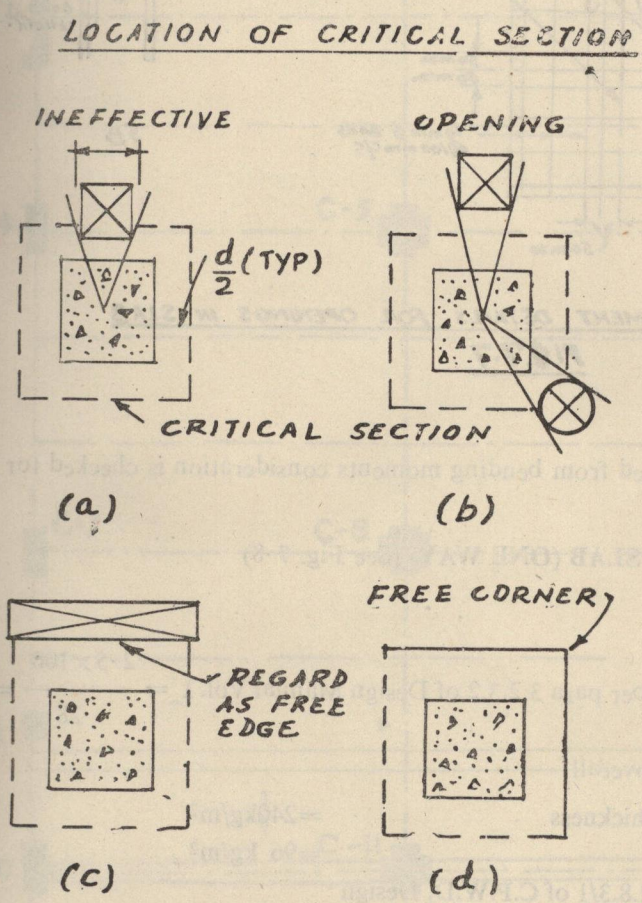
When the slab is designed on the basis of ultimate load theory the minimum reinforcement in the main span shall not be less than 8 mm diameter and the maximum spacing shall be limited to 20 times the diameter of the bar or 20cms whichever is less. The spacing shall also be not more than 3 times the effective depth. The minimum reinforcement in either direction shall be not less than 0.15 per cent of the gross cross-sectional area of the concrete when plain mild steel bars are used and 0·12 percent when high strength deformed bars are used. The maximum spacing for transverse reinforcement should not exceed 5 times the effective depth, 30 times dia of bar or 30 cms whichever is least.

Slabs spaning in two directions at right angles have to be designed as per Para 3.2.4 of the Manual on Planning and Design of Reinforced Concrete Multistoreyed Buildings. Vol. I.

Where a slab is built into the end beam and is constructed monolithic with the beam, provision is to be made for the negative moments that may occur in the slab at the support. The negative moments depend on the degree of the fixity afforded to the edge of the slab. In general this negative moment may be taken as 2/3 rd of the moments at the mid span of the slab. Where a slab is built into a brick or masonry wall which develops partial restraint the negative moment in the slab at the face of the support may be taken as $\frac{Wl^2}{24}$ (where W is the uniformerly distributed load per unit area of the slab).

7.3.1. Opening in Slab

If openings in slabs are located at a distance less than 10 times the thickness of the slab from the reaction or from concentrated load, the part of periphery of the critical section which is enclosed by radial projections of the opening to the centroid of the loading area is considered ineffective (Refer Fig. 7·6)

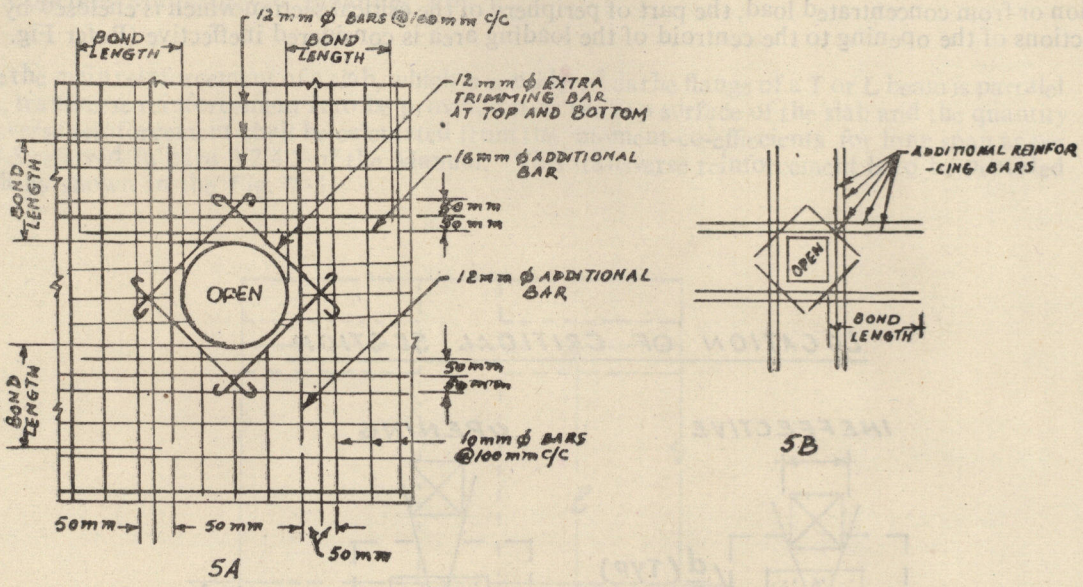


EFFECT OF OPENINGS & FREE EDGE.

FIG. 7.6

When openings are placed in the area within the middle half of the span in each direction, the quantity of reinforcement cut by the opening in either direction is provided extra. This quantity is distributed equally near the two edges of opening in each direction. Additional bars may also be provided at the corners to prevent splitting.

The detailing of the reinforcement around the opening is shown in the Fig. 7·7.



TYPICAL REINFORCEMENT DETAILS FOR OPENINGS IN SLAB

FIG.7.7

The reinforcement provided from bending moments consideration is checked for the adequacy of bond.

DESIGN OF CONTINUOUS SLAB (ONE WAY) (See Fig. 7·8)

LOADING

Min. thickness of slab as per para 3.2.3.2 of Design Manual Vol. I $= \frac{2 \cdot 5 \times 100}{28} = 8 \cdot 93$cms

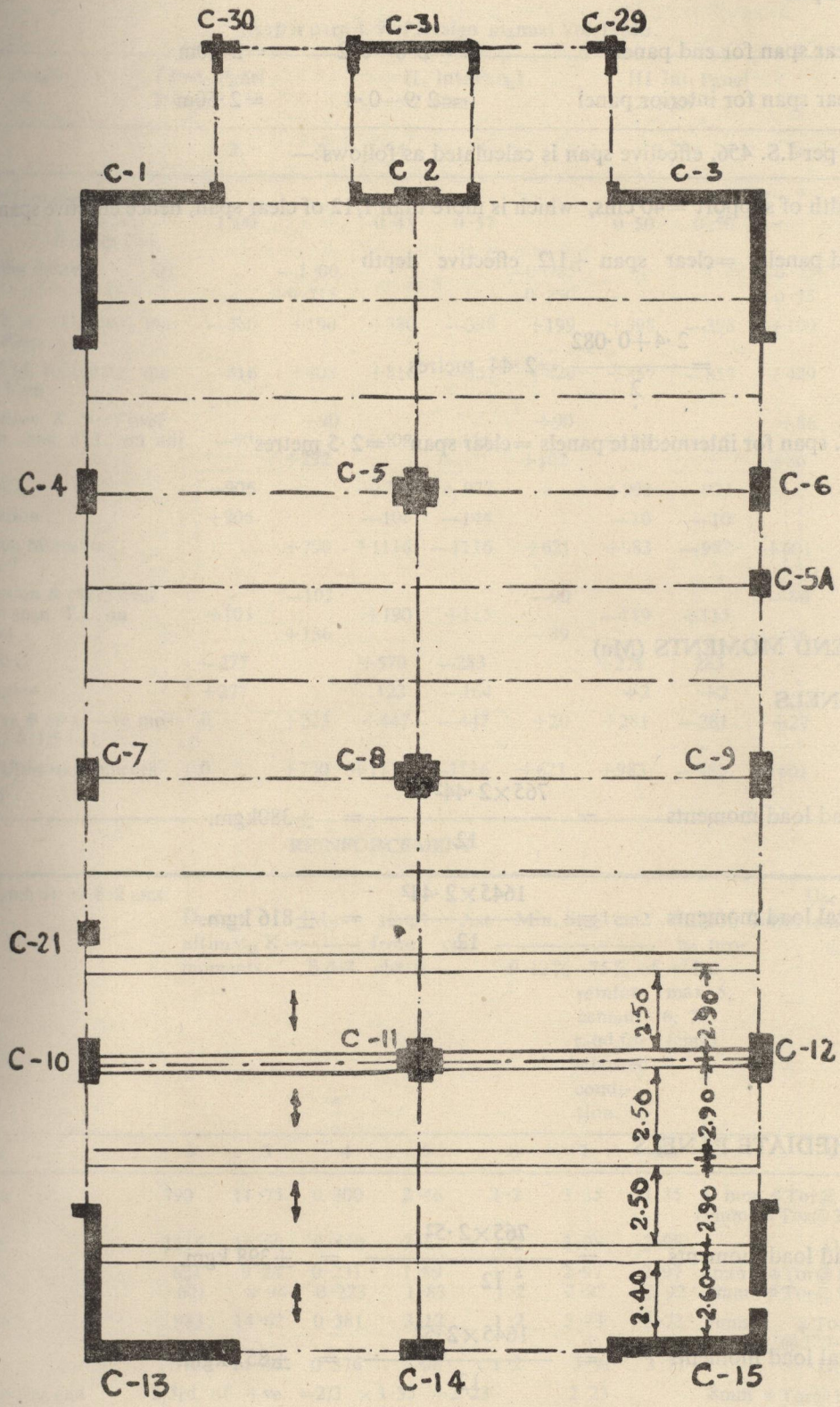
Assume 10 cm thickness overall

Self wt. of slab (10 cms) thickness	=240kg/m²
Floor finish 4 cm thick	=96 kg/m²
False ceiling (vide clause 2.8.3/1 of C.P.W.D. Design Manual Vol. I)	= 40kg/m²
Partition walls (vide clause 2.2.1 of C.P.W.D. Design Manual Vol. I)	=100 kg/m²
Ceiling finish	=24 kg/m² (Assumed)
A/C ducts	=10 kg/m² (Assumed)
Total dead load	=510 kg/m²

LIVE LOAD

Loading class 250, as per appendix 2·1 of C.P.W.D. Design Manual Vol. I (400 kg/m² taken due to non-provision of separate storage facility) =400 kg/m2.

Ultimate loads	=510×1·5	=765kg/m2 (D.L.)
and	=400×2·2	=880kg/m2 (L.L.)
Total ultimate load		1645 kg/m2


C-30
C-31
C-29
C-1
C 2
C-3
C-4
C-5
C-6
C-5A
C-7
C-8
C-9
C-21
C-10
C-11
C-12
C-13
C-14
C-15
2.50
2.90
2.50
2.90
2.50
2.90
2.40
2.60

PLAN

Fig. 7.8

Effective Span

Clear span for end panel $=2\cdot6-0\cdot2$ $=2\cdot40$m

Clear span for interior panel $=2\cdot9-0\cdot4$ $=2\cdot50$m

As per I.S. 456, effective span is calculated as follows:—

Width of support =40 cms, which is more than 1/12 of clear span, hence effective span for

End panels =clear span +1/2 effective depth

$$=\frac{2\cdot4+0\cdot082}{2}=2\cdot44 \text{ metres}$$

Eff. span for intermediate panels =clear span $=2\cdot5$ metres

FIXED END MOMENTS (Mu)

END PANELS

$$\text{Dead load moments} = \frac{765\times2\cdot44^2}{12} = \pm380\text{kgm.}$$

$$\text{Total load moments} = \frac{1645\times2\cdot44^2}{12} = \pm816 \text{ kgm.}$$

INTERMEDIATE PANELS

$$\text{Dead load moments} = \frac{765\times2\cdot5^2}{12} = \pm398 \text{ kgm.}$$

$$\text{Total load moments} = \frac{1645\times2\cdot5^2}{12} = \pm857 \text{ kgm.}$$

Max. ultimate moment =1116 kgms. (After moment distribution) Min. effective depth required.

$$=\sqrt{\frac{1116\times100}{\cdot185\times150\times100}} = 6\cdot34 \text{ cms.}$$

Computation of slab moments by two cycle moment distribution

(As per para 4·9 of Design manual Vol. I) Mu.

1. Span of slab	I End. Panel		II. Int. Panel			III Int. Panel				IV
2. K	2·44m.		2·50m			2·50m				
1	2	3	4	5	6	7	8	9	10	11
3. Distribution factor Top Col.										
Beam	1·00		0·43	0·57		0·50	0·50		0·50	0·50
Bottom Col.										
4. Mid point factor Q1		—1·00			—0·785			—0·75		
Qr		+0·715			+0·750			+0·75		
5. D.L.F.E.M. (Ultimate moments) Kgm.	—380	+190	+380	—398	+199	+398	—398	+199	+398	—398
6. T.L.F.E.M. (Ultimate moments) Kgm	—816	+408	+816	—857	+429	+857	—857	+429	+857	—857
7. Distribution & carry over T.L. on span, D.L. on adj. span.	—90	+90 +292	+408	—115	+90 +102	+136	—115	+86 +86	+115	—115
8. Add 6 &7	—906		+1224	—972		+993	—972		+972	—972
9. Distribution	+906		—108	—144		—10	—10			..
10. Maximum Moments 8+9/6+7		+790	+1116	—1116	+621	+983	—982	+601	+972	—972
11. Distribution & carry over D. L. on span, T.L. on adj. span	+103	—103 +136	+190	+115	—90 —89	—119	+115	—86 —86	—115	+115
12. Add 5 & 11	—277		+570	—283		+279	283		+283	—283
13. Distribution	+277		..123	—164		+2	+2		..	..
14. Min. +ve & max. —ve moments 12+13/5+11	0	+223	+447	—447	+20	+281	—281	+27	+283	—283
15. Design Ultimate moments 10 or 14	0	+790	+1116	—1116	+621	+983	—982	+601	+972	—972

REINFORCEMENT

Effective depth d1 = 8·2 cms.

Use :— M—150

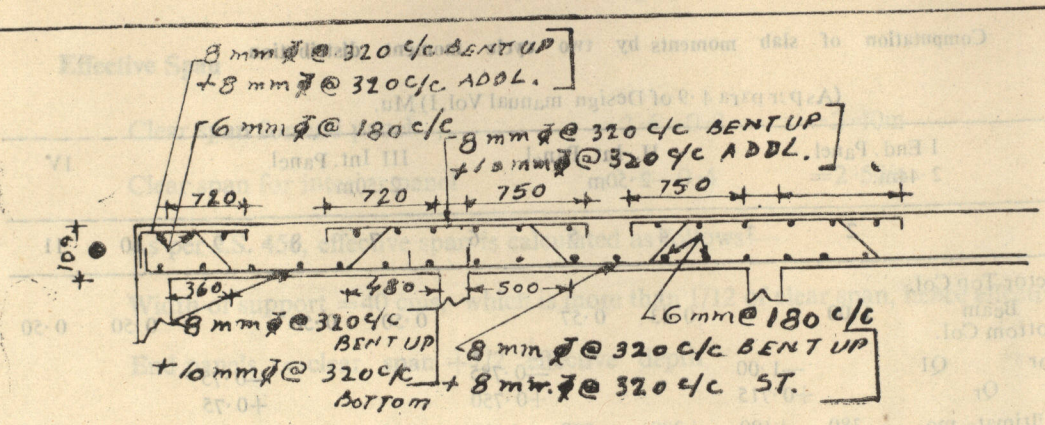
1	Design ultimate moments	K = Mu / B.d1²	100pt from charts	Ast cm2	Min. Steel cm2: 0·12%	Min. Steel cm2: 75% of reinforcement reqd.for balanced condition.	Steel to be provided max. 5, 6, 7 (cm2)	9 Rinforcement perovided @mm c/c
+ve end span	790	11·75	0·300	2·46	1·2	3·35	3·35	8 mm ϕTor@ 320+ 10mm ϕ Tor@320=4·02cm²
—ve end span	1116	16·60	0·440	3·61	1·2	3·98	3·98	Do.
+ve Int. span	621	9·24	0·231	1·89	1·2	2·97	2·97	8mm ϕTor@160=3·14cm²
	601	8·94	0·223	1·83	1·2	2·92	2·92	8mm ϕTor@160=3·14cm²
—ve Int. span	983	14·62	0·381	3·12	1·2	3·73	3·73	8mm ϕTor320+10mm² @320=4·02cm²
	972	14·46	0·376	3·08	1·2	3·71	3·71	Do.
—ve end span free end	2/3rd of +ve =2/3 ×3·35 =2·23					2·23		8mm ϕ Tor@160=3·14cm²

Distribution steel @0·12% of gross cross sectional area of concrete = $\frac{0·12 \times 10 \times 100}{100}$ = 1·2cm²

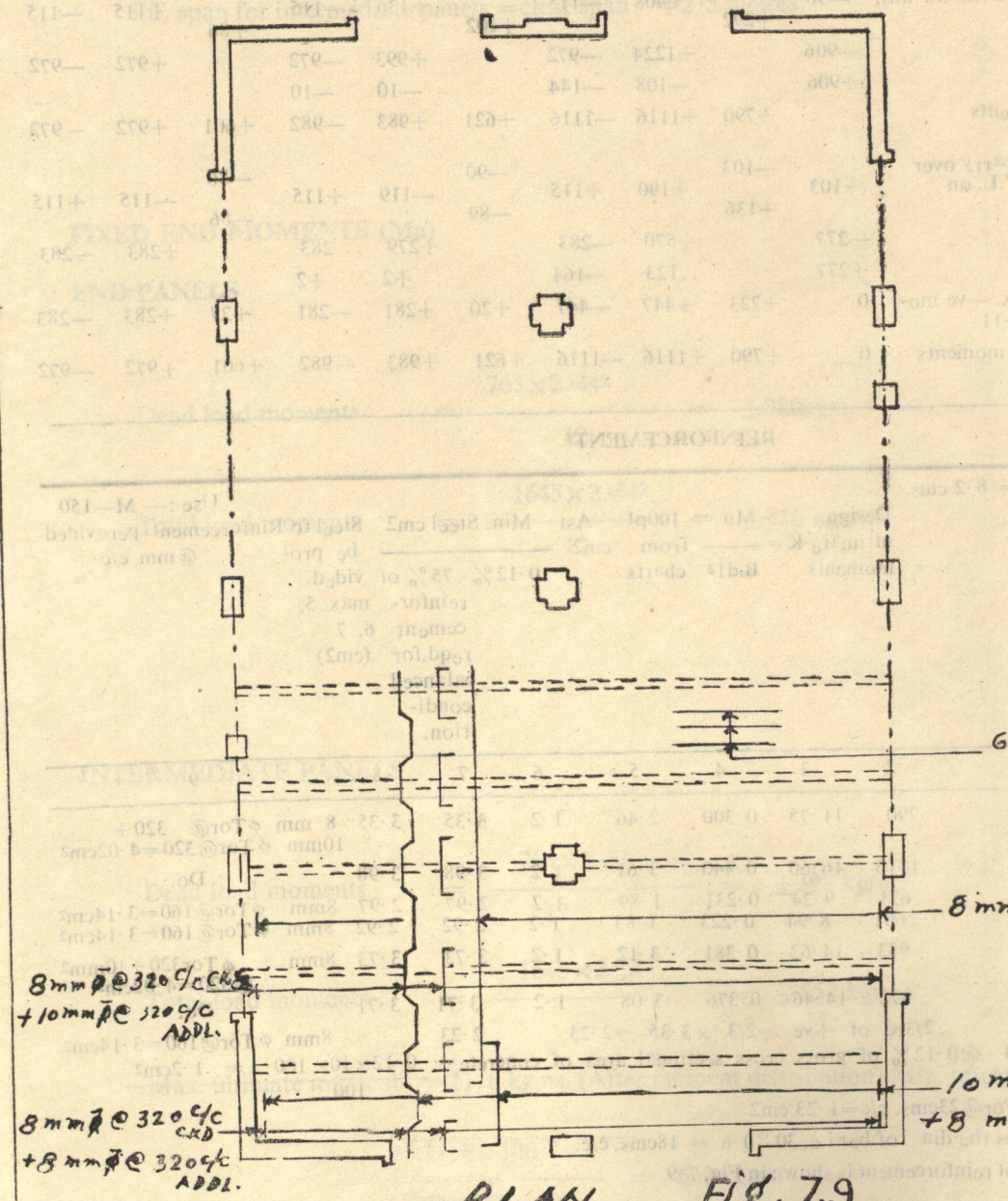
Provide 6 mm ϕ Tor@23cms. c/c=1·23 cm2

Limited to 30 times the dia of bar i.e. 30×0·6 = 18cms. c/c.

The arrangement of reinforcement is shown in Fig. 7·9.



L. SECTION



PLAN

Fig. 7.9

7.3.2 Design of two way slabs

The two way slabs are designed as per annexure 3·3 of the Manual on Planning & Design of Reinforced concrete multistoreyed buildings vol 1. In the edge strips the reinforcement parallel to the adjascent edges of the slab shall be the minimum reinforcement as specified in para 7·2 above.

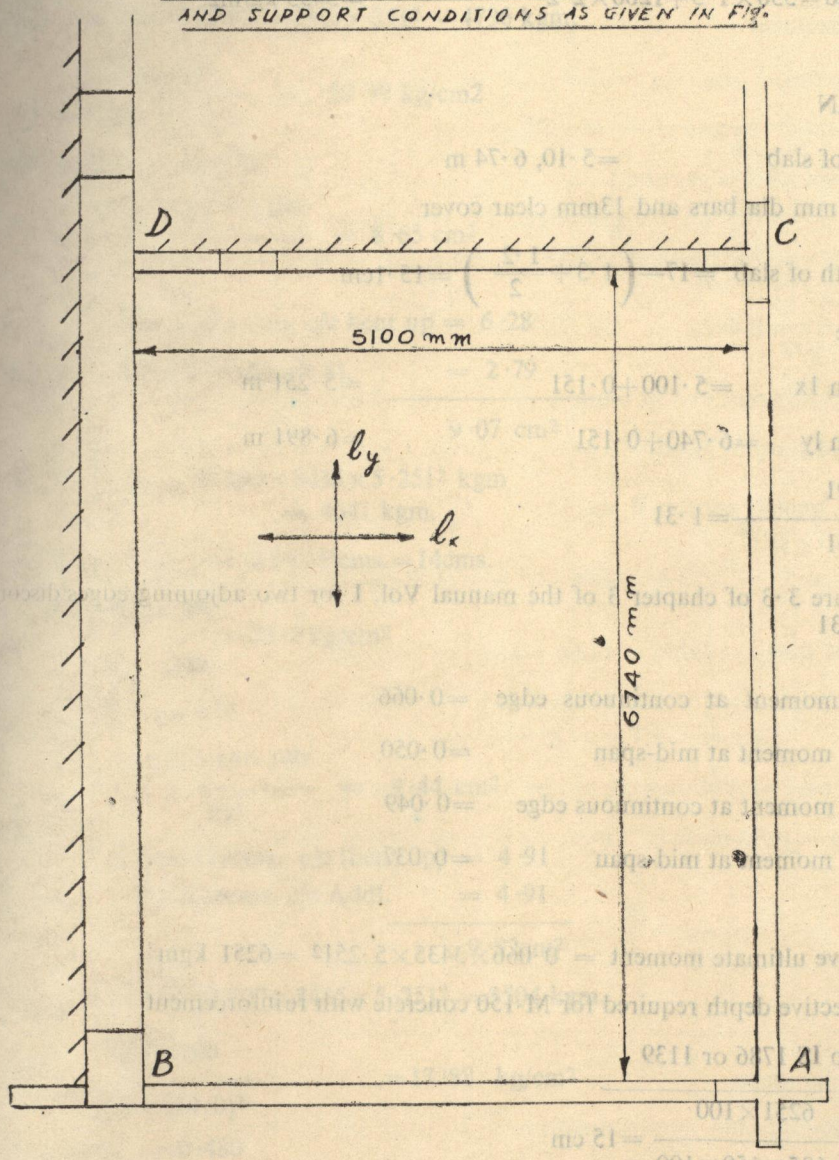
Torsion reinforcement is required to be provided at corners of the slab over the edges where the slab is discontinuous. At corners contained by edges over which the slab is discontinuous in both directions, the torsion reinforcement consists of two layers of bars placed parallel to the sides of slab at top & bottom of slab extending in both the directions for a distance equal to one fifth of shorter span. The area of the bars in each of the four layers, per unit width of slab, is three quarters of the area required for positive moment in the slab.

At corners contained by edges over only one of which the slab is discontinuous, the torsional reinforcement may be reduced to one half of that required as per above paragraph.

Any reinforcement provided for any other purpose may be included as part of reinforcement required to resist torsion at corners contained by discontinuous edge or edges.

EXAMPLE:

DESIGN TWO-WAY SLAB FOR THE DIMENSION AND SUPPORT CONDITIONS AS GIVEN IN Fig.



PLAN OF A WEATHER MAKER ROOM IN A M.S. BUILDING

Fig. 7.10

LOADING

Self wt. of slab 17 cm (assumed) thick	=408 kg/m2
Flooring 4 cm thick=4×24	=96 kg/m2
Ceiling finish	=26 kg/m2 (Assumed)
Total dead load	530 kg/m2

LIVE LOAD

As per para 2.8.3/2 of the Manual on Planning and design of Reinforced multistoreyed buildings Vol. I

live load for weather makers room =1200 kg/m2

Ultimate load=530×1·5+1200×2·2 =3435 kg/m2

EFFECTIVE SPAN

Clear spans of slab =5·10, 6·74 m

adopting 12 mm dia bars and 13mm clear cover

effective depth of slab $=17-\left(1\cdot3+\frac{1\cdot2}{2}\right)=15\cdot1$cm

Hence

Effective span 1x =5·100+0·151 =5·251 m

Effective span ly =6·740+0·151 =6·891 m

$$\frac{ly}{lx}=\frac{6\cdot891}{5\cdot251}=1\cdot31$$

From annexure 3·3 of chapter 3 of the manual Vol. I for two adjoining edges discontinuous and ly/lx =1·31

z'x for —ve moment at continuous edge =0·066

z'x for +ve moment at mid-span =0·050

z'y for —ve moment at continuous edge =0·049

z'y for +ve moment at mid-span =0·037

Maximum —ve ultimate moment = $0\cdot066\times3435\times5\cdot251^2$ =6251 kgm

Minimum effective depth required for M-150 concrete with reinforcement conforming to IS 1786 or 1139

$$\sqrt{\frac{6251\times100}{\cdot185\times150\times100}}=15 \text{ cm}$$

Actual effective depth provided =15·1 cm

Hence O.K.

Reinforcement :

Short span —ve Mu = 6251 kgm

$$K=\frac{6251\times100}{100\times(15\cdot1)^2}=27\cdot4\ kg/m2$$

For ultimate strength design of member 100 pt= 0·816

$$\text{From table Ast}=\frac{0\cdot816\times15\cdot1\times100}{100}=12\cdot32\ cm2$$

Provide 12 mm ϕ Tor @18cm c/c bent up =6·28
and 12 mm ϕTor @18cm c/c Addl. =6·28

12·56 cm²

Short span +ve Mu=0·050×3435×5·251² =4736 kgm.

$$K=\frac{4736\times100}{100\times(15\cdot1)^2}=20\cdot77\ kg/cm2$$

From table 100 pt =0·573

$$Ast=\frac{0\cdot573\times15\cdot1\times100}{100}=8\cdot65\ cm^2$$

Provide 12 mm ϕ Tor@18 cms. c/c bent up = 6·28
and 8 mm ϕ Tor@18cms c/c St. = 2·79

9·07 cm²

long span —ve Mu = 0·049×3435×5·251² kgm
= 4641 kgm.

Eff. depth =17·0—(1·3+1·2+0·5)cms.=14cms.

$$K=\frac{4641\times100}{100\times(14)^2}=23\cdot7\ kg/cm^2$$

100 pt from table = 0·674

$$Ast=\frac{0\cdot674\times14\times100}{100}=9\cdot44\ cm^2$$

Provide 10 mm ϕ Tor@16cms. c/c (bent up) = 4·91
and 10 mm ϕ Tor@16cms. c/c Addl. = 4·91

9·82cm²

Long span +ve Mu = 0·037×3435×5·251² =3504 kgm.

$$K=\frac{3504\times100}{100\times(14\cdot0)^2}=17\cdot88\ kg/cm^2$$

100 pt from table =0·480

$$Ast=\frac{0\cdot480\times14\cdot0\times100}{100}=6\cdot72\ cm^2$$

Provide 10 mm ϕ Tor @16 cms c/c bent up $= 4 \cdot 91$

and 8 mm ϕ Tor 16cms c/c St. $= 3 \cdot 14$

$8 \cdot 05 \text{ cm}^2$

At free ends there is a monolithic connection between the slab and the supporting beam. As per para 7 ·2, reinforcement required at the beam edges in short span direction $= 8 \cdot 65 \times \frac{2}{3} = 5 \cdot 77 \text{ cm}^2$

12 mm ϕ Tor @ 18 cms c/c bent up $= 6 \cdot 28 \text{ cm}^2$ (Provided)

Reinforcement required at beam edge in long span direction

$$= 6 \cdot 72 \times \frac{2}{3} = 4 \cdot 48 \text{ cm}^2$$

10 mm ϕ Tor@ 16cms c/c bent up $= 4 \cdot 91 \text{ cm}^2$ (Provided)

Torsional reinforcement

Torsional reinforcement shall be provided as per para 7.3.2

Area of maximum +ve steel=8. 65cm^2

Area of torsional steel at corner A

$= 0 \cdot 75 \times 8 \cdot 65 = 6 \cdot 5 \text{ cm}^2$/layer/100 cm width

for 17 cm thick slab minimum steel available at the corner shall be

$$= \frac{0 \cdot 12 \times 17}{100} \times 100 = 2 \cdot 04 \text{ cm}^2$$

Hence additional steel required at corners A $= 6 \cdot 50 - 2 \cdot 04 = 4 \cdot 46 \text{ cm}^2$

Provide at top and bottom mesh of 8 mm ϕ Tor@ 11cms. c/c of length 1x/5 and width 1x/5

Area of torsional steel at corners B and C

$$= \frac{1 \times 6 \cdot 5}{2} = 3 \cdot 25 \text{ cm}^2\text{/layer/100 cm width}$$

Minimum area of steel available at these places $= 2 \cdot 04 \text{ cm}^2$

Hence additional steel required at corners B & C $= 3 \cdot 25 - 2 \cdot 04 = 1 \cdot 21 \text{ cm}^2$

(8 mm ϕ Tor@ 40 cms. c/c)

Limit spacing of bars to 30 times the dia of bars i.e. $\cdot 8 \times 30 = 24$ cms.

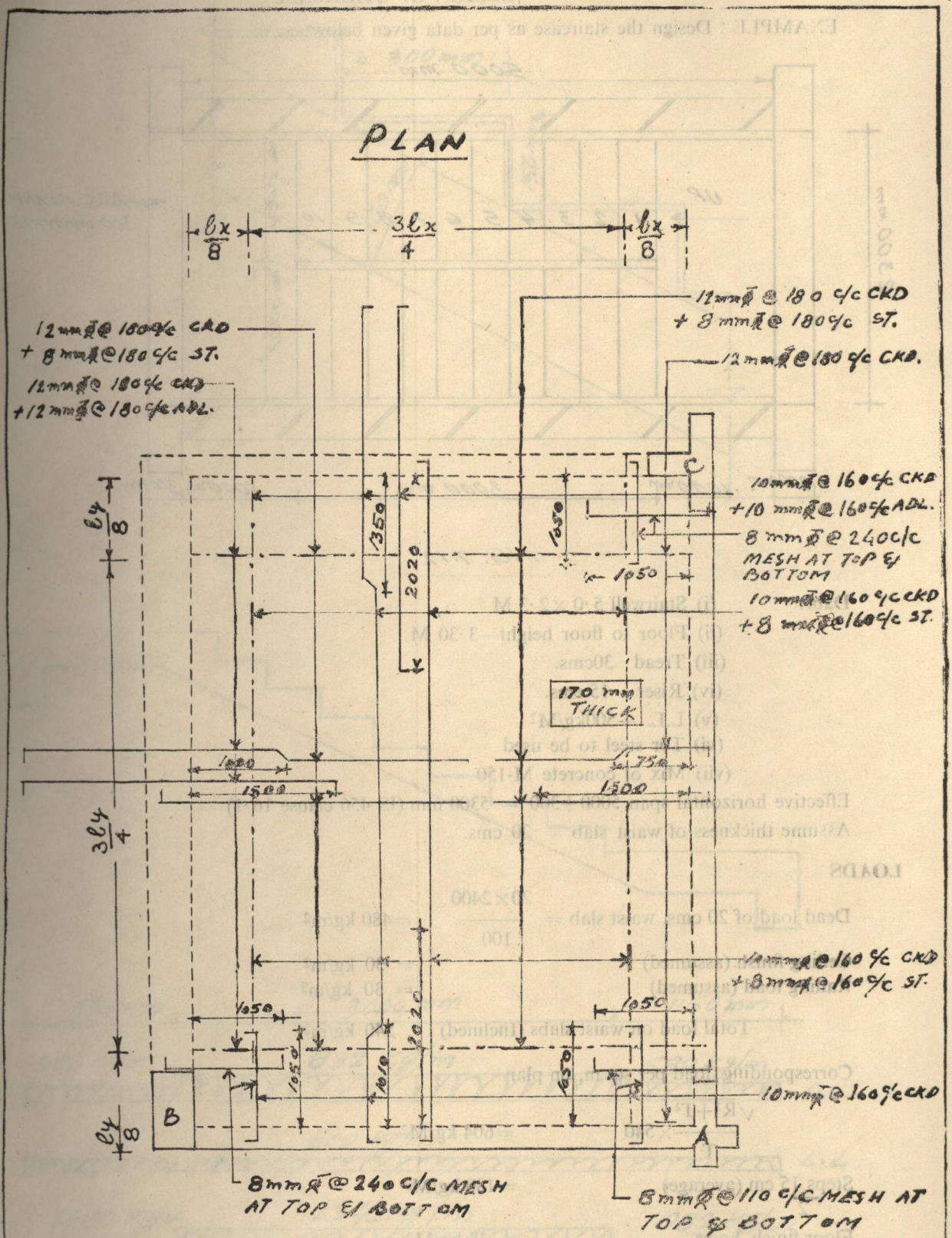
Provide a top and bottom mesh of 8 mm ϕ Tor @ 24cms. c/c for length

$\frac{lx}{5}$ and width $\frac{1x}{5}$

Reinforcemet in edge strips

As per para 7.2.2 minimum reinforcement need be provided in all edge strips of width 1x/8 and 1y/8

The arrangement of reinforcement is shown in Fig. 7 ·11.



NOTE: REINFORCEMENT IN EDGE STRIPS IS NOT SHOWN. FOR EACH EDGE STRIP OF WIDTH $\frac{l_x}{3}$ and $\frac{l_y}{8}$ PROVIDE MIN. REINFORCEMENT AS PER PARA 7.2.2.

Fig. 7.11

DESIGN OF STAIRCASE

EXAMPLE : Design the staircase as per data given below:

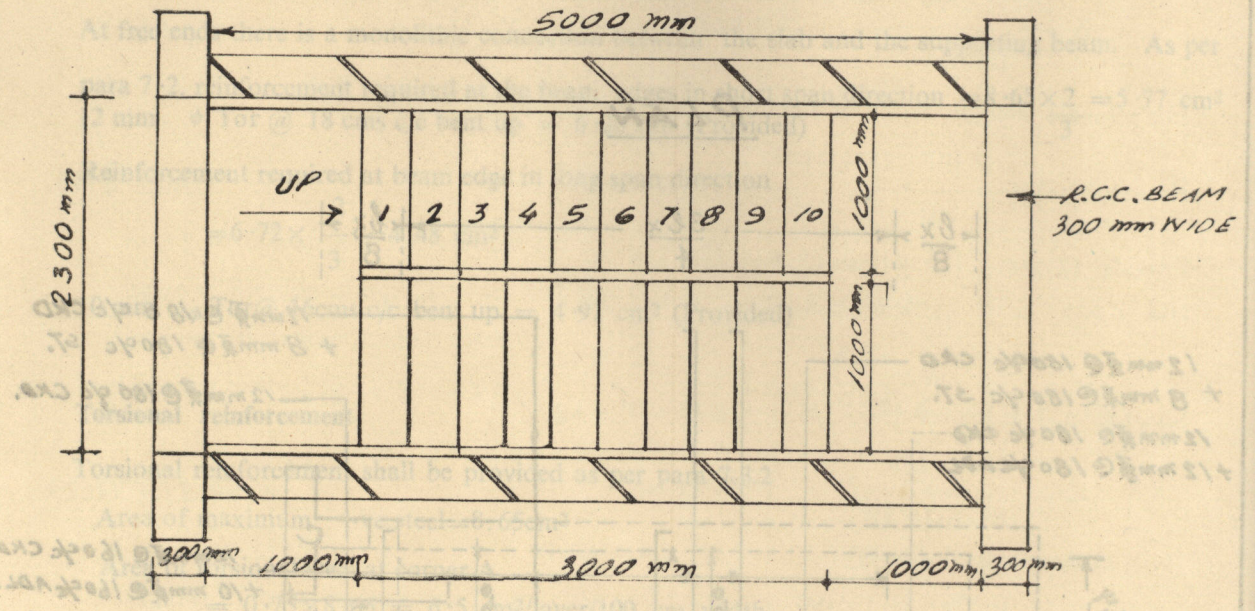


FIG. 7.12

Data:
(i) Stairwell 5·0 × 2·3 M
(ii) Floor to floor height—3·30 M
(iii) Tread 30cms.
(iv) Riser —15 cms.
(v) L.L. —500kg/M²
(vi) Tor steel to be used
(vii) Mix of concrete M-150

Effective horizontal span 5000+300 = 5300 mm (IS 456 clause 16·1)
Assume thickness of waist slab = 20 cms.

LOADS

Dead load of 20 cms. waist slab = $\frac{20 \times 2400}{100}$ = 480 kg/m²

Ceiling finish (assumed) = 30 kg/m²
Railing load (assumed) = 30 kg/m²

Total load on waist slabs (Inclined) 540 kg/m²

Corresponding load per sq. m. on plan

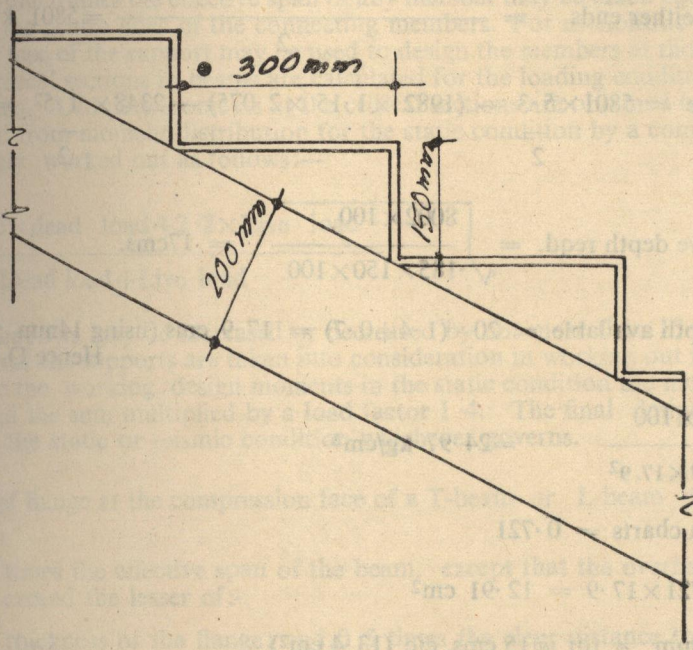
$\frac{\sqrt{R^2+T^2}}{T} \times 540$ = 604 kg/M²

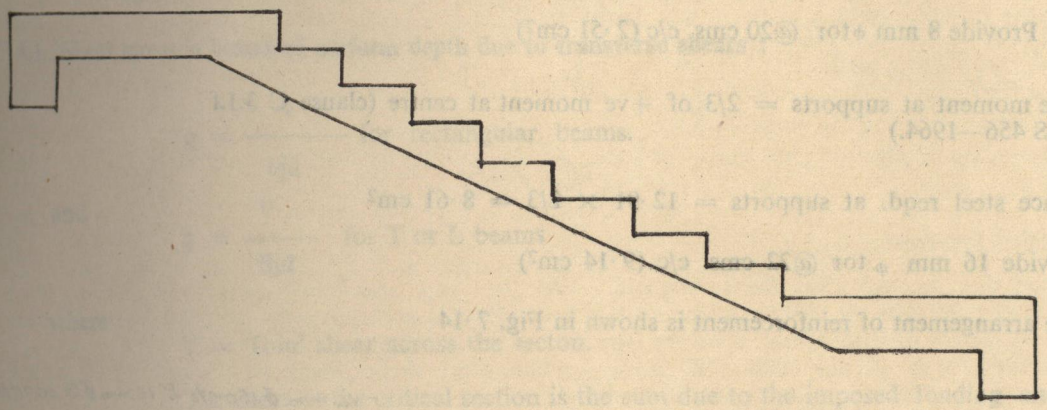
Steps $\frac{15}{2}$ cm (average) = 180 kg/M²

Floor finish 2 cms. = 48 kg/M²

832 kg/M²

Load on landing = 540+48 = 588 kg/m²
D.L., L.L. & U.L. diagrams shall be as shown (see fig. 7.13)


300 mm
150 mm
200 mm



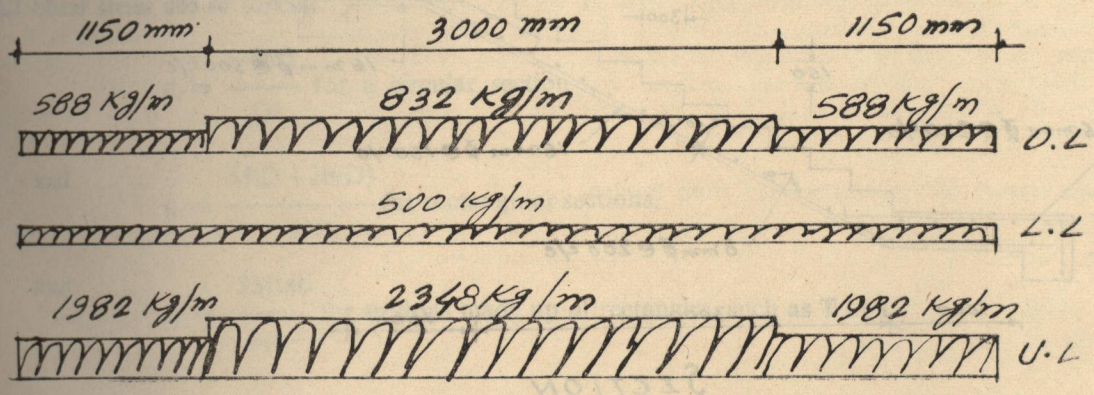

1150 mm
3000 mm
1150 mm
588 Kg/m
832 Kg/m
588 Kg/m
D.L
500 Kg/m
L.L.
1982 Kg/m
2348 Kg/m
1982 Kg/m
U.L

FIG. 7.13

Reaction at either ends $= \dfrac{1982 \times 1 \cdot 15 \times 2 + 2348 \times 3 \cdot 0}{2} = 5801$ kg.

Mu at centre $= 5801 \times 5 \cdot 3 - \dfrac{(1982 \times 1 \cdot 15 \times 2 \cdot 075)}{2} - \dfrac{2348 \times 1 \cdot 5^2}{2} = 8002$ kg.m.

Min. effective depth reqd. $= \sqrt{\dfrac{8002 \times 100}{\cdot 185 \times 150 \times 100}} = 17$cms.

Effective depth available $= 20 - (1 \cdot 4 + 0 \cdot 7) = 17 \cdot 9$ cms (using 14mm ϕ tor bars)
Hence O.K.

$$K = \frac{8002 \times 100}{100 \times 17.9^2} = 24 \cdot 97 \text{ kg/cm}^2$$

100 pt from charts $= 0 \cdot 721$

Ast $= \cdot 721 \times 17 \cdot 9 = 12 \cdot 91$ cm^2

Provide 16 mm ϕ tor @15 cms. c/c (13 ·4 cm^2)

Distribution steel @0 ·12% $= \dfrac{\cdot 12 \times 20}{100} = 2 \cdot 4$ cm^2

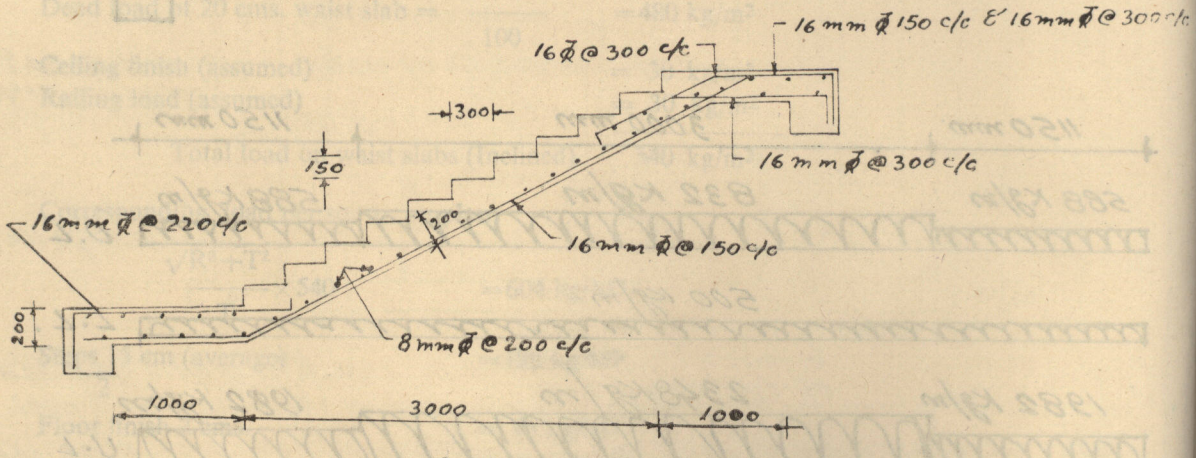
Provide 8 mm ϕtor @20 cms. c/c (2 ·51 cm^2)

—ve moment at supports = 2/3 of +ve moment at centre (clause C 3.1.f of IS 456—1964.)

Hence steel reqd. at supports $= 12 \cdot 91 \times 2/3 = 8 \cdot 61$ cm^2

Provide 16 mm ϕ tor @22 cms. c/c (9 ·14 cm^2)

The arrangement of reinforcement is shown in Fig. 7 ·14



SECTION

Fig. 7.14

7.4 Design of Beams :

In the case of monolithic frames the effective span of any member may be taken equal to the distance between inter-sections of the centre lines of the connecting members. For monolithic construction, the moments computed at the face of the support may be used to design the members at those sections. The bending moment at the critical sections in beams are calculated for the loading condition similar to that indicated for design of slabs. Ultimate moments at the critical sections are obtained by multiplying the working moments obtained from moment distribution for the static condition by a combined load factor which may be approximately worked out as follows:—

$$\text{L.F.} = \frac{1{\cdot}5 \times \text{dead load} + 2{\cdot}2 \times \text{Live load}}{\text{Dead load} + \text{Live load}}$$

The moments over supports may be increased or decreased by not more than 15 percent, provided these modified moments over the supports are taken into consideration in working out the corresponding moments in the spans. To the working design moments in the static condition the moment due to seismic condition is added and the sum multiplied by a load factor 1 ·4. The final design moment for the section is then adopted in the static or seismic condition whichever governs.

The effective width of flange at the compression face of a T-beam or L-beam may be taken as follows:

(a) **T-sections** : 0 ·25 times the effective span of the beam, except that the overhanging part of the flange shall not exceed the lesser of:-

8 times the thickness of the flange; and 0 ·5 times the clear distance to the next parallel supporting member.

(b) **L-Sections** : The width of the web plus 0 ·083 times the effective span of the beam, except that the overhanging part of the flange shall not exceed the lesser of 6 times the thickness of the flange; and 0 ·5 times the clear distance to the next parallel supporting member.

7.4.1. Shear stress in beams of uniform depth due to transverse shears :

$$q = \frac{V}{bjd} \quad \text{for rectangular beams.}$$

and

$$q = \frac{V}{Bjd} \quad \text{for T or L beams}$$

where

V = Total shear across the secton.

The shear at the critical section is the sum due to the imposed loading and due to the moments at both ends under static and seismic conditions.

7.4.2 Shear stress due to torsion

$$q = \frac{1{\cdot}6Mt}{D^3} \quad \text{for a circular section}$$

and

$$q = \frac{Mt(3+2b/D)}{b^2D} \quad \text{for rectangular sections.}$$

and

$$q = \frac{3Mtxti}{b_1t_1^3} \quad \text{for sections made up of rectangles, such as T or L}$$

Where ti is t_1 or t_2 according to the section under consideration.

SHEAR STRESS IN MEMBERS DUE TO TORSION

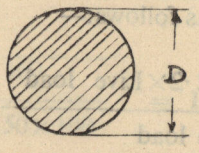
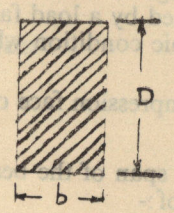
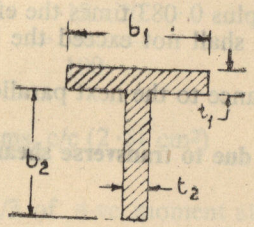
DESCRIPTION OF SECTION	SHAPE	MAXIMUM SHEAR STRESS
1) CIRCLE		$\frac{16 M_t}{\pi D^3}$
2) RECTANGLE		$\frac{M_t(3+2b/D)}{b^2 D}$
3) SECTION MADE UP OF RECTANGLES, SUCH AS 'T' OR 'L'		$\frac{3 M_t t_i}{\Sigma b_i t_i^3}$ WHERE t_i IS t_1 OR t_2 ACCORDING TO THE SECTION UNDER CONSIDERATION.

Fig. 7.15

7·4·3 Design for shear

When the sum of the shear stresses, due to the transverse shear and the torsion, is less than the permissible shear stress in the working condition, only nominal shear reinforcement is provided in the entire span of the beam in the form of stirrups at a pitch not more than the lever arm of the beam. If, however, the total shear stress is more than the permissible shear stress, shear reinforcement is provided to resist the whole shearing force due to transverse shear and the shear resulting from torsion. In no case, however, the total shear stress is allowed to be more than 4 times the permissible shear stress. To resist shear, the stirrups are to be spaced not farther apart than jd or 60 cms. whichever is less (where jd=depth of leverarm). Tensile reinforcement, which is inclined and carried throughout the depth of the beam, is also considered as shear reinforcement, provided it is anchored sufficiently. The resistance to shear at a section due to inclined bars is calculated on the assumption that these bars serve as tension members of one or more single system of lattice girders with the concrete forming the compression members.

Reinforcement for shear resulting from torsion

The reinforcement to resist shear resulting from torsional moment is calculated as follows.

For rectangular sections, the area of the two vertical legs in a closed stirrup shall be calculated as follows :—

$$A_w = \frac{M_t . S}{0 \cdot 8 \times \sigma_{ss} X_1 Y_1}$$

SHEAR STRESS IN MEMBERS DUE TO TORSION

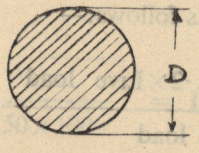
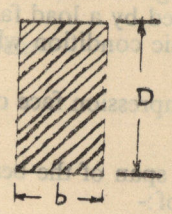
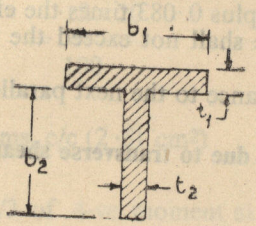
DESCRIPTION OF SECTION	SHAPE	MAXIMUM SHEAR STRESS
1) CIRCLE		$\frac{16 M_t}{\pi D^3}$
2) RECTANGLE		$\frac{M_t(3+2b/D)}{b^2 D}$
3) SECTION MADE UP OF RECTANGLES, SUCH AS 'T' OR 'L'		$\frac{3 M_t t_i}{\Sigma b_i t_i^3}$ WHERE t_i IS t_1 OR t_2 ACCORDING TO THE SECTION UNDER CONSIDERATION.

Fig. 7.15

7·4·3 Design for shear

When the sum of the shear stresses, due to the transverse shear and the torsion, is less than the permissible shear stress in the working condition, only nominal shear reinforcement is provided in the entire span of the beam in the form of stirrups at a pitch not more than the lever arm of the beam. If, however, the total shear stress is more than the permissible shear stress, shear reinforcement is provided to resist the whole shearing force due to transverse shear and the shear resulting from torsion. In no case, however, the total shear stress is allowed to be more than 4 times the permissible shear stress. To resist shear, the stirrups are to be spaced not farther apart than jd or 60 cms. whichever is less (where jd=depth of leverarm). Tensile reinforcement, which is inclined and carried throughout the depth of the beam, is also considered as shear reinforcement, provided it is anchored sufficiently. The resistance to shear at a section due to inclined bars is calculated on the assumption that these bars serve as tension members of one or more single system of lattice girders with the concrete forming the compression members.

Reinforcement for shear resulting from torsion

The reinforcement to resist shear resulting from torsional moment is calculated as follows.

For rectangular sections, the area of the two vertical legs in a closed stirrup shall be calculated as follows :—

$$A_w = \frac{M_t . S}{0{\cdot}8 \times \sigma_{ss} X_1 Y_1}$$

LOAD FACTOR $= \dfrac{2391 \times 1 \cdot 5 + 1160 \times 2 \cdot 2}{2391 + 1160} = 1 \cdot 73$

The design working moments are worked out by moment distribution. Maximum and minimum moments at critical section are tabulated for a typical beam.

DESIGN ULTIMATE MOMENTS (Tonnes metre)

		End C4	Mid span	End C5	End C5	Mid span	End C6
1.	(D.L.+L.L.) Moments	—24 ·26	+21 ·60	+41 ·52	—41 ·52	+21 ·60	+24 ·26
2.	E.L. Moments	±57 ·02	+5 ·83	±68 ·68	±68 ·68	+5 ·83	+57 ·02
3.	(D.L.+L.L.)x1 ·73	—41 ·97	+37 ·37	+71 ·83	+71 ·83	+37 ·36	—41 ·97
4.	(D.L.+L.L.+E.L.) x1 ·4	—113 ·79 +45 ·86	—38 ·40	+154 ·28 —38 ·02	—154 ·28 +38 ·02	+38 ·40	+113 ·79 —45 ·86
5.	Design ultimate moment	—113 ·79 +45 ·86	+38 ·40	+154 ·28 —38 ·02	—154 ·28 +38 ·02	+38 ·40	+113 ·79 —45 ·86

Design ultimate moments are worked out for the maximum value of the following conditions;

(i) (D.L.+L.L.)×load factor, under static conditions.

(ii) (D.L.+L.L.+E.L.)×1 ·4 under seismic conditions.

Effective depth of the beam assuming 25 mm cover and 25 mm bars, when two layers are provided.

Using M-150 conc. and tor steel bars of over 20 mm dia

shear capacity of the beam

$= 5 \times 40 \times 0 \cdot 896 \times 93 \cdot 75 = 16800$ kg or 16 ·8 T.

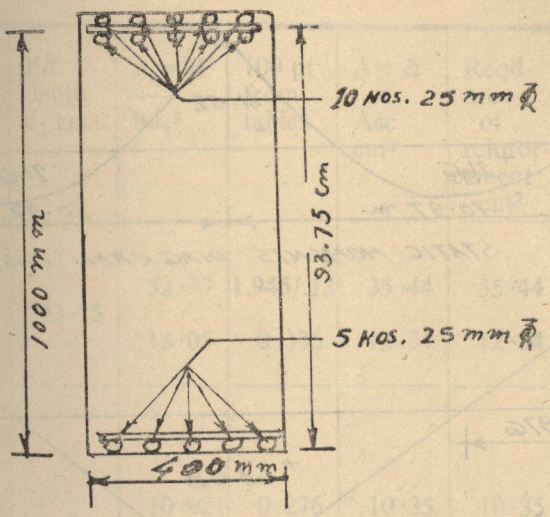


FIG. 7.17

Shear capacity under seismic conditions will be more because of 33% increase in allowable shear stress=16·8×1·33=22·34 T.

$$\frac{57\cdot02}{X} = \frac{68\cdot68}{10\cdot97-X}$$

$$57\cdot02\times10\cdot97 - 57\cdot02X = 68\cdot68X$$

$$125\cdot70X = 57\cdot02\times10\cdot97$$

$$X = \frac{57\cdot02\times10\cdot97}{125\cdot70} = 4\cdot976 \text{ m.}$$

$$Y = 10\cdot97-4\cdot976 = 5\cdot994 \text{ m.}$$

Reqd. at 5·485 m

$$\text{i.e. at centre} = \frac{(5\cdot485-4\cdot970)\times68\cdot68}{5\cdot994} = 5\cdot83$$

Value at P = 21·6+5·83 = 27·43

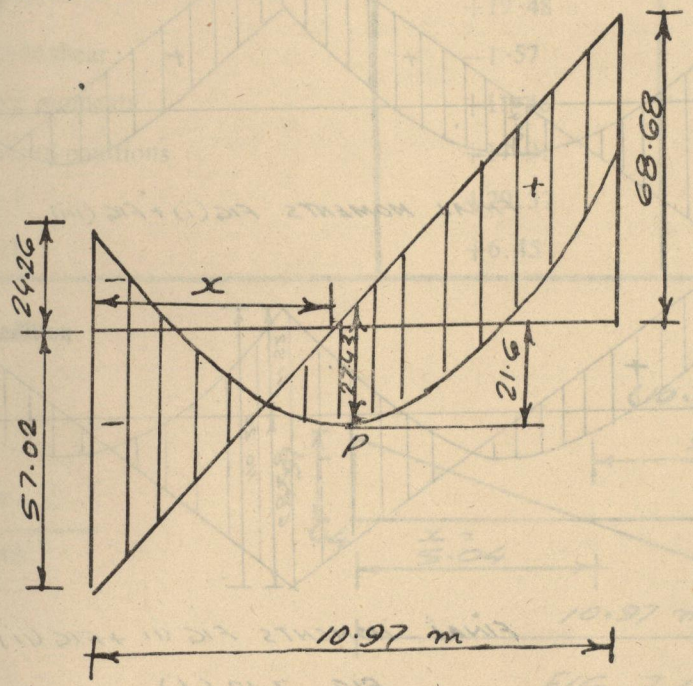
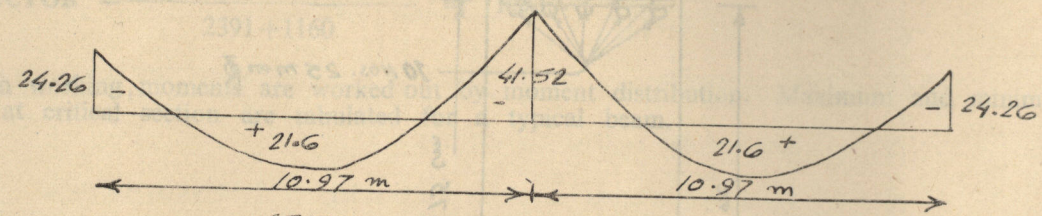
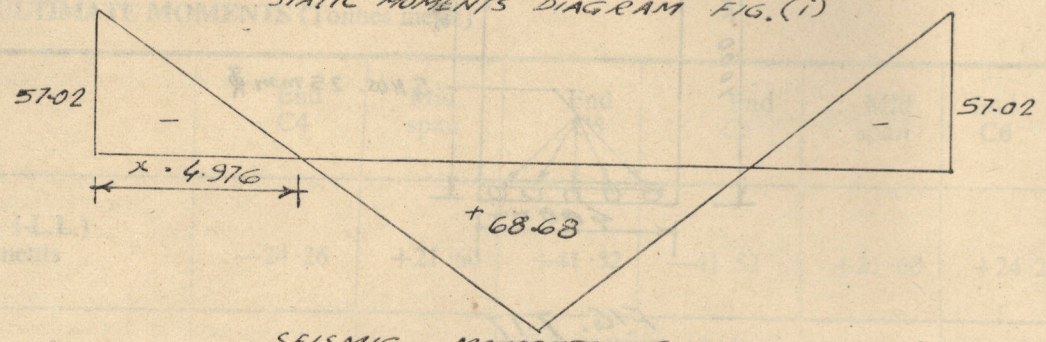


FIG. 7.18 (a)

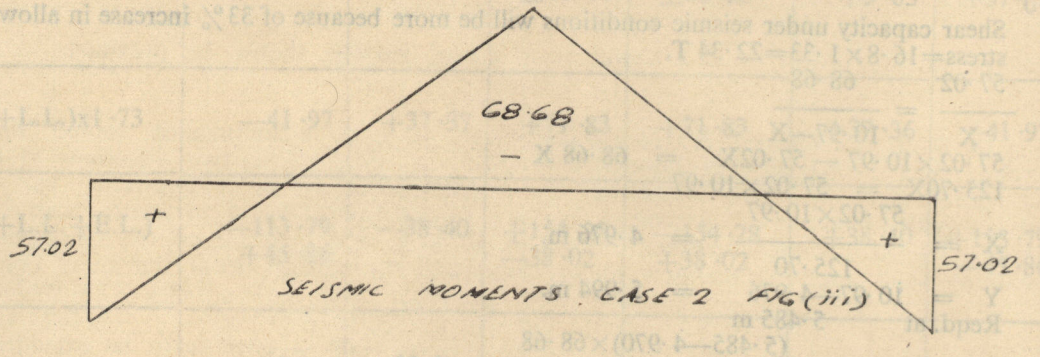
B. M. Diagram



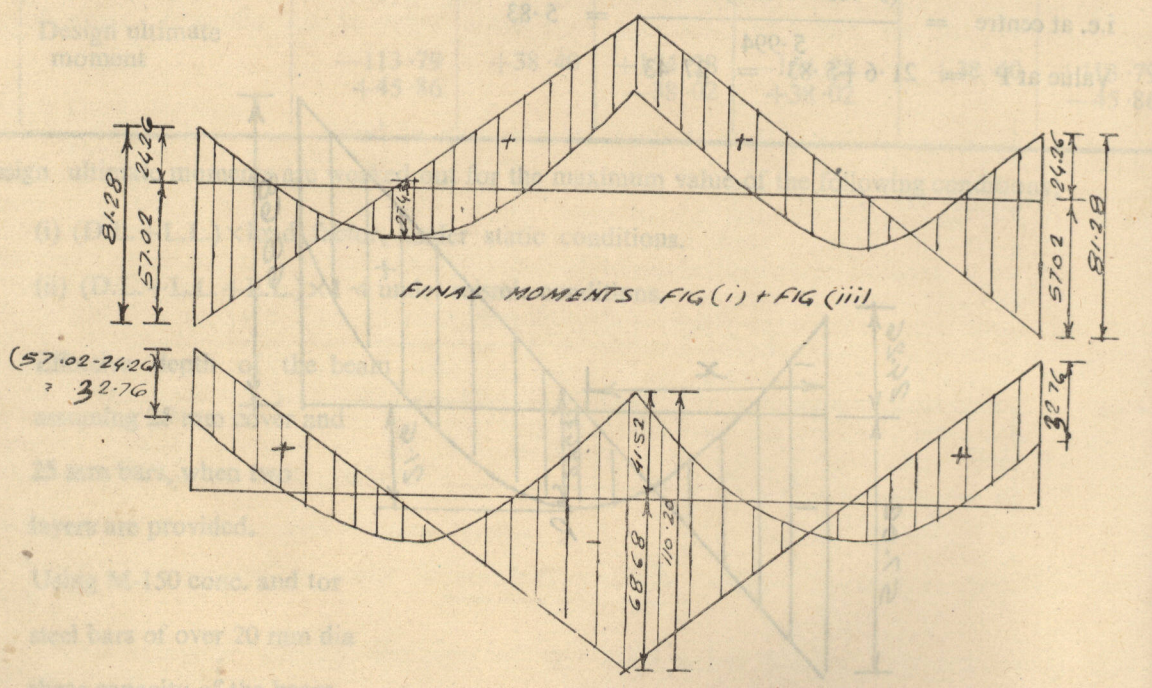
STATIC MOMENTS DIAGRAM FIG. (i)



SEISMIC MOMENTS CASE 1 FIG (ii)



SEISMIC MOMENTS CASE 2 FIG (iii)



FINAL MOMENTS FIG (i) + FIG (iii)

FINAL MOMENTS FIG (i) + FIG (ii)

FIG. 7.18 (b)

Sections		Mu Tm	Eff. depth d_1 cms.	$K=\frac{M}{bd_1^2}$	100 pt from tables	Ast & or Asc cm^2	Reqd area of reinforcement cm^2	Reinforcement provided
At col. C4, C6	Top	113.79	93·75	32·37	1.945/.13	35·44	35·44	8 Nos. 25 ϕ = 39·27 cm^2
	Bott	45·86		13·05	0·335	12·54	12·54	4 Nos. 22 ϕ = 5·120 cm^2
Mid span of C4-C5 and C5-C6	Top							
	Bott	38·40		10·92	0·276	10·35	10·35	4 Nos. 22 ϕ = 15·20 cm^2
At col. C5	Top	154·28	93·75	43·88	1·23/·46	46·13	46·13	10 Nos. 25 ϕ = 49·08 cm^2
	Bott	38·02		10·82	0·274	17·25	17·25	5 Nos. 22 = 19·00 cm^2

DESIGN FOR SHEAR

Shear force in Tonnes

	C4	C5
Vertical load shear	+19·48	—19·48
Vertical load moment shear	—1·57	— 1·57
Shear due to seismic moments	+11·46	+11·46
Shear under static load conitions	+17·91	—21·05
S. F. Seismic case I	+29.37	—9.59
case II	+6.45	—32.51

Under vertical load condition

$$\frac{X}{17\cdot91} = \frac{10\cdot97-X}{21\cdot05}$$

$$X = 5\cdot04 \text{ m}$$

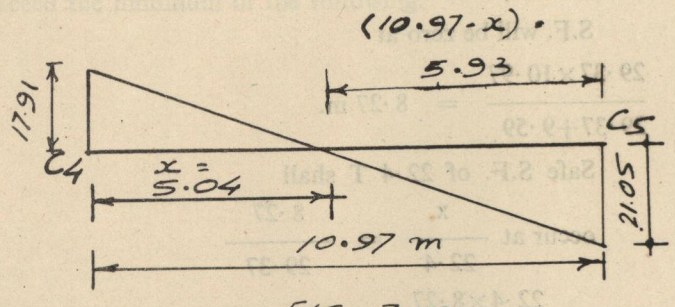


FIG. 7.19

Sections		Mu Tm	Eff. depth d_1 cms.	$K=\frac{M}{bd_1^2}$	100 pt from tables	Ast & or Asc cm^2	Reqd area of reinforcement cm^2	Reinforcement provided
At col. C4, C6	Top	113.79	93·75	32·37	1.945/.13	35·44	35·44	8 Nos. 25 ϕ =39·27 cm^2
	Bott	45·86		13·05	0·335	12·54	12·54	4 Nos. 22 ϕ =5·120 cm^2
Mid span of C4-C5 and C5-C6	Top							
	Bott	38·40		10·92	0·276	10·35	10·35	4 Nos. 22 ϕ =15·20cm^2
At col. C5	Top	154·28	93·75	43·88	1·23/·46	46·13	46·13	10 Nos· 25 ϕ=49·08 cm^2
	Bott	38·02		10·82	0·274	17·25	17·25	5 Nos. 22 =19·00cm^2

DESIGN FOR SHEAR

Shear force in Tonnes

	C4	C5
Vertical load shear	+19·48	—19·48
Vertical load moment shear	—1·57	— 1·57
Shear due to seismic moments	+11·46	+11·46
Shear under static load conitions	+17·91	—21·05
S. F. Seismic case I	+29.37	—9.59
case II	+6.45	—32. 51

Under vertical load condition

$$\frac{X}{17\cdot91} = \frac{10\cdot97-X}{21\cdot05}$$

$$X = 5\cdot04 \text{ m}$$

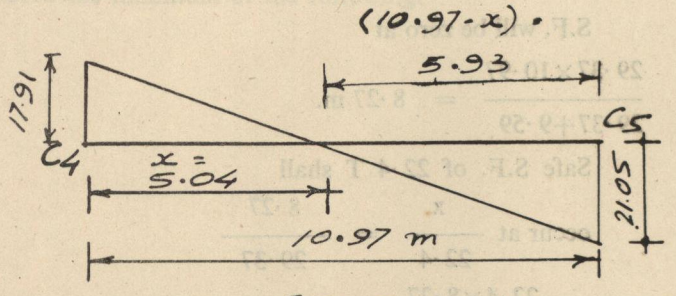


FIG. 7.19

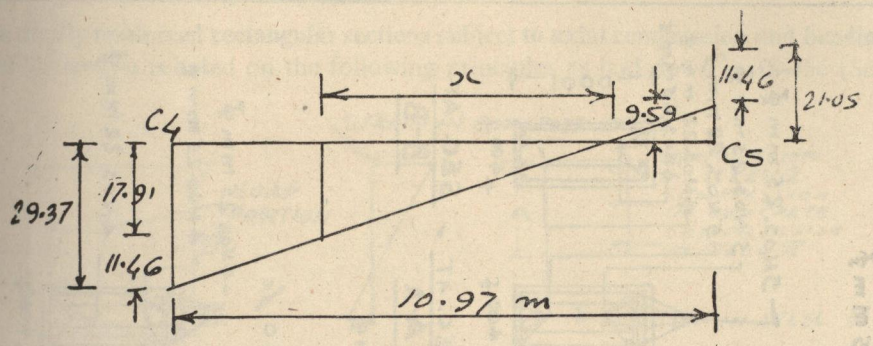


FIG. 7·21

Stirrups reqd. upto $8 \cdot 27 - 6 \cdot 31 + jd_1$ = 2·80 metres from C4

Shear force under seismic conditions being more, we design shear reinforcement under this condition.

At C5 :—Max S.F. = 32·51 T.

Design shear after increase in stresses by 33% = $\frac{32 \cdot 51}{1 \cdot 33}$ = 24·44 T.

$$\frac{Q}{jd_1} = \frac{24 \cdot 44 \times 1000}{\cdot 896 \times 100} = 272 \cdot 77$$

from table A4 spacing of 8mm ϕ stirrups (two legged) = 6 cm c/c.

Provide 4 L-8 mm ϕ stirrups @ 12 cms c/c.

At C 4 & C6 Max. S.F. = 29·37 T

design shear = $\frac{29 \cdot 37}{1 \cdot 33}$ = 22·08 T

$$\frac{Q}{jd} = \frac{22 \cdot 08 \times 1000}{\cdot 896 \times 100} = 246 \cdot 43$$

from table-14 spacing of 8 mm ϕ stirrups (two legged) = 7 cms. c/c.

Provide 4L-8 mm stirrups @ 14 cms. c/c.

8mm-4L 14 cms. c/c	Nominal stirrups	8mm-4L @ 12 cms. c/c	Nominal stirrups	8mm-4 L 14 cms. c/c.
⟵2·80⟶	⟵3·70⟵	3·70⟶		⟵2·80⟶

At other places nominal stirrups shall be provided.

Spacing of nominal stirrups have not to exceed the minimum of the following.

16 times dia of bars

i.e. 16×22 = 352 mm

or 48 times dia of stirrup

i.e. 48×8 = 384 mm

or d1/2 = $\frac{93 \cdot 75}{2}$ = 46·87 cms.

Provide 8 mm ϕ -4L-350 mm c/c

The arrangement of reinforcement is as shown in Fig. 7·22.

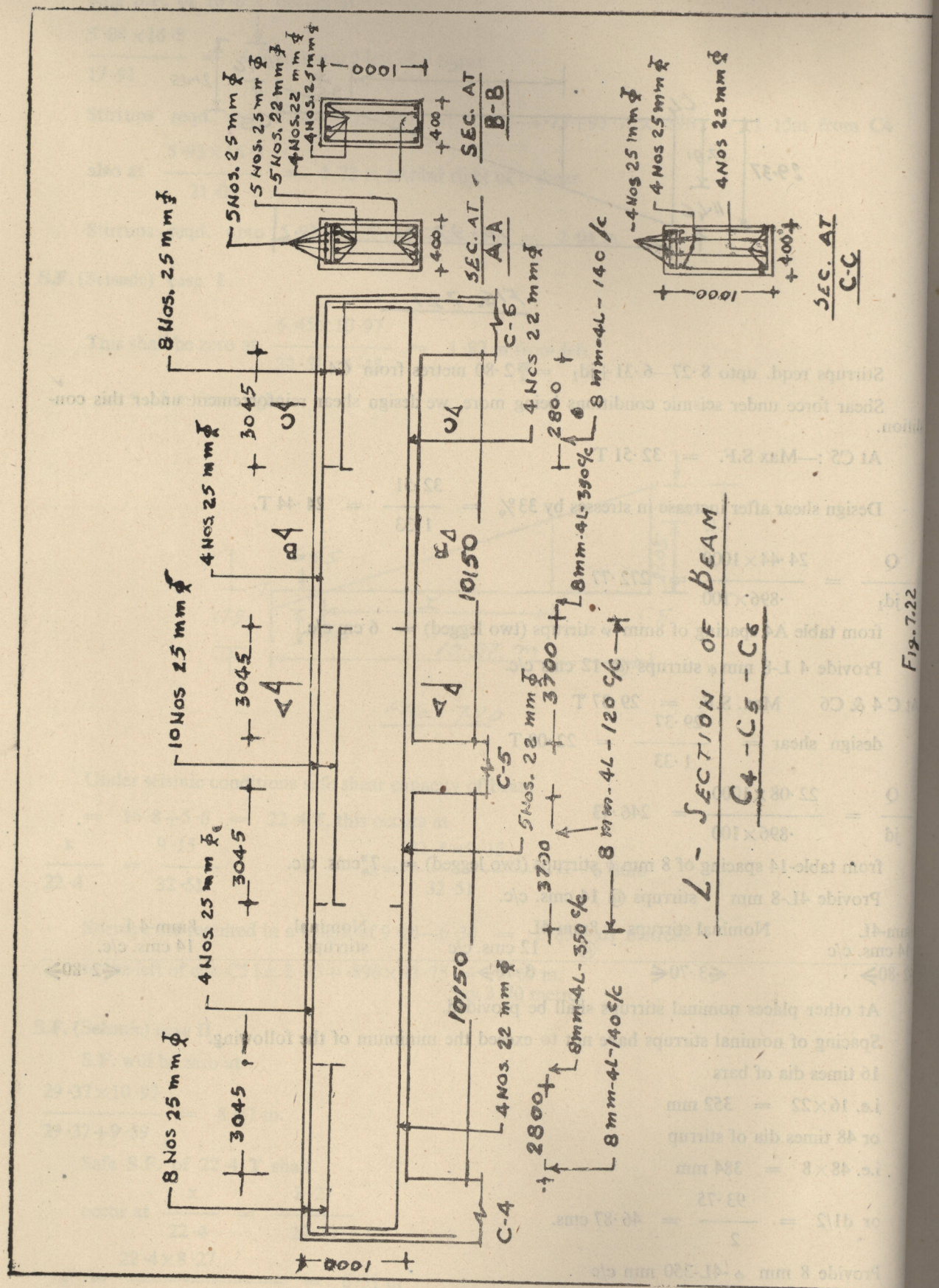

8 Nos. 25 mm ϕ
10 Nos 25 mm ϕ
4 Nos. 25 mm ϕ
4 Nos 25 mm ϕ
8 Nos 25 mm ϕ
3045
3045
3045
3045
C-6
C-5
C-4
4 Nos 22 mm ϕ
2800
8 mm-4L-140 c/c
8mm-4L-390c/c
3700
8mm-4L-120 c/c
5 Nos. 22 mm ϕ
8mm-4L-350 c/c
8 mm-4L-140c/c
10150
10150
1000
400
SEC. AT A-A
SEC. AT B-B
SEC. AT C-C
4 Nos 25 mm ϕ
4 Nos 22 mm ϕ
7 - SECTION OF BEAM C4 - C5 - C6

Fig. 7.22

7·5 Design of Columns

7·5·1 Symmetrically reinforced rectangular sections subject to axial compression and bending in one plane

The design of the section is based on the following principles as laid down in IS-456 (See Fig. 7.23)

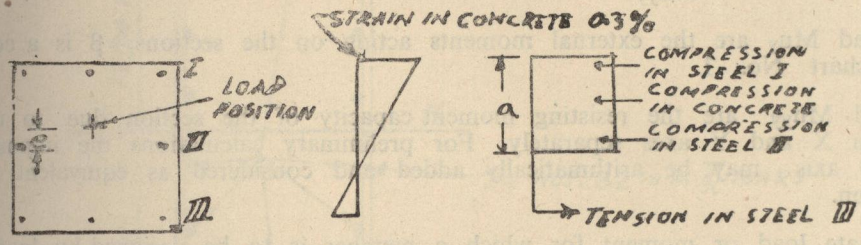


Fig. 7.23

(a) The depth of the rectangular compressive stress block should be equal to 3/4th the depth of the neutral axis subject to the limitation that this should not exceed a value equal to the depth of the cross section

(b) If the depth of the compressive stress block is less than half the effective depth of the section, the average compressive stress in the rectangular stress block is to be considered as 0·55 σ cu.

(c) If the depth of the rectangular stress block is equel to or more than half the effective depth of the section, then the average compressive stress in the rectangular stress block is reduced in a manner such that the resultant moment of the compression about the centroid of the tension reinforcement, or the reinforcement least in the compression, is equal to the moment obtained for the condition when the depth of the stress block is equal to the half the effective depth of the section.

(d) The stress in steel is obtained by multiplying the appropriate strain by the modulus of elasticity subject to the limit of yield stress.

(e) The maximum strain in the concrete may be assumed 0·3% for the purpose of design of the section.

(f) The section is designed from the conditions that the algebric sum of the forces in concrete and steel is equal to the ultimate compressive load on the section and that the ultimate moment due to the external forces about any reference line is equal to the algebric sum of moments of forces in concrete and steel about the same reference line.

The equilibrium of forces results in the equation :

$$Pu = 0{\cdot}55\ \sigma cu\ bxa + Asc\ (\sigma scy - 0{\cdot}55\ \sigma cu) - Ast \times \sigma\ sy$$

The moment equilibrium gives the equation :

$$Mu = 0{\cdot}55 \sigma cu\ \ bxa\ (d_1 - a/_2) + Asc\ (\sigma scy - 0{\cdot}55\ \sigma cu)\ (d_1 - d_2)$$

For design of a section a trial depth of the neutral axis is considered and based on the above principles it is verified whether the forces and moment equilibrium equation is satisfied. Depending on the depth of the neutral axis the failure may be due to the crushing of the concrete even before the steel yields or it may be due to the yielding of the reinforcement. Since a large number of trials are required standard charts are available which indicate the interaction diagram of $\frac{Pu}{\sigma cu \times bd}$ against $\frac{Mu}{\sigma cubd^2}$ for a particular percentage of steel arrangedin a symetrical manner.

For practical design reference is made to the charts 2 to 17.

7·5·2 Rectangular sections subject to axial compression and bending in two planes.

The solution may be obtained on the above principles as for bending in one plane. As this process involves considerable calculations it is convenient to transform the bi-axial moment to the uni-axial by any of the standard methods such as Pannels method. (Alternatively, a suitable percentage of steel is assumed and the capacity of the section to resist the external moments in the two planes acting simultaneously is compared with the external moments acting on the section, the relevant equation being

$$\left(\frac{Mu_x}{Mu_{xx}}\right)^{\frac{\log 0\cdot 5}{\log \beta}} + \left(\frac{Mu_y}{Mu_{yy}}\right)^{\frac{\log 0\cdot 5}{\log \beta}} = 1$$

Where Mu_x and Mu_y are the external moments acting on the sections. β is a coefficient obtained from chart No. 1.

Mu_{xx} and Mu_{yy} are the resisting moment capacity of the section due to uni-axial moment acting on X and Y axes separately. For preliminary calculations the imposed moments about the two axis may be arithmatically added and considered as equivalent moment for uniaxial condition.

The ultimate load or moment for which a member is to be designed by load factor method is obtained by greatest 'U' value obtained from the following :—

(a) $U = 1\cdot5\ DL + 2\cdot2\ LL$

(b) When wind load is considered in addition to dead and live load.
$U = 1\cdot5\ DL + 2\cdot2\ LL + 0\cdot5\ WL$

(c) $U = 1\cdot5\ DL + 0\cdot5\ LL + 2\cdot2\ WL$

(d) For structures in which earthquake load is considered in addition to dead load and live load.
$U = 1\cdot4\ (DL + LL + EL)$

where DL=Dead load
LL=Live load
WL=Wind load
EL=Earthquake load, and
U=Ultimate load

The member is designed as per the condition which gives the highest ultimate load/ moment on the member. However wind and earthquake loads are not taken to act together.

The percentage of steel is calculated with the hlep of charts from $\frac{Pu}{\sigma cu\ bd}$ and $\frac{Mu}{\sigma cubd^2}$

Example: **COLUMN DESIGN**

Data : P=664 tonnes

Pu=228 tonnes

Mu_x=64 tonnes metre

Mu_y=55 tonnes metre

Mix M 300

Size 61×122 cms.

First approximate solution :—

Assuming it as uniaxial with Moment (64+55) Tm=119 Tm about x x—axis.

$$\frac{Pu}{\sigma cubd} = \frac{1228000}{300\times 61\times 122^2} = 0\cdot55$$

$$\frac{Mu}{\sigma cu\ bd^2} = \frac{11900000}{300\times 61\times 122^2} = 0\cdot044$$

From Chart 13 $\frac{r}{\sigma cu} = 0\cdot65\times 10^{-4}$

$$Ast = 0\cdot65\times 10^{-4}\times 300\times 61\times 122 = 145\cdot12\ cm^2$$

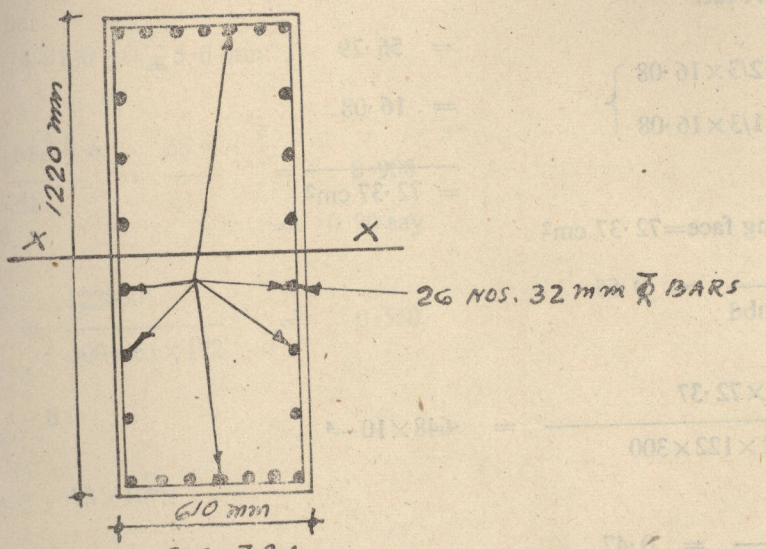


FIG 7·24

Providing 26—32 ϕ-Tor (209·1 cm2) to give effective area as under steel on each short face.

7—32 dia	=	56·29
2—32 dia=5/7×16·08	=	11·48
2—32 dia=3/7×16·08		6·89
2—32 dia=1/7×16·08	=	2·29
		76·96 cm²

$$\frac{r}{\sigma cu} = \frac{2\times 76\cdot 96}{61\times 122\times 300} = 0\cdot 68\times 10^{-4}$$

Which is in close resemblance with the previously obtained value from chart.

Assumed section for check for the axial load and moments about both axis :—

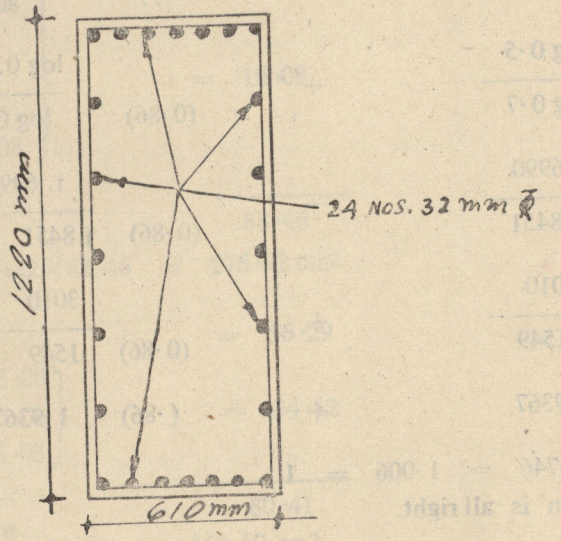


FIG. 7·25

Steel on each short face

7—32 dia = 56·29

2—32 dia = 2/3 × 16·08 } = 16·08

2—32 dia = 1/3 × 16·08 }

= 72·37 cm²

Steel on each long face = 72·37 cm²

For Mxx $\frac{Pu}{\sigma cubd} = 0 \cdot 55$

$$\frac{r}{\sigma cu} = \frac{2 \times 72 \cdot 37}{61 \times 122 \times 300} = \cdot 648 \times 10^{-4}$$

From chart 4

$$\frac{Mu}{\sigma cubd^2} = 0 \cdot 47$$

$$Mu = \frac{\cdot 047 \times 300 \times 61 \times 122^2}{100000} = 128 \cdot 01 \text{ Tm}$$

For Myy:— $\frac{Mu}{\sigma cubd^2} = \frac{\cdot 047 \times 300 \times 122 \times 61^2}{100000} = 64 \cdot 00$ Tm

$$\frac{Mux}{Muxx} = \frac{64}{128 \cdot 01} = 0 \cdot 5$$

$$\frac{Muy}{Muyy} = \frac{55}{64 \cdot 00} = 0 \cdot 86$$

From chart

$\beta = 0 \cdot 70$

$$\left(\frac{Mux}{Muxx}\right)^{\frac{\log 0 \cdot 5}{\log \beta}} + \left(\frac{Muy}{Muyy}\right)^{\frac{\log 0 \cdot 5}{\log \beta}}$$

L.H.S. $= (0 \cdot 5)^{\frac{\log 0 \cdot 5}{\log 0 \cdot 7}} + (0 \cdot 86)^{\frac{\log 0.5}{\log 0 \cdot 7}}$

$(0 \cdot 5)^{\frac{\bar{1} .6990}{\bar{1} \cdot 8451}} + (0 \cdot 86)^{\frac{\bar{1}. 6990}{\bar{1}\ 8451}}$

$(0 \cdot 5)^{\frac{\cdot 3010}{\cdot 1549}} + (0 \cdot 86)^{\frac{\cdot 3010}{\cdot 1549}}$

$(0 \cdot 5)^{1 \cdot 9367} + (\cdot 86)^{1 \cdot 9367}$

$= 0 \cdot 260 + 0 \cdot 746 = 1 \cdot 006 = 1$

Hence assumed section is all right.

Check for the condition $\frac{e}{d} = 0 \cdot 1$

Providing clear over= 4 cms.

Using 32 mm dia bar

effective cover = 4+1·6 = 5·6 mm

About yy Axis

$$\frac{d1}{d} = \frac{61-5\cdot6}{61} = \frac{55\cdot4}{61} = 0\cdot908$$

$$= 0\cdot90 \text{ say}$$

From chart No. 12

for $\frac{Pu}{\sigma cubd} = \frac{122800}{300\times61\times122} = 0\cdot550$

and $\frac{e}{d} = 0\cdot1$

$$\frac{r}{\sigma cu} = \cdot73\times10^{-4}$$

Ast = r b d

$= \cdot73\times10^{-4}\times300\times61\times122 = 162\cdot98 \text{ cm}^2$

About xx Axis

$$\frac{d1}{d} = \frac{122-5\cdot6}{122} = 0\cdot95$$

From chart 13

for $\frac{Pu}{\sigma cubd} = 0\cdot550$ & e/d = 0·1

$$\frac{r}{\sigma cu} = 0.7\text{x}10^{-4}$$

Ast = $\cdot7\times10^{-4}\times300\times61\times122 = 156\cdot3 \text{ cm}^2$

Providing 28 Nos. of 32 dia bars.

Steel on each long face

9 Nos. 32 ϕ-Tor = 72·38

2 Nos. 32 ϕ-Tor = $\frac{2}{3}\times16\cdot08$

2 Nos. 32 ϕ-Tor = $\frac{1}{3}\times16\cdot08$

} = 16·08

88·46

Total steel for Myy = $2\times88\cdot46$ = 176·92 cm²

Steel on each short face.

7 Nos. 32 ϕ-Tor = 56·29

2 Nos. 32 ϕ-Tor = 3/4×16·08

2 Nos. 32 ϕ-Tor = 2/4×16·08

2 Nos. 32 ϕ-Tor = 1/4×16·08

} = 24·12

80·41

Total steel for Mxx = $2\times80\cdot41$ = 160·82 cm²

Transverse Reinforcement

$$\text{Min. dia of Tr. reinf.} = \frac{d}{4} = \frac{32}{4} = 8\text{mm}$$

Pitch

1. Least lateral dimension of column = 61 cms.
2. Yx dia of longitudinal bar. where

$$Y = \frac{12\times(80\times 7216\cdot 8)+(1750\times 225\cdot 2)}{664000}$$

See IS 456

$$= 17\cdot 56 \text{ (Value should lie between 16 \& 24)}$$

$$Y\times D$$

$$= 17\cdot 56\times 3\cdot 2 = 56\cdot 19 \text{ cms.}$$

3. 48×dia of transverse reinforcement providing 8 mm dia.

$$48\times 0\cdot 8 = 38\cdot 4 \text{ cms.}$$

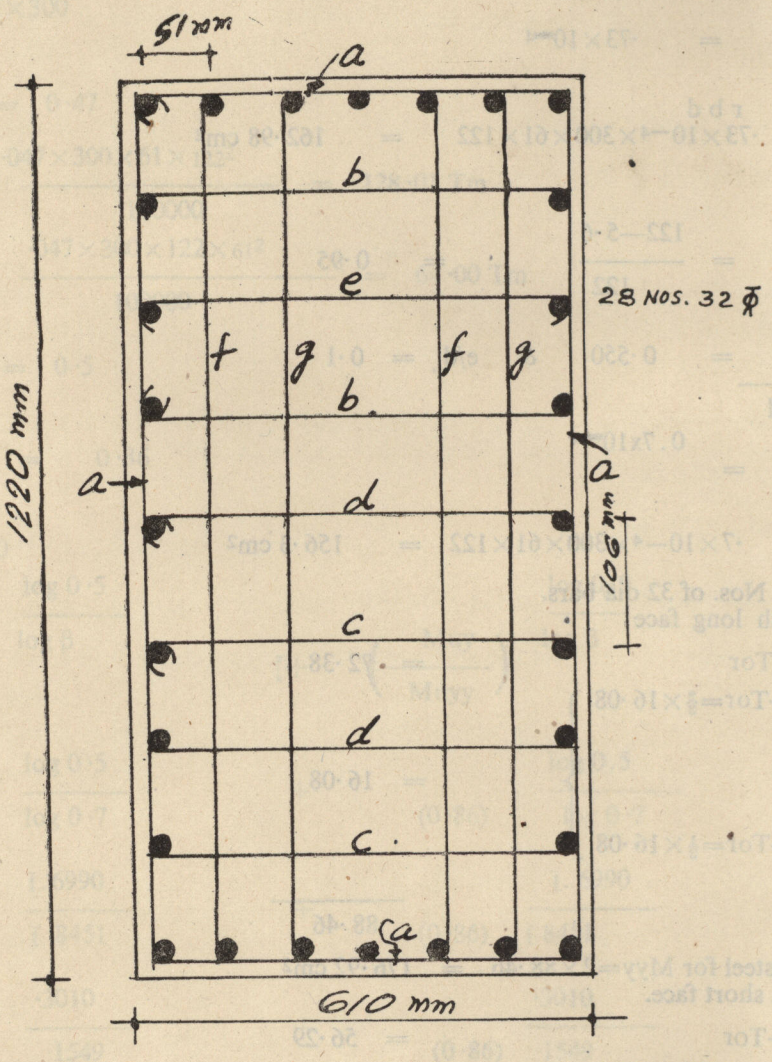


FIG. 7.26

Providing 8mm ϕ-Tor @ 30 cm ties. In 20% of the column height at top & bottom spacing of ties will be as per I.S. 4326 spacing = 4×dia of vertical bars or 7·5 cm whichever is more

so spacing = 12·8 cm

Say 12 cm. c/c.

BIBLIOGRAPHY

V. K. GHANEKAR, R. CHANDRA & S. SARKAR—

"HAND BOOK FOR ULTIMATE STRENGTH DESIGN OF REINFORCED CONCRETE MEMBERS", Structural Engineering, Research Centre, Roorkee, U. P.

CLARENCE W. DUNHAM, "Advanced Reinforced Concrete", McGraw— Hill Book Company, INC., New York—San Francisco—Toronto—London, 1964.

CHAS E. REYNOIDS "Reinforced Concrete Designer's Handbook", Sixth Edition (Revised—1964), Concrete Publications Limited 60, Buckingham Gate, London, SWI, 1964.

LEONARD CHURCH URQUHARD, CHARLES EDWARD O'ROURKE, GEORGE WINTER, "Design of Concrete Structures", Fifth Edition, McGraw-Hill Book Company, Inc., New York-Toronto—London, 1954.

REFERENCES

ACI Standard Building Code Requirements for Reinforced Concrete (ACI 318—71).

IS : 456—1964 . . . . . INDIAN STANDARD CODE OF PRACTICE FOR PLAIN AND REINFORCED CONCRETE

CHAPTER 8

DETAILING OF REINFORCEMENT

8·1 A Manual on Structural Design will not be complete without reference to the important aspec of structural detailing. The structural drawings should take into account the limitations o construction and adequate details of structural members. At the same time multiplicity o drawings will also create problems at site. There is, therefore, need for evolving compact struc tural drawings and reinforcement schedules which could be easily understood by the site staf and translated into practice. The drawings should also contain relevant notes regarding mix o concrete to be used, length of anchorages and laps for different types of reinforcements. Sufficien Cross references must be given in the notes so as to enable the man at site to read an interpret the various drawings and details properly.

8·2 Four typical drawings covering the various items are appended at the end of th manual, viz.

(*i*) footings

(*ii*) columns;

(*iii*) beams; and

(*iv*) slabs.

(Plates 1 to 4 at the end).

These drawings may serve to indicate the method of presenting design information in an unm biguous and clear manner. Typical cross sections and longitudinal sections are given. Inste of giving and elaborate bar bending schedule arrangement of bending of bars are shown in th plans and sections. These, together with the notes on anchorages and laps would be adequa for site staff to bend the barss and assemble the reinforcements for various structural el ments.

8·3 In case of long beams and large spans of slabs it may be necessary to lap t reinforcement bars or weld them; both are permissible. Care, however, must be taken to s that lapping or welding of bars is done at the point of minimum bending moments and a properly staggerred. The lapping of bars can be done side by side or one over the oth depending upon the availability of space, configuration of reinforcement etc.

8·4 **General Detailing principles for Cover.**

Reinforcement shall have concrete cover and the thickness of such cover (exclusive plaster or other decorative finish) shall be as follows :

(*i*) At each end of reinforcing bar not less than 25 mm, nor less than twice the diame of such rod or bar;

(*ii*) For a longitudinal reinforcing bar in a column not less than 40 mm nor less th diameter of such rod or bar. In the case of columns with a minimum diamens of 20 cms. or under, whose reinforcing bars do not exceed 12 mm, the cover of may be used.

(*iii*) For longitudinal reinforcing bar in a beam not less than 25 mm nor less than diameter of such rod or bar.

(*iv*) For tensile, compressive, shear or other reinforcement in a slab not less than 13 m nor less than the diameter of such reinforcement, and

(*v*) For any other reinforcement not less than 13 mm, nor less than the diameter of s reinforcement.

8·5 Increased cover thickness may be provided when the surfaces of concrete memb are exposed to the action of harmful chemicals (as in the case of concrete in contact w earth faces contaminated with such chemicals), acid, vapour, saline atmosphere, sulphur smoke (as in the case of stress-operated railways) etc, and such increase of cover may be b ween 15 mm and 40 mm beyond the figures given in 8·4 (1) to (v) above as may be spe fied by the Engineer-in-Charge.

8·6 For reinforced concrete members, totally or periodically immensed in sea water or subject to sea water spray, the cover of concrete shall be 50 mm more than that specified in 8·4.

8·7 Protection to reinforcement in case of concrete exposed to harmful surroundings as mentioned in 8·5 and 8·6 can also be given by providing a dense concrete impermeable to water. The Engineer-in-Charge may reduce the extra cover mentioned in para 8·5 & 8·6 if the concrete mix is designed to ensure impermeable concrete.

8·8 Anchoring reinforcement bars.

Straight anchorage shall be permitted only for deformed bars. The anchorage value of a bend shall be taken as 4 times the diameter of the bar for each 45° bend subject to a maximum of 16 times the diameter of the bar, for 180° bend hooks. The internal radius of the bend shall not be less than twice the diameter of the bar. The length of the straight part of the bar beyond the end of the curve shall be atleast 4 times the diameter of the bar. Plain bars in tension shall always end in hooks. The bends and hooks shall conform to IS : 2502-1963.

For anchoring the bars in compression the hooks bends and straight length beyond bends if provided for a bar in a member in direct compression shall not be accounted for anchorage purposes.

Deformed bars may be used without hooks provided anchorage requirements are adequately met with. Where hooks are formed in deformed bars, the entire radius of the bend should be atleast 3 times the diameter of the bars. The length of straight bar beyond the end of the curve should be atleast 4 times the diameter of the bar.

In case of stirrups and transverse ties, complete bond length and anchorage shall be deemed to have been provided when the bar is bent through an angle of atleast 90° round a bar of atleast its own diameter and is continued beyond the end of the curve for a length of atleast 8 diameters, or, alternatively, when the bar is bent through an angle of 180° and is continued beyond the end of the curve for a length of atleast 4 bars diameters.

The compression reinforcement shall be effectively anchored in two directions at right angles over the distance where it is required to act in compression, at points not further apart, centre to centre, than 16 times the diameter of the anchored bar or 48 times the diameter of the anchoring bar whichever is less.

8·9 Spacing of reinforcement.

The horizontal distance between the two parallel main reinforcement bars in reinforced concrete shall usually be not less than the greater of the following :

(*a*) The diamter of the bar; if the diameters are equal,

(*b*) The diameter of the larger bar if the diameters are unequal, and

(*c*) 6 mm more than the nominal maximum size of the coarse aggregate used in the concrete.

Greater horizontal distance than the minimum specified in 8·9 should be provided, where possible, However, when needle vibrators are intended to be used, the horizontal distance between bars of a group may be reduced to two-thirds of the nominal maximum size of the coarse aggregate provided that sufficient space is left between groups of bars to enable the vibrator to be immersed.

The minimum vertical distance between two horizontal main reinforcing bars shall normally be 15 mm, the maximum size of the coarse aggregate or the maximum size of the bar whichever is the greatest. In locations where reinforcement is congested, grouping of bars by touching one another may be permitted provided the requirements of minimum horizontal distance as specified in 8·9 are complied with and provided further that bond length is adequately increased. The pitch of bars of main tensile reinforcement in solid slabs shall be not more than three times the effective depth of such slab or 600 mm whichever is smaller. It is the general practice to limit this spacing to 200 mm as per International Code of Practice.

The pitch of distributing bars in reinforced concrete solid slab or the pitch of bars provided against shrinkage and temperature shall be not more than five times the effective depth of such slab or 600 mm whichever is smaller.

8·10 Splices in reinforcement.

Splices at point of maximum tensile stress shall be avoided as far as possible. The splice shall transfer the entire stress from one bar to another bar.

For contact splices, the space laterally closer than 12 bar diameter or located closer than 15 cm or 6 bars diameter from outside edge, the lap shall be increased by 20% or stirrups or closely spaced spirals shall enclose the splice for its full length.

Where more than one-half of the bars are spliced within a length of 40 bar diameter or where splices are met at points of maximum stress, special precautions shall be taken such as increasing the length of lap/or using closely spaced stirrups around and for the length of the splice.

For splices in compression reinforcement the lap lengths shall be sufficient to transfer the stress from one bar to another. In columns where longitudinal bars are off-set at a splice, the slope of the inclined portion of the bar with the axis of the column shall not exceed 1:6, and the portions of the bar above and below the off-set shall be parallel to the axis of the column.

TABLE 1

(Para 7·1·1)

MIX M150 MILD STEEL UNDER-REINFORCED RECTANGULAR SECTIONS

K Kg/sqcm	100 Pt	K Kg/sqcm	100 Pt	K Kg/sqcm	100 Pt	K Kg/sqcm	100 Pt
4·00	0·158	11·50	0·478	17·00	0·740	22·50	1·034
4·20	0·166	11·60	0·483	17·10	0·745	22·60	1·040
4·40	0·174	11·70	0·487	17·20	0·750	22·70	1·045
4·60	0·182	11·80	0·492	17·30	0·755	22·80	1·051
4·80	0·190	11·90	0.497	17·40	0·760	22·90	1·057
5·00	0·199	12·00	0·501	17·50	0·766	23·00	1·063
5·20	0·207	12·10	0·506	17·60	0·771	23·10	1·069
5·40	0·215	12·20	0·510	17·70	0·776	23·20	1·074
5·60	0·223	12·30	0·515	17·80	0·781	23·30	1·080
5·80	0·232	12·40	0·519	17·90	0·786	23·40	1·086
6·00	0·240	12·50	0·524	18·00	0·791	23·50	1·092
6·20	0·248	12·60	0·529	18·10	0·796	23·60	1·098
6·40	0·257	12·70	0·533	18·20	0·801	23·70	1·104
6·60	0·265	12·80	0·538	18·30	0·806	23·80	1·110
6·80	0·273	12·90	0·543	18·40	0·812	23·90	1·115
7·00	0·282	13·00	0·547	18·50	0·817	24·00	1·121
7·20	0·290	13·10	0·552	18·60	0·822	24·10	1·127
7·40	0·299	13·20	0·557	18·70	0·827	24·20	1·133
7·60	0·307	13·30	0·561	18·80	0·832	24·30	1·139
7·80	0·316	13·40	0·566	18·90	0·838	24·40	1·145
8·00	0·324	13·50	0·571	19·00	0·843	24·50	1·151
8·10	0·329	13·60	0·575	19·10	0·848	24·60	1·157
8·20	0·333	13·70	0·580	19·20	0·853	24·70	1·163
8·30	0·337	13·80	0·585	19·30	0·859	24·80	1·170
8·40	0·341	13·90	0·589	19·40	0·864	24·90	1·176
8·50	0·346	14·00	0·594	19·50	0·869	25·00	1·182
8·60	0·350	14·10	0·599	19·60	0·874	25·10	1·188
8·70	0·354	14·20	0·604	19·70	0·880	25·20	1·194
8·80	0·359	14·30	0·608	19·80	0·885	25·30	1·200
8·90	0·363	14·40	0·613	19·90	0·890	25·40	1·206
9·00	0·367	14·50	0·618	20·00	0·896	25·50	1·213
9·10	0·372	14·60	0·623	20·10	0·901	25·60	1·219
9·20	0·376	14·70	0·627	20·20	0·907	25·70	1·225
9·30	0·381	14·80	0·632	20·30	0·912	25·80	1·231
9·40	0·385	14·90	0·637	20·40	0·917	25·90	1·238
9·50	0·389	15·00	0·642	20·50	0·923	26·00	1·244
9·60	0·394	15·10	0·647	20·60	0·928	26·10	1·250
9·70	0·398	15·20	0·652	20·70	0·934	26·20	1·257
9·80	0·402	15·30	0·656	20·80	0·939	26·30	1·263
9·90	0·407	15·40	0·661	20·90	0·945	26·40	1·270
10·00	0·411	15·50	0·666	21·00	0·950	26·50	1·276
10·10	0·416	15·60	0·671	21·10	0·956	26·60	1·282
10·20	0·420	15·70	0·676	21·20	0·961	26·70	1·289
10·30	0·425	15·80	0·681	21·30	0·967	26·80	1·295
10·40	0·429	15·90	0·686	21·40	0·972	26·90	1·302
10·50	0·433	16·00	0·691	21·50	0·978	27·00	1·308
10·60	0·438	16·10	0·696	21·60	0·983	27·10	1·315
10·70	0·442	15·20	0·700	21·70	0·989	27·20	1.322
10·80	0·447	16·30	0·705	21·80	0·994	27·30	1·328
10·90	0·451	16·40	0·710	21·90	1·000	27·40	1·335
11·00	0·456	16·50	0·715	22·00	1·006	27·50	1·342
11·10	0·460	16·60	0·720	22·10	1·011	27·60	1·348
11·20	0·465	17·70	0·725	22·20	1·017	27·70	1·355
11·30	0·469	16·80	0·730	22·30	1·023	27·80	1·362
11·40	0·474	16·90	0·735	22·40	1·028	27·84	1·364

TABLE 2

(Para 7·1·1)

MIX M200 MILD STEEL UNDER-REINFORCED RECTANGULAR SECTIONS

K Kg/sqcm	100 Pt	K Kg/sqcm	100 Pt	K Kg/sqcm	100 Pt	K Kg/sqcm	130 Pt
4·00	0·157	15·00	0·623	26·00	1·159	32·00	1·495
4·20	0·165	15·20	0·632	26·20	1·169	32·10	1·501
4·40	0·173	15·40	0·641	26·40	1·180	32·20	1·507
4·60	0·181	15·60	0·650	26·60	1·191	32·30	1·513
4·80	0·189	15·80	0·659	26·80	1·202	32·40	1·519
5·00	0·197	16·00	0·668	27·00	1·212	32·50	1·525
5·20	0·205	16·20	0·677	27·10	1·218	32·60	1·531
5·40	0·213	16·40	0·687	27·20	1·223	32·70	1·537
5·60	0·221	16·60	0·696	27·30	1·229	32·80	1·543
5·80	0·229	16·80	0·705	27·40	1·234	32·90	1·549
6·00	0·237	17·00	0·714	27·50	1·239	33·00	1·555
6·20	0·246	17·20	0·723	27·60	1·245	33·10	1·561
6·40	0·254	17·40	0·733	27·70	1·250	33·20	1·568
6·60	0·262	17·60	0·742	27·80	1·256	33·30	1·574
6·80	0·270	17·80	0·751	27·90	1·261	33·40	1·580
7·00	0·278	18·00	0·761	28·00	1·267	33·50	1·586
7·20	0·287	18·20	0·770	28·10	1·272	33·60	1·592
7·40	0·295	18·40	0·780	28·20	1·278	33·70	1·598
7·60	0·303	18·60	0·789	28·30	1·283	33·80	1·605
7·80	0·311	18·80	0·799	28·40	1·289	33·90	1·611
8·00	0·320	19·00	0·808	28·50	1·294	34·00	1·617
8·20	0·328	19·20	0·818	28·60	1·300	34·10	1·623
8·40	0·336	19·40	0·827	28·70	1·305	34·20	1·629
8·60	0·345	19·60	0·837	28·80	1·311	34·30	1·636
8·80	0·353	19·80	0·846	28·90	1·317	34·40	1·642
9·00	0·362	20·00	0·856	29·00	1·322	34·50	1·648
9·20	0·370	20·20	0·866	29·10	1·328	34·60	1·655
9·40	0·378	20·40	0·875	29·20	1·333	34·70	1·661
9·60	0·387	20·60	0·885	29·30	1·339	34·80	1·667
9·80	0·395	20·80	0·895	29·40	1·345	34·90	1·674
10·00	0·404	21·00	0·904	29·50	1·350	35·00	1·680
10·20	0·412	21·20	0·914	29·60	1·356	35·10	1·686
10·40	0·421	21·40	0·924	29·70	1·362	35·20	1·693
10·60	0·430	21·60	0·934	29·80	1·367	35·30	1·699
10·80	0·438	21·80	0·944	29·90	1·373	35·40	1·706
11·00	0·447	22·00	0·954	30·00	1·379	35·50	1·712
11·20	0·455	22·20	0·964	30·10	1·384	35·60	1·719
11·40	0·464	22·40	0·974	30·20	1·390	35·70	1·725
11·60	0·473	22·60	0·984	30·30	1·396	25·80	1·732
11·80	0·481	22·80	0·994	30·40	1·402	35·90	1·738
12·00	0·490	23·00	1·004	30·50	1·407	36·00	1·745
12·20	0·499	23·20	1·014	30·60	1·413	36·10	1·751
12·40	0·507	23·40	1·024	30·70	1·419	36·20	1·758
12·60	0·516	23·60	1·034	30·80	1·425	36·30	1·764
12·80	0·525	23·80	1·044	30·90	1·431	36·40	1·771
13·00	0·534	24·00	1·055	31·00	1·436	36·50	1·778
13·20	0·543	24·20	1·065	31·10	1·442	36·60	1·784
13·40	0·551	24·40	1·075	31·20	1·448	36·70	1·791
13·60	0·560	24·60	1·086	31·30	1·454	36·80	1·798
13·80	0·569	24·80	1·096	31·40	1·460	36·90	1·804
14·00	0·578	25·00	1·106	31·50	1·466	37·00	1·811
14·20	0·587	25·20	1·117	31·60	1·472	37·10	1·818
14·40	0·596	25·40	1·127	31·70	1·477	37·12	1·819
14·60	0·605	25·60	1·138	31·80	1·483		
14·80	0·614	25·80	1·148	31·90	1·489		

TABLE 3

(Para 7.1.1)

MIX M25 MILD STEEL — UNDER-REINFORCED RECTANGULAR SECTIONS

K Kg/sqcm	100 Pt	K Kg/Sqcm	100 Pt	K Kg/sqcm	100 Pt	K Kg/sqcm	100 Pt
4·00	0·156	31·50	1·396	37·00	1·695	42·50	2·021
4·50	0·176	31·60	1·401	37·10	1·701	42·60	2·027
5·00	0·196	31·70	1·406	37 20	1 .706	42 70	2·034
5·50	0·216	31·80	1·412	37·30	1·712	42·80	2·040
6·00	0·236	31·90	1·417	37·40	1·718	42·90	2·046
6·50	0·256	32·00	1·422	37·50	1·723	43·00	2·053
7·00	0·276	32·10	1·427	37·60	1·729	43·10	2·059
7·50	0·297	32·20	1·433	37·70	1·735	43·20	2·065
8·00	0·317	32·30	1·438	37·80	1·741	43·30	2·071
8·50	0·338	32·40	1·443	37·90	1·746	43·40	2·078
9·00	0·358	32·50	1·449	38·00	1·752	43·50	2·084
9·50	0·379	32·60	1·454	38·10	1·758	43·60	2·090
10·00	0·400	32·70	1·459	38·20	1·764	43·70	2·097
10·50	0·421	32·80	1·465	38·30	1·769	43·80	2·103
11·00	0·442	32·90	1·470	38·40	1·775	43·90	2·110
11·50	0·463	33·00	1·475	38 50	1·781	44·00	2·116
12·00	0·484	33·10	1·481	38·60	1·787	44·10	2·122
12·50	0·505	33·20	1·486	38·70	1·793	44·20	2·129
13·00	0·526	33·30	1·491	38·80	1·798	44·30	2·135
13·50	0·548	33·40	1·497	38·90	1·804	44·40	2·142
14·00	0·569	33·50	1·502	39·00	1·810	44·50	2·148
14·50	0·591	33·60	1·507	39·10	1·816	44·60	2·155
15·00	0·612	33·70	1·513	39·20	1·822	44·70	2·161
15·50	0·634	33·80	1·518	39·30	1·828	44·80	2·168
16·00	0·656	33·90	1·524	39·40	1·834	44·90	2·174
16·50	0.678	34·00	1·529	39·50	1·839	45·00	2·181
17·00	0.700	34·10	1·534	39·60	1·845	45·10	2·187
17·50	0.722	34·20	1·540	39·70	1·851	45·20	2·194
18·00	0.745	34·30	1·545	39·80	1·857	45·30	2·201
18·50	0.767	34·40	1·551	39·90	1·863	45·40	2·207
19·00	0·790	34·50	1·556	40·00	1·869	45·50	2·214
19·50	0·812	34·60	1·562	40·10	1·875	45·60	2·220
20·00	0·835	34·70	1·567	40·20	1·881	45·70	2·227
20·50	0·858	34·80	1·572	40·30	1·887	45·80	2·234
21·00	0·881	34·90	1·578	40·40	1·893	45·90	2·240
21·50	0·904	35·00	1·583	40·50	1·899	46·00	2·247
22·00	0·928	35·10	1·589	40·60	1·905	46·10	2·254
22·50	0·951	35·20	1·594	40·70	1·911	46·20	2·260
23·00	0·974	35·30	1·600	40·80	1·917	46·30	2·267
23·50	0·998	35·40	1·605	40·90	1·923	46·40	2·274
24·00	1·022	35·50	1·611	41·00	1·929		
24·50	1. 046	35·60	1·617	41·10	1·935		
25·00	1·070	35·70	1·622	41·20	1·941		
25·50	1·094	35·80	1·628	41·30	1·947		
26·00	1·118	35·90	1·633	41·40	1·953		
26·50	1·143	36·00	1·639	41·50	1·960		
27·00	1·167	36·10	1·644	41·60	1·966		
27·50	1·192	36·20	1·650	41·70	1·972		
28·00	1·217	36·30	1·656	41·80	1·978		
28·50	1·242	36·40	1·661	41·90	1·984		
29·00	1·267	36·50	1·667	42·00	1·990		
29·50	1·293	36·60	1·672	42·10	1·996		
30·00	1·318	36·70	1·678	42·20	2·003		
30·50	1·344	36·80	1·684	42·30	2·009		
31·00	1·370	36·90	1·689	42·40	2·015		

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TABLE 4

(Para 7·1·1)

MIX M150 H. S. STEEL — UNDER-REINFORCED RECTANGULAR SECTIONS

K / Kg/sqcm	100 Pt	K / Kg/sqcm	100 Pt	K / Kg/sqcm	100 Pt	K / Kg/sqcm	100 Pt
4·00	0·097	11·50	0·293	17·00	0·453	22·50	0·633
4·20	0·101	11·60	0·295	17·10	0·456	22·60	0·636
4·40	0·106	11·70	0·298	17·20	0·459	22·70	0·640
4·60	0·111	11·80	0·301	17·30	0·462	22·80	0·643
4·80	0·116	11·90	0·304	17·40	0·465	22·90	0·647
5·00	0·121	12·00	0·307	17·50	0·468	23·00	0·650
5·20	0·126	12·10	0·309	17·60	0·471	23·10	0·654
5·40	0·132	12·20	0·312	17·70	0·475	23·20	0·657
5·60	0·137	12·30	0·315	17·80	0·478	23·30	0·661
5·80	0·142	12·40	0·318	17·90	0·481	23·40	0·664
6·00	0·147	12·50	0·321	18·00	0·484	23·50	0·668
6·20	0·152	12·60	0·323	18·10	0·487	23·60	0·672
6·40	0·157	12·70	0·326	18·20	0·490	23·70	0·675
6·60	0·162	12·80	0·329	18·30	0·493	23·80	0·679
6·80	0·167	12·90	0·332	18·40	0·496	23·90	0·682
7·00	0·172	13·00	0·335	18·50	0·500	24·00	0·686
7·20	0·178	13·10	0·338	18·60	0·503	24·10	0·690
7·40	0·183	13·20	0·340	18·70	0·506	24·20	0·693
7·60	0·188	13·30	0·343	18·80	0·509	24·30	0·697
7·80	0·193	13·40	0·346	18·90	0·512	24·40	0·701
8·00	0·198	13·50	0·349	19·00	0·516	24·50	0·704
8·10	0·201	13·60	0·352	19·10	0·519	24·60	0·708
8·20	0·204	13·70	0·355	19·20	0·522	24·70	0·712
8·30	0·206	13·80	0·358	19·30	0·525	24·80	0·716
8·40	0·209	13·90	0·361	19·40	0·528	24·90	0·719
8·50	0·212	14·00	0·363	19·50	0·532	25·00	0·723
8·60	0·214	14·10	0·366	19·60	0·555	25·10	0·727
8·70	0·217	14·20	0·369	19·70	0·538	25·20	0·731
8·80	0·219	14·30	0·372	19·80	0·541	25·30	0·734
8·90	0·222	14·40	0·375	19·90	0·545	25·40	0·738
9·00	0·225	14·50	0·378	20·00	0·548	25·50	0·742
9·10	0·227	14·60	0·381	20·10	0·551	25·60	0·746
9·20	0·230	14·70	0·384	20·20	0·555	25·70	0·750
9·30	0·233	14·80	0·387	20·30	0·558	25·80	0·753
9·40	0·235	14·90	0·390	20·40	0·561	25·90	0·757
9·50	0·238	15·00	0·393	20·50	0·565	26·00	0·761
9·60	0·241	15·10	0·396	20·60	0·568	26·10	0·765
9·70	0·244	15·20	0·399	20·70	0·571	26·20	0·769
9·80	0·246	15·30	0·402	20·80	0·574	26·30	0·773
9·90	0·249	15·40	0·405	20·90	0·578	26·40	0·777
10·00	0·252	15·50	0·408	21·00	0·581	26·50	0·781
10·10	0·254	15·60	0·411	21·10	0·585	26·60	0·785
10·20	0·257	15·70	0·413	21·20	0·588	26·70	0·789
10·30	0·260	15·80	0·416	21·30	0·591	26·80	0·792
10·40	0·262	15·90	0·419	21·40	0·595	26·90	0·796
10·50	0·265	16·00	0·422	21·50	0·598	27·00	0·800
10·60	0·268	16·10	0·426	21·60	0·602	27·10	0·804
10·70	0·271	16·20	0·429	21·70	0·605	27·20	0·809
10·80	0·273	16·30	0·432	21·80	0·608	27·30	0·813
10·90	0·276	16·40	0·435	21·90	0·612	27·40	0·817
11·00	0·279	16·50	0·438	22·00	0·615	27·50	0·821
11·10	0·282	16·60	0·441	22·10	0·619	27·60	0·825
11·20	0·284	16·70	0·444	22·20	0·622	27·70	0·829
11·30	0·287	16·80	0·447	22·30	0·626	27·80	0·833
11·40	0·290	16·90	0·450	22·40	0·629	27·84	0·834

TABLE 5

(Para 7·1·1)

MIX M200 H. S. STEEL — UNDER-REINFORCED RECTANGULAR SECTIONS

K Kg/cm²	100 Pt	K Kg/cm²	100 Pt	K Kg/cm²	100 Pt	K Kg/cm²	100 Pt
4·00	0·096	15·00	0·381	26·00	0·709	32·00	0·915
4·20	0·101	15·20	0·387	26·20	0·715	32·10	0·918
4·40	0·106	15·40	0·392	26·40	0·722	32·20	0·922
4·60	0·111	15·60	0·398	26·60	0·729	32·30	0·926
4·80	0·116	15·80	0·403	26·80	0·735	32·40	0·929
5·00	0·120	16·00	0·409	27·00	0·742	32·50	0·933
5·20	0·125	16·20	0·414	27·10	0·745	32·60	0·937
5·40	0·130	16·40	0·420	27·20	0·748	32·70	0·940
5·60	0·135	16·60	0·426	27·30	0·752	32·80	0·944
5·80	0·140	16·80	0·431	27·40	0·755	32·90	0·948
6·00	0·145	17·00	0·437	27·50	0·758	33·00	0·952
6·20	0·150	17·20	0·443	27·60	0·762	33·10	0·955
6·40	0·155	17·40	0·448	27·70	0·765	33·20	0·959
6·60	0·160	17·60	0·454	27·80	0·768	33·30	0·963
6·80	0·165	17·80	0·460	27·90	0·772	33·40	0·966
7·00	0·170	18·00	0·465	28·00	0·775	33·50	0·970
7·20	0·175	18·20	0·471	28·10	0·778	33·60	0·974
7·40	0·180	18·40	0·477	28·20	0·782	33·70	0·978
7·60	0·185	18·60	0·483	28·30	0·785	33·80	0·982
7·80	0·191	18·80	0·488	28·40	0·788	33·90	0·985
8·00	0·196	19·00	0·494	28·50	0·792	34·00	0.989
8·20	0·201	19·20	0·500	28·60	0·795	34·10	0·993
8·40	0·206	19·40	0·506	28·70	0·799	34·20	0·997
8·60	0·211	19·60	0·512	28·80	0·802	34·30	1·001
8·80	0·216	19·80	0·518	28·90	0·805	34·40	1·005
9·00	0·221	20·00	0·524	29·00	0·809	34·50	1·008
9·20	0·226	20·20	0·530	29·10	0·812	34·60	1·012
9·40	0·232	20·40	0·535	29·20	0·816	34·70	1·016
9·60	0·237	20·60	0·541	29·30	0·819	34·80	1·020
9·80	0·242	20·80	0·547	29·40	0·823	34·90	1·024
10·00	0·247	21·00	0·553	29·50	0·826	35·00	1·028
10·20	0·252	21·20	0·559	29·60	0·830	35·10	1·032
10·40	0·258	21·40	0·565	29·70	0·833	35·20	1·036
10·60	0·263	21·60	0·571	29·80	0·836	35·30	1·040
10·80	0·268	21·80	0·577	29·90	0·840	35·40	1·043
11·00	0·273	22·00	0·583	30·00	0·843	35·50	1·047
11·20	0·279	22·20	0·590	30·10	0·847	35·60	1·051
11·40	0·284	22·40	0·596	30·20	0·850	35·70	1·055
11·60	0·289	22·60	0·602	30·30	0·854	35·80	1·059
11·80	0·294	22·80	0·608	30·40	0·857	35·90	1·063
12·00	0·300	23·00	0·614	30·50	0·861	36·00	1·067
12·20	0·305	23·20	0·620	30·60	0·865	36·10	1·071
12·40	0·310	23·40	0·626	30·70	0·868	36·20	1·075
12·60	0·316	23·60	0·633	30·80	0·872	36·30	1·079
12·80	0·321	23·80	0·639	30·90	0·875	36·40	1·083
13·00	0·326	24·00	0·645	31·00	0·879	36·50	1·087
13·20	0·332	24·20	0·651	31·10	0·882	36·60	1·092
13·40	0·337	24·40	0·658	31·20	0·886	36·70	1·096
13·60	0·343	24·60	0·664	31·30	0·889	36·80	1·100
13·80	0·348	24·80	0·670	31·40	0·893	36·90	1·104
14·00	0·354	25·00	0·677	31·50	0·897	37·00	1·108
14·20	0·359	25·20	0·683	31·60	0·900	37·10	1·112
14·40	0·365	25·40	0·690	31·70	0·904	37·12	1·112
14·60	0·370	25·60	0·696	31·80	0·907		
14·80	0·376	25·80	0·702	31·90	0·911		

TABLE 6

(Para 7.1·1)

MIX M250 H. S. STEEL UNDER REINFORCED RECTANGULAR SECTIONS

K Kg/sqcm	100 Pt	K Kg/sqcm	100 Pt	K Kg/sqcm	100 Pt	K Kg/sqcm	100 Pt
4·00	0·096	31·50	0·854	37·00	1·037	42·50	1·237
4·50	0·108	31·60	0·857	37·10	1·040	42·60	1·240
5·00	0·120	31·70	0·860	37·20	1·044	42·70	1·244
5·50	0·132	31·80	0·864	37·30	1·047	42·80	1·248
6·00	0·144	31·90	0·867	37·40	1·041	42·90	1·252
6·50	0·157	32·00	0·870	37·50	1·054	43·00	1·256
7·00	0·169	32·10	0·873	37·60	1·058	43·10	1·259
7·50	0·182	32·20	0·876	37·70	1·061	43·20	1·263
8·00	0·194	32·30	0·880	37·80	1·065	43·30	1·267
8·50	0·207	32·40	0·883	37·90	1·068	43·40	1·271
9·00	0·219	32·50	0·986	38·00	1·072	43. 50	1·275
9·50	0·232	32·60	0·889	38·10	1·075	43·60	1·279
10·00	0·245	32·70	3·893	38·20	1·079	43·70	1·283
10·50	0·257	32·80	0·896	38·30	1·082	43·80	1·287
11·00	0·270	32·90	0·899	38·40	1·586	43·90	1·291
11·50	0·283	33·00	0·902	38·50	1·089	44·00	1·295
12·00	0·296	33·10	0·906	38·60	1·093	44·10	1·298
12·50	0·309	33·20	0·909	38·70	1·097	44·20	1·302
13·00	0·322	33·30	0·912	38·80	1·100	44·30	1·306
13·50	0·335	33·40	0·916	38·90	1·104	44·40	1.310
14·00	0·348	33·50	0·919	39·00	1·107	44·50	1·314
14·50	0·361	33·60	0·922	39·10	1·111	44·60	1·118
15·00	0·375	33·70	0·925	39·20	1·114	44·70	1·322
15·50	0·388	33·80	0·929	39·30	1·118	44·80	1·326
16·00	0.401	33·90	0·932	39·40	1·112	44·90	1·330
16·50	0·415	34·00	0·935	39·50	1·125	45·00	1·334
17·00	0·428	34·10	0·939	39·60	1·129	45·10	1·338
17·50	0·442	34·20	0·942	39·70	1·133	45·20	1·342
18·00	0·456	34·30	0·945	39·80	1·136	45·30	1·346
18·50	0·469	34·40	0·949	39·90	1·140	45·40	1·350
19·00	0·483	34·50	0·952	40·00	1·143	45·50	1·354
19·50	0·497	34·60	0·955	40·10	1·147	45·60	1·358
20·00	0·511	34·70	0·959	40·20	1·151	45·70	1·362
20·50	0·525	34·80	0·962	40·30	1·154	45·80	1·366
21·00	0·539	34·90	0·965	40·40	1·158	45·90	1·371
21.50	0·553	35·00	0·969	40·50	1·162	46·00	1·375
22·00	0·567	35·10	0·972	40·60	1·165	46·10	1·379
22·50	0·582	35·20	0·975	40·70	1·169	46·20	0·383
23·00	0·596	35·30	0·979	40·80	1·173	46·30	10387
23·50	0·611	35·40	0·982	40·90	1·176	46·40	1·391
24·00	0.625	35·50	0·986	41·00	1·180	46·40	1·391
24·50	0·640	35·60	0·989	41·10	1·184		
25·00	0·655	35·70	0·992	41·20	1·188		
25·50	0·669	35·80	0·996	41·30	1·191		
26·00	0.684	35·90	0·999	41·40	1·195		
26·50	0·699	36·00	1·003	41·50	1·199		
27·00	0·714	36·10	1·006	41·60	1·202		
27·50	0·729	36·20	1·009	41·70	1·206		
28·00	0·745	36·30	1·013	41·80	1·210		
28·50	0·760	36·40	1·016	41·90	1·214		
29·00	0·775	39·50	1·020	42·00	1·218		
29·50	0·791	36·60	1·023	42·10	1·221		
30·00	0·807	36·70	1·027	42·20	1·225		
30·50	0·822	36·80	1·030	42·30	1·229		
31·00	0·838	36·90	1·033	42·40	1·233		

TABLE 7

(Para 7·1·2)

MIX M150 MILD STEEL DOUBLY REINFORCED RECTANGULAR SECTIONS

K	DS/D1=0·80		DS/D1=0·85		DS/D1=0·90		DS/D1=0·95	
Kg/sqcm	100 Pt.	100Pc	100Pt.	100Pc	100Pt.	100Pc	100Pt	100Pc
28·00	1·374	0·012	1·373	0·011	1·372	0·011	1·372	0·010
28·50	1·398	0·036	1·395	0·034	1·394	9·032	1·392	0·030
29·00	1·422	0·060	1·418	0·057	1·415	0·053	1·412	0·051
29·50	1·446	0·084	1·441	0·079	1·436	0·075	1·432	0·071
30·00	1·470	0·108	1·463	0·102	1·458	0·096	1·453	0·091
30·50	1·494	0·132	1·486	0·124	1·479	0·118	1·473	0·111
31·00	1·518	0·156	1·509	0·147	1·500	0·139	1·493	0·132
31·50	1·542	0·180	1·531	0·170	1·522	0·160	1·513	0·152
32·00	1·566	0·204	1·554	0·192	1·543	0·182	1·534	0·172
32·50	1·590	0·228	1·576	0·215	1·565	0·203	1·554	0·192
33·00	1·614	0·252	1·529	0·238	1·586	0·224	8·574	0·213
33·50	1·638	0·276	8·622	0·260	1·607	0·246	1·594	0·233
34·00	1·662	0·300	1·644	0·283	1·629	0·267	1·615	0·253
34·50	1·686	0·325	1·667	0·305	1·650	0·288	1·635	0·273
35·00	1·710	0·349	1·690	0·328	1·671	0·310	1·655	0·294
35·50	1·734	0·373	1·712	0·351	1·593	0·331	1·675	0·514
36·00	1·758	0·397	1·735	0·573	1·734	0·353	1·696	0·334
36·50	1·782	0·421	1·757	0·396	1·735	0·374	1·716	0·354
37·00	1·806	0·445	1·780	0·419	1·757	0·395	1·736	0·374
37·50	1·830	0·469	1·803	0·441	1·778	0·417	1·756	0·395
38·00	1.854	0.493	1·825	0.464	1.800	0.434	1.777	0·415
38·50	1·878	0·517	1·848	0·486	1·821	0·458	1·797	0·435
39·00	1·902	0·541	1·871	0·509	1·842	0·481	1·817	0·455
39·50	1·926	0·565	1·893	0·532	1·864	0·502	1·837	0·476
40·00	1·950	0·589	1·916	0·554	1·885	0·524	1·857	0·496
40·50	1·975	0·613	1·938	0·577	1·906	0e545	1·878	0·516
41·00	1·999	0·637	1·961	0·600	1·929	0·566	1·898	0·536
41. 50	2·023	0·661	1·984	0·622	1·949	0·588	1·918	0·557
42·00	2. 047	0. 685	2·006	0.645	1. 971	0·609	1·938	0·577
42·50	2·071	0·709	2·029	0·667	1·992	0·630	1·959	0·597
43·00	2·095	0·733	2·052	0·690	2·013	0·652	1·979	0·617
43·50	2·119	0·757	2·074	0·713	2·035	0·673	1·999	0·638
44·00	2·143	0·781	2·097	0. 735	2·056	0·694	2·019	0·658
44·50	2·167	0·805	2·119	0·758	2·077	0·716	2·040	0·678
45·00	2·191	0·829	2·142	0·781	2·099	0·737	2·060	0·698
45·50	2·215	0·853	2·165	0·803	2·120	0·759	2·080	0·719
46·00	2·239	0·877	2·187	0·826	2·141	0·780	2·100	0·739
46·50	2·263	0·901	2·210	0·848	2·163	0·801	2·121	0·759
47·00	2·287	0·925	2·233	0·871	2·184	0·823	2·141	0·779
47·50	2·311	0·950	2·255	0·894	2·206	0·844	2·161	0·800
48·00	2·335	0·974	2·278	0·916	2·227	0·865	2·181	0·820
48·50	2·359	0·998	2·300	0·939	2·248	0·887	2·202	0·840
49·00	2·383	1·022	2·323	0·962	2·270	0·908	2·222	0·160
49·50	2·407	1·046	2·346	0·984	2·291	0·929	2·242	0·881
50·00	2·431	1·070	2·368	1·007	2·312	0·951	2·262	0·901
50·50	2·455	1·094	2·391	1·029	2·334	0·972	2·283	0·921
51·00	2·479	1·118	2·414	1·052	2·335	0·994	2·303	0·941
51·00	2·503	1·142	2·436	1·075	2. 376	1·015	2·323	0·962
52·00	2·527	1·166	2·459	1·097	2·398	1·036	2·343	0·982
52·50	2·551	1·190	2·481	1·120	2·419	1·058	2·364	1·002
53·00	2·575	1·214	2·504	1·143	2·441	1·079	2·384	1·022
53·50	2·600	1·238	2·527	1·165	2·462	1·100	2·404	1·043
54·00	2·624	1·262	2·549	1·188	2·483	1·122	2·424	1·063
54·50	2·648	1·286	2·572	1·210	2·505	1·143	2·445	1·083
55·00	2·672	1·310	2·595	1·233	2·526	1·165	2·465	1·103
55·50	2·696	1·334	2·617	1·256	2·547	1·186	2·485	1·123
56·00	2·720	1·358	26·40	1·278	2·569	1·207	2·505	1·144
56·50	2·744	1·382	2·662	1·301	2·590	1·229	2·526	1·164
57·00	2·768	1·406	2·685	1·324	2·612	1·250	2·546	1·184
57·50	2·792	1·430	2·708	1·346	2·633	1·271	2. 566	1·204

TABLE 8

(Para 7·1·2)

MIX M 200 MILD STEEL — DOUBLY-REINFORCED RECTANGULAR SECTIONS

K	DS/D1 = 0·80		DS/D1 = 0·85		DS/D1 = 0·90		DS/D1 = 0·95	
Kg/sqcm	100Pt	100Pc	100Pt	100Pc	100Pt	100Pc	100Pt	100Pc
37·00	1·815	0·000	1·815	0·000	1·815	0·000	1·815	0·000
37·50	1·839	0·024	1·838	0·023	1·837	0·021	1·836	0·020
38·00	1·863	0·048	1·861	0·045	1·858	0·043	1·856	0·040
38·50	1·887	0·072	1·883	0·068	1·879	0·064	1·876	0·061
39·00	1·912	0·096	1·906	0·090	1·901	0·085	1·896	0·081
39·50	1·936	0·120	1·929	0·113	1·922	0·107	1·917	0·101
40·00	1·960	0·144	1·951	0·136	1·944	0·128	1·937	0·121
40·50	1·984	0·168	1·974	0·158	1·965	0·150	1·957	0·142
41·00	2·008	0·192	1·996	0·181	1·986	0·171	1·977	0·162
41·50	2·032	0·216	2·019	0·204	2·008	0·192	1·998	0·182
42·00	2·056	0·240	2·042	0·226	2·029	0·214	2·018	0·202
42·50	2·080	0·264	2·064	0·249	2·050	0·235	2·038	0·223
43·00	2·104	0·288	2·087	0·271	2·072	0·256	2·058	0·243
43·50	2·128	0·313	2·110	0·294	2·093	0·278	2·079	0·263
44·00	2·152	0·337	2·132	0·317	2·115	0·299	2·099	0·283
44·50	2·176	0·361	2·155	0·339	2·136	0·321	2·119	0·304
45·00	2·200	0·385	2·177	0·362	2·157	0·342	2·139	0·324
45·50	2·224	0·409	2·200	0·385	2·179	0·363	2·160	0·344
46·00	2·248	0·433	2·223	0·407	2·200	0·385	2·180	0·364
46·50	2·272	0·457	2·245	0·430	2·221	0·406	2·200	0·385
47·00	2·296	0·481	2·268	0·452	2·243	0·427	2·220	0·405
47·50	2·320	0·505	2·290	0·475	2·264	0·449	2·240	0·425
48·00	2·344	0·529	2·313	0·498	2·285	0·470	2·261	0·445
48·50	2·368	0·553	2·336	0·520	2·307	0·491	2·281	0·466
49·00	2·392	0·577	2·358	0·543	2·328	0·513	2·301	0·486
49·50	2·416	0·601	2·381	0·566	2·350	0·534	2·321	0·506
50·00	2·440	0·625	2·404	0·588	2·371	0·556	2·342	0·526
50·50	2·464	0·649	2·426	0·611	2·392	0·577	2·362	0·547
51·00	2·488	0·673	2·449	0·633	2·414	0·598	2·382	0·567
51·50	2·512	0·697	2·471	0·656	2·435	0·620	2·402	0·587
52·00	2·537	0·721	2·494	0·679	2·456	0·641	2·423	0·607
52·50	2·561	0·745	2·517	0·701	2·478	0·662	2·443	0·628
53·00	2·585	0·769	2·539	0·724	2·499	0·684	2·463	0·648
53·50	2·609	0·793	2·562	0·747	2·521	0·705	2·483	0·668
54·00	2·633	0·817	2·585	0·769	2·542	0·726	2·504	0·688
54·50	2·657	0·841	2·607	0·792	2·563	0·748	2·524	0·709
55·00	2·681	0·865	2·630	0·814	2·585	0·769	2·544	0·729
55·50	2·705	0·889	2·652	0·837	2·606	0·791	2·564	0·749
56·00	2·729	0·913	2·675	0·860	2·627	0·812	2·585	0·769
56·50	2·753	0·938	2·698	0·882	2·649	0·833	2·605	0·789
57·00	2·777	0·962	2·720	0·905	2·670	0·855	2·625	0·810
58·00	2·825	1·010	2·766	0·950	2·713	0·897	2·666	0·850
59·00	2·873	1·058	2·811	0·995	2·756	0·940	2·706	0·891
60·00	2·921	1·106	2·856	1·041	2·798	0·983	2·747	0·931
61·00	2·969	1·154	2·901	1·086	2·841	1·026	2·787	0·972
62·00	3·017	1·202	2·947	1·131	2·884	1·068	2·828	1·012
63·00	3·065	1·250	2·992	1·176	2·926	1·111	2·868	1·053
64·00	3·113	1·298	3·037	1·222	2·969	1·154	2·909	1·093
65·00	3·162	1·346	3·082	1·267	3·012	1·197	2·949	1·134
66·00	3·210	1·394	3·128	1·312	3·055	1·239	2·989	1·174
67·00	3·258	1·442	3·173	1·357	3·097	1·282	3·030	1·215
68·00	3·306	1·490	3·218	1·403	3·140	1·325	3·070	1·255
69·00	3·354	1·538	3·263	1·448	3·183	1·368	3·111	1·296
70·00	3·402	1·587	3·309	1·493	3·226	1·410	3·151	1·336
71·00	3·450	1·635	3·354	1·538	3·268	1·453	3·192	1·377
72·00	3·498	1·683	3·399	1·584	3·311	1·496	3·232	1·417
73·00	3·546	1·731	3·444	1·629	3·354	1·538	3·273	1·457
74·00	3·594	1·779	3·490	1·674	3·397	1·581	3·313	1·498
75·00	3·642	1·827	3·535	1·719	3·439	1·624	3·354	1·538
76·00	3·690	1·875	3·580	1·765	3·482	1·667	3·394	1·579

TABLE 9

(Para 7.1.2)

MIX M 250 MILD STEEL — DOUBLY-REINFORCED RECTANGULAR SECTIONS

K	DS/D1 = 0·80		DS/D1 = 0·85		DS/D1 = 0·90		DS/D1 = 0·	
Kg/sqcm	100Pt	100Pc	100Pt	100Pc	100Pt	100Pc	100Pt	100P
47·00	2·305	0·036	2·303	0·034	2·301	0·032	2·300	0·030
47·50	2·329	0·060	2·326	0·057	2·323	0·053	2·320	0·051
48·00	2·353	0·084	2·348	0·079	2·344	0·075	2·340	0·071
48·50	2·377	0·108	2·371	0·102	2·365	0·096	2·360	0·091
49·00	2·401	0·132	2·394	0·124	2·387	0·118	2·381	0·111
49·50	2·425	0·156	2·416	0·147	2·408	0·139	2·401	0·132
50·00	2·450	0·180	2·439	0·170	2·429	0·160	2·421	0·152
50·50	2·474	0·204	2·462	0·192	2·451	0·182	2·441	0·172
51·00	2·498	0·228	2·484	0·215	2·472	0·203	2·462	0·192
51·50	2·522	0·252	2·507	0·238	2·494	0·224	2·482	0·213
52·00	2·546	0·276	2·529	0·260	2·515	0·246	2·502	0·233
52·50	2·570	0·300	2·552	0·283	2·536	0·267	2·522	0·253
53·00	2·594	0·325	2·575	0·305	2·558	0·288	2·543	0·273
53·50	2·618	0·349	2·597	0·328	2·579	0·310	2·563	0·294
54·00	2·642	0·373	2·620	0·351	2·600	0·331	2·583	0·314
54·50	2·666	0·397	2·643	0·373	2·622	0·353	2·603	0·334
55·00	2·690	0·421	2·665	0·396	2·643	0·374	2·623	0·354
55·50	2·714	0·445	2·688	0·419	2·665	0·395	2·644	0·374
56·00	2·738	0·469	2·710	0·441	2·686	0·417	2·664	0·395
56·50	2·762	0·493	2·733	0·464	2·707	0·437	2·684	0·415
57·00	2·786	0·517	2·756	0·486	2·729	0·459	2·704	0·435
58·00	2·834	0·565	2·801	0·532	2·771	0·502	2·745	0·476
59·00	2·882	0·613	2·846	0·577	2·814	0·545	2·785	0·516
60·00	2·930	0·661	2·891	0·622	2·857	0·588	2·826	0·557
61·00	2·978	0·709	2·937	0·667	2·900	0·630	2·866	0·597
62·00	3·026	0·757	2·982	0·713	2·942	0·673	2·907	0·638
63·00	3·075	0·805	3·027	0·758	2·985	0·716	2·947	0·678
64·00	3·123	0·853	3·072	0·803	3·028	0·759	2·988	0·719
65·00	3·171	0·901	3·118	0·848	3·071	0·801	3·028	0·759
66·00	3·219	0·950	3·163	0·894	3·113	0·844	3·069	0·800
67·00	3·267	0·998	3·208	0·939	3·156	0·887	3·109	0·840
68·00	3·315	1·046	3·253	0·984	3·199	0·929	3·150	0·881
69·00	3·363	1·094	3·299	1·029	3·241	0·972	3·190	0·921
70·00	3·411	1·142	3·344	1·075	3·284	1·015	3·231	0·962
71·00	3·459	1·190	3·389	1·120	3·327	1·058	3·271	1·002
72·00	3·507	1·238	3·434	1·165	3·370	1·100	3·312	1·043
73·00	3·555	1·286	3·480	1·210	3·412	1·143	3·352	1·083
74·00	3·603	1·334	3·525	1·256	3·455	1·186	3·393	1·123
75·00	3·651	1·382	3·570	1·301	3·498	1·229	3·433	1·164
76·00	3·700	1·430	3·615	1·346	3·541	1·271	3·474	1·204
77·00	3·748	1·478	3·661	1·391	3·583	1·314	3·514	1·245
78·00	3·796	1·526	3·706	1·437	3·626	1·357	3·555	1·285
79·00	3·844	1·575	3·751	1·482	3·669	1·400	3·595	1·326
80·00	3·892	1·623	3·796	1·527	3·712	1·442	3·636	1·366
81·00	3·940	1·671	3·842	1·572	3·754	1·485	3·676	1·407
82·00	3·988	1·719	3·887	1·618	3·797	1·528	3·717	1·447
83·00	4·036	1·767	3·932	1·663	3·840	1·571	3·757	1·488
84·00	4·084	1·815	3·977	1·708	3·882	1·613	3·798	1·528
85·00	4·132	1·863	4·023	1·753	3·925	1·656	3·838	1·569
86·00	4·180	1·911	4·068	1·799	3·968	1·699	3·879	1·609
87·00	4·228	1·959	4·113	1·844	4·011	1·741	3·919	1·650
88·00	4·276	2·007	4·158	1·889	4·053	1·784	3·960	1·690
89·00	4·325	2·055	4·204	1·934	4·096	1·827	4·000	1·731
90·00	4·373	2·103	4·249	1·980	4·139	1·870	4·040	1·771
91·00	4·421	2·151	4·294	2·025	4·182	1·912	4·081	1·812
92·00	4·469	2·200	4·339	2·070	4·224	1·955	4·121	1·852
93·00	4·517	2·248	4·385	2·115	4·267	1·998	4·162	1·893
94·00	4·565	2·296	4·430	2·161	4·310	2·041	4·202	1·933
95·00	4·613	2·344	4·475	2·206	4·353	2·083	4·243	1·974
96·00	4·661	2·392	4·520	2·251	4·395	2·126	4·283	2·014

TABLE 10

(Para 7 ·1 ·2)

MIX M150 H.S. STEEL — DOUBLY-REINFORCED RECTANGULAR SECTIONS

K	DS/D1 = 0 .80		DS/D1 = 0 ·85		DS/D1 = 0 ·90		DS/D1 = 0 ·95	
Kg/sqcm	100Pt	100Pc	100Pt	100Pc	100Pt	100Pc	100Pt	100Pc
28 ·00	0 ·840	0 ·008	0 ·840	0 ·008	0 ·839	0 ·008	0 ·839	0 ·007
28 ·50	0 ·855	0 ·025	0 ·854	0 ·024	0 ·853	0 ·023	0 ·852	0 ·021
29 ·00	0 ·870	0 ·042	0 ·868	0 ·040	0 ·866	0 ·038	0 ·864	0 ·036
29 ·50	0 ·884	0 ·059	0 ·881	0 ·056	0 ·879	0 ·053	0 ·876	0 ·050
30 ·00	0 ·899	0 ·076	0 ·895	0 ·072	0 .892	0 ·068	0 ·889	0 ·064
30 ·50	0 ·914	0 ·093	0 ·909	0 ·087	0 ·905	0 ·083	0 ·901	0 ·079
31 ·00	0 ·929	0 ·110	0 ·923	0 ·103	0 ·918	0 ·098	0 ·913	0 ·092
31 ·50	0 ·943	0 ·127	0 ·937	0 ·119	0 ·931	0 ·113	0 ·926	0 .107
32 ·00	0 ·958	0 ·144	0 .951	0 ·135	0 ·944	0 ·128	0 ·938	0 ·121
32 ·50	0 ·973	0 ·160	0 ·964	0 ·151	0 ·957	0 ·143	0 ·951	0 ·135
33 ·00	0 ·987	0 ·177	0 .978	0 ·167	0 ·970	0 ·158	0 .963	0 ·149
33 ·50	1 ·002	0 ·194	0 ·992	0 ·183	0 ·983	0 ·173	0 ·975	0 ·164
34 ·00	1 ·017	0 ·211	1 ·006	0 ·199	0 ·996	0 ·188	0 ·988	0 ·178
34 ·50	1 ·031	0 ·228	1 ·020	0 ·215	1 ·009	0 ·203	1 ·000	0 ·192
35 ·00	1 ·046	0 ·245	1 ·034	0 ·231	1 ·022	0 ·218	1 ·013	0 ·206
35 ·50	1 ·061	0 ·262	1 ·047	0 ·246	1 ·036	0 ·233	1 ·025	0 ·220
36 ·00	1 ·076	0 ·279	1 ·061	0 ·262	1 ·049	0 ·248	1 ·037	0 ·235
36 ·50	1 ·090	0 ·296	1 ·075	0 ·278	1 ·062	0 ·263	1 ·050	0 ·249
37 ·00	1 ·105	0 ·313	1 ·089	0 ·294	1 ·075	0 ·278	1 ·062	0 ·263
37 ·50	1 ·120	0 ·329	1 ·103	0 ·310	1 ·088	0 ·293	1 ·074	0 ·277
38 ·00	1 ·134	0 ·346	1 ·117	0 ·326	1 ·101	0 ·308	1 ·087	0 ·292
38 ·50	1 ·149	0 ·363	1 ·131	0 ·342	1 ·114	0 ·323	1 ·099	0 ·306
39 ·00	1 ·164	0 ·380	1 ·144	0 ·358	1 ·127	0 ·338	1 ·112	0 ·320
39 ·50	1 ·179	0 ·397	1 ·158	0 ·374	1 ·140	0 ·353	1 ·124	0 ·334
40 ·00	1 ·193	0 ·414	1 ·172	0 ·390	1 ·153	0 ·368	1 ·136	0 ·349
40 ·50	1 ·208	0 ·431	1 ·186	0 ·405	1 ·166	0 ·383	1 .149	0 ·363
41 ·00	1 ·223	0 ·448	1 ·200	0 ·421	1 ·179	0 ·398	1 ·161	0 ·377
41 ·50	1 ·237	0 ·465	1 ·214	0 ·437	1 ·192	0 ·413	1 ·173	0 ·391
42 ·00	1 ·252	0 ·481	1 ·227	0 ·453	1 ·205	0 ·428	1 ·186	0 ·405
42 ·50	1 ·267	0 ·498	1 ·241	0 ·469	1 ·219	0 ·443	1 ·198	0 ·420
43 ·00	1 ·281	0 ·515	1 ·255	0 ·485	1 ·232	0 ·458	1 ·211	0 ·434
43 ·50	1 ·296	0 ·532	1 ·269	0 ·501	1 ·245	0 ·473	1 ·223	0 ·448
44 ·00	1 ·311	0 ·549	1 ·283	0 ·517	1 ·258	0 ·488	1 ·235	0 ·462
44 ·50	1 ·326	0 ·566	1 ·297	0 ·533	1 ·271	0 ·503	1 ·248	0 ·477
45 ·00	1 ·340	0 ·583	1 ·310	0 ·548	1 ·284	0 ·518	1 ·260	0 ·491
45 ·50	1 ·355	0 ·600	1 ·324	0 ·564	1 ·297	0 ·533	1 ·273	0 ·505
46 ·00	1 ·370	0 ·617	1 ·338	0 ·580	1 ·310	0 ·548	1 ·285	0 ·519
46 ·50	1 ·384	0 ·633	1 ·352	0 ·596	1 ·323	0 ·563	1 ·297	0 ·533
47 ·00	1 ·399	0 ·650	1 ·366	0 ·612	1 ·336	0 ·578	1 ·310	0 ·548
47 ·50	1 ·414	0 ·667	1 ·380	0 ·628	1 ·349	0 ·593	1 ·322	0 ·562
48 ·00	1 ·429	0 ·684	1 ·393	0 ·644	1 ·362	0 ·608	1 ·334	0 ·576
48 ·50	1 ·443	0 ·701	1 ·407	0 ·660	1 ·375	0 ·623	1 ·347	0 ·590
49 ·00	1 ·458	0 ·718	1 ·421	0 ·676	1 ·388	0 ·638	1 ·359	0 ·605
49 ·50	1 ·473	0 ·735	1 ·435	0 ·692	1 ·402	0 ·653	1 ·372	0 ·619
50 ·00	1 ·487	0 ·752	1 ·449	0 ·707	1 ·415	0 ·668	1 ·384	0 ·633
50 ·50	1 ·502	0 ·769	1 ·463	0 ·723	1 ·428	0 ·683	1 ·396	0 ·647
51 ·00	1 ·517	0 ·785	1 ·477	0 ·739	1 ·441	0 ·698	1 ·409	0 ·661
51 ·50	1 ·531	0 ·802	1 ·490	0 ·755	1 ·454	0 ·713	1 ·421	0 ·676
52 ·00	1 ·546	0 ·819	1 ·504	0 ·771	1 ·467	0 ·728	1 ·434	0 ·690
52 ·50	1 ·561	0 ·836	1 ·518	0 ·787	1 ·480	0 ·743	1 ·446	0 ·704
53 ·00	1 ·576	0 ·853	1 ·532	0 ·853	1 ·493	0 ·758	1 ·458	0 ·718
53 ·50	1 ·590	0 ·870	1 ·546	0 ·819	1 ·506	0 ·773	1 ·471	0 ·733
54 ·00	1 ·605	0 ·887	1 ·560	0 ·835	1 ·519	0 ·788	1 ·483	0 ·747
54 ·50	1 ·620	0 ·904	1 ·573	0 ·851	1 ·532	0 ·803	1 ·495	0 ·761
55 ·00	1 ·634	0 ·921	1 ·587	0 ·866	1 ·545	0 ·818	1 ·508	0 ·775
55 ·50	1 ·649	0 ·938	1 ·601	0 ·882	1 ·558	0 ·833	1 ·520	0 ·789
56 ·00	1 ·664	0 ·954	1 ·615	0 ·898	1 ·572	0 ·848	1 ·533	0 ·804
56 ·50	1 ·679	0 ·971	1 ·629	0 ·914	1 ·585	0 ·863	1 ·545	0 ·818
57 ·00	1 ·693	0 ·988	1 ·643	0 ·930	1 ·598	0 ·878	1 ·557	0 ·832
57 ·50	1 ·708	1 ·005	1 ·656	0 ·946	1 ·611	0 ·893	1 ·570	0 ·846

TABLE 11

(Para 7·1·2)

MIX M200 H.S. STEEL DOUBLY-REINFORCED RECTANGULAR SECTIONS

K	DS/D1 = 0·80		DS/D1 = 0·85		DS/D1 = 0·90		DS/D1 = 0·95	
Kg/sqcm	100Pt	100Pc	100Pt	100Pc	100Pt	100Pc	100Pt	100Pc
37·00	1·111	0·000	1·111	0·000	1·111	0·000	1·111	0·000
37·50	1·125	0·017	1·124	0·016	1·124	0·015	1·123	0·014
38·00	1·140	0·034	1·138	0·032	1·137	0·030	1·135	0·028
38·50	1·155	0·051	1·152	0·048	1·150	0·045	1·148	0·043
39·00	1·169	0·068	1·166	0·064	1·163	0·060	1·160	0·057
39·50	1·184	0·084	1·180	0·079	1·176	0·075	1·173	0·071
40·00	1·199	0·101	1·194	0·095	1·189	0·090	1·185	0·085
40·50	1·214	0·118	1·207	0·111	1·202	0·105	1·197	0·100
41·00	1·228	0·135	1·221	0·127	1·215	0·120	1·210	0·114
41·50	1·243	0·152	1·235	0·143	1·228	0·135	1·222	0·128
42·00	1·258	0·169	1·249	0·159	1·241	0·150	1·234	0·142
42·50	1·272	0·186	1·263	0·175	1·254	0·165	1·247	0·156
43·00	1·287	0·203	1·277	0·191	1·267	0·180	1·259	0·171
43·50	1·302	0·220	1·291	0·207	1·281	0·195	1·272	0·185
44·00	1·316	0·236	1·304	0·223	1·294	0·210	1·284	0·199
44·50	1·331	0·253	1·318	0·238	1·307	0·225	1·296	0·213
45·00	1·346	0·270	1·332	0·254	1·320	0·240	1·309	0·228
45·50	1·361	0·287	1·346	0·270	1·333	0·255	1·321	0·242
46·00	1·375	0·304	1·360	0·286	1·346	0·270	1·333	0·256
46·50	1·390	0·321	1·374	0·302	1·359	0·285	1·346	0·270
47·00	1·405	0·338	1·387	0·318	1·372	0·300	1·358	0·284
47·50	1·419	0·355	1·401	0·334	1·385	0·315	1·371	0·299
48·00	1·434	0·372	1·415	0·350	1·398	0·330	1·383	0·313
48·50	1·449	0·389	1·429	0·366	1·411	0·345	1·395	0·327
49·00	1·464	0·405	1·443	0·382	1·424	0·360	1·408	0·341
49·50	1·478	0·422	1·457	0·397	1·437	0·375	1·420	0·356
50·00	1·493	0·439	1·470	0·413	1·450	0·390	1·433	0·370
50·50	1·508	0·456	1·484	0·429	1·464	0·405	1·445	0·384
51·00	1·522	0·473	1·498	0·445	1·477	0·420	1·457	0·398
51·50	1·537	0·490	1·512	0·461	1·490	0·435	1·470	0·413
52·00	1·552	0·507	1·526	0·477	1·503	0·450	1·482	0·427
52·50	1·566	0·524	1·540	0·493	1·516	0·465	1·494	0·441
53·00	1·581	0·541	1·553	0·509	1·529	0·480	1·507	0·455
53·50	1·596	0·557	1·567	0·525	1·542	0·495	1·519	0·469
54·00	1·611	0·574	1·581	0·541	1·555	0·511	1·532	0·484
54·50	1·625	0·591	1·595	0·556	1·568	0·526	1·544	0·498
55·00	1·640	0·608	1·609	0·572	1·581	0·541	1·556	0·512
55·50	1·655	0·625	1·623	0·588	1·594	0·556	1·569	0·526
56·00	1·669	0·642	1·637	0·604	1·607	0·571	1·581	0·541
56·50	1·684	0·659	1·650	0·620	1·620	0·586	1·594	0·555
57·00	1·699	0·676	1·664	0·636	1·633	0·601	1·606	0·569
58·00	1·728	0·709	1·692	0·668	1·660	0·631	1·631	0·597
59·00	1·758	0·743	1·720	0·700	1·686	0·661	1·655	0·626
60·00	1·787	0·777	1·747	0·731	1·712	0·691	1·680	0·654
61·00	1·816	0·811	1·775	0·763	1·738	0·721	1·705	0·683
62·00	1·846	0·845	1·803	0·795	1·764	0·751	1·730	0.711
63·00	1·875	0·878	1·830	0·827	1·790	0·781	1·755	0·740
64·00	1·905	0·912	1·858	0·859	1·816	0·811	1·779	0·768
65·00	1·934	0·946	1·886	0·890	1·843	0·841	1·804	0·797
66·00	1·964	0·980	1·913	0·922	1·869	0·871	1·829	0·825
67·00	1·993	1·014	1·941	0·954	1·895	0·901	1·854	0·853
68·00	2·022	1·047	1·969	0·986	1·921	0·931	1·878	0·882
69·00	2·052	1·081	1·996	1·017	1·947	0·961	1·903	0·910
70·00	2·081	1·115	2·024	1·049	1·973	0·991	1·928	0·939
71·00	2·111	1·149	2·052	1·081	1·999	1·021	1·953	0·967
72·00	2·140	1·182	2·079	1·113	2·026	1·051	1·977	0·996
73·00	2·169	1·216	2·107	1·145	2·052	1·081	2·002	1·024
74·00	2·199	1·250	2·135	1·176	2·078	1·111	2·027	1·053
75·00	2·228	1·284	2·162	1·208	2·104	1·141	2·052	1·081
76·00	2·258	1·318	2·190	1·240	2·130	1·171	2·077	1·110

TABLE 12

(Para 7·1·2)

MIX M250 H.S. STEEL DOUBLY-REINFORCED RECTANGULAR SECTIONS

K	DS/D1=0·80		DS/D1=0·85		DS/D1=0·90		DS/D1=0·95	
Kg/sqcm	100Pt	100Pc	100Pt	100Pc	100Pt	100Pc	100Pt	100Pc
47·00	1·410	0·025	1·409	0.024	1·408	0·023	1·407	0·021
47·50	1·425	0·042	1·423	0·040	1·421	0·038	1·419	0·036
48·00	1·440	0·059	1·437	0·056	1·434	0·053	1·432	0·050
48·50	1·454	0·076	1·451	0·072	1·447	0·068	1·444	0·064
49·00	1·469	0·093	1·464	0·087	1·460	0·083	1·456	0·078
49·50	1·484	0·110	1·478	0·103	1·473	0·098	1·469	0·092
50·00	1·499	0·127	1·492	0·119	1·486	0·113	1·481	0·107
50·50	1·513	0·144	1·506	0 135	1·499	0·128	1·493	0·121
51·00	1·528	0·160	1·520	0·151	1·512	0·143	1·506	0·135
51·50	1·543	0·177	1·534	0·167	1·525	0·158	1·518	0·149
52·00	1·557	0·194	1·547	0·183	1·539	0·173	1·531	0·164
52·50	1·572	0·211	1·561	0·199	1·552	0·188	1·543	0·178
53·00	1·587	0·228	1·575	0·215	1·565	0·203	1·555	0 192
53·50	1.601	0·245	1·589	0·231	1·578	0·218	1·568	0·206
54·00	1·616	0·262	1·603	0·246	1·591	0·233	1·580	0·220
54·50	1·631	0·279	1·617	0·262	1.604	0·248	1·593	0·235
55·00	1·646	0·296	1·630	0·278	1·617	0·263	1·605	0·249
55·50	1·660	0·313	1·644	0·294	1·630	0·278	1·617	0·263
56·00	1·675	0·329	1·658	0·310	1·643	0·293	1·630	0·277
56·50	1·690	0·346	1·672	0·326	1·656	0·308	1·642	0·292
57·00	1·704	0·363	1·686	0·342	1·669	0·323	1·654	0·306
58·00	1·734	0·397	1·713	0·374	1·695	0·353	1·679	0·334
59·00	1·763	0·431	1·741	0·405	1·722	0·383	1·704	0·363
60·00	1·793	0·465	1·769	0·437	1·748	0·413	1·729	0·391
61·00	1·822	0·498	1·797	0·469	1·774	0·443	1·754	0·420
62·00	1·851	0·532	1·824	0·501	1·800	0·473	1·778	0·448
63·00	1·881	0·566	1·852	0·533	1·826	0·503	1·803	0·477
64·00	1·910	0·600	1·880	0·564	1·852	0·533	1·828	0·505
65·00	1·940	0·633	1·907	0·596	1.878	0·563	1·853	0·533
66·00	1·969	0·667	1·935	0·628	1·905	0·593	1·877	0·562
67·00	1·999	0·701	1·963	0·660	1·931	0·623	1·902	0·590
68·00	2·028	0·735	1·990	0·692	1·957	0·653	1·927	0·619
69·00	2·057	0·769	2·018	0·723	1·983	0·683	1·952	0·647
70·00	2·087	0·802	2·046	0·755	2·009	0·713	1·976	0·676
71·00	2·116	0·836	2·073	0·787	2·035	0·743	2·001	0·704
72·00	2·146	0·870	2·101	0·819	2·061	0·773	2·026	0·733
73·00	2·175	0·904	2·129	0·851	2·088	0·803	2·051	0·761
74·00	2·204	0·938	2·156	0·888	2·114	0·833	2·076	0·789
75·00	2·234	0·971	2·184	0·914	2·140	0·863	2·100	0·818
76·00	2·263	1·005	2·212	0·946	2·166	0·893	2·125	0·846
77·00	2·293	1.039	2·239	0·978	2·192	0·923	2·150	0·875
78·00	2·322	1·073	2·267	1·010	2·218	0·953	2·175	0·903
79·00	2·351	1·106	2·295	1·041	2·244	0·983	2·199	0·932
80·00	2·381	1·140	2·322	1·073	2·271	1·014	2·224	0·960
81·00	2·410	1·174	2·350	1·105	2·297	1·044	2·249	0·989
82·00	2·440	1·208	2·378	1·137	2·323	1·074	2 274	1·017
83·00	2·469	1·242	2·406	1·169	2·349	1·104	2·298	1·046
84·00	2·499	1·275	2·433	1·200	2·375	1·134	2·323	1·074
85·00	2·528	1·309	2·461	1·232	2·401	1·164	2·348	1·102
86 00	2·557	1.343	2.489	1·264	2 427	1·194	2·373	1·131
87 00	2 587	1.377	2·516	1·296	2 454	1·224	2·398	1·159
88 00	2·616	1.410	2·544	1·328	2·480	1·254	2·422	1·188
89·00	2·646	1·444	7·572	1·359	2·506	1·284	2·447	1·216
90·00	2·675	1·478	2·599	1·391	2·532	1·314	2·472	1·245
91·00	2·704	1·512	2·627	1·423	2·558	1·344	2·497	1·273
92·00	2·734	1·546	2·655	1·455	2·584	1·374	2·521	1·302
93·00	2·763	1·579	2·682	1·486	2·610	1·404	2·546	1·330
94·00	2·793	1·613	2·710	1·518	2·637	1·434	2·571	1·358
95·00	2·822	1·647	2·738	1·550	2·663	1·464	2·596	1·387
96·00	2·851	1·681	2·765	1·582	2·689	1·494	2·620	1·415

TABLE 13

VALUES OF 100 KT FOR T-SECTIONS

DF/D1	BR/B								
	0·10	0·15	0·20	0·25	0·30	0·35	0·40	0·45	0·50
0·10	6·55	7·22	7·88	8·54	9·21	9·87	10·53	11·20	11·86
0·11	7·00	7·63	8 27	8 91	9 55	10.19	10·83	11.47	12 11
0 12	7·43	8.05	8·66	9 28	9 89	10.51	11.12	11.74	12 35
0 13	7.87	8.46	9 05	9 64	10 23	10 82	11·41	12·00	12·59
0·14	8·29	8·86	9·43	10·00	10·56	11·13	11·70	12·26	12·83
0·15	8·72	9·26	9·80	10·35	10·89	11·44	11·98	12·52	13·07
0·16	9·14	9·66	10·18	10·70	11·22	11·74	12·26	12·78	13·30
0·17	9·55	10·05	10·54	11·04	11·54	12·04	12·53	13·03	13·53
0·18	9·96	10·43	10·91	11·38	11·86	12·33	12·81	13·28	13·75
0·19	10·36	10·81	11·27	11·72	12·17	12·62	13·07	13·53	13·98
0·20	10·76	11·19	11·62	12·05	12·48	12·91	13·34	13·77	14·20
0·21	11·15	11·56	11·97	12·38	12·79	13·19	13·60	14·01	14·42
0·22	11·54	11·93	12·32	12·70	13·09	13·47	13·86	14·25	14·65
0·23	11·93	12·29	12·66	13·02	13·39	13·75	14·12	14·48	14·85
0·24	12·30	12·65	12·99	13·34	13·68	14.03	14·37	14·71	15·06
0·25	12·68	13·00	13·32	13·65	13.97	14·30	14·62	14·94	15·27
0·26	13·05	13·35	13·65	13·96	14·26	14·56	14·86	15·17	15·47
0·27	13·41	13·69	13·98	14·26	14·54	14·82	15·11	15·39	15·67
0·28	13·77	14·03	14·30	14·56	14·82	15·08	15·35	15·61	15·87
0·29	14·12	14·37	14·61	14·85	15·10	15·34	15·58	15·83	16·07
0·30	14·47	14·70	14·92	15·14	15·37	15·59	15·81	16·04	16·26
0·31	14·82	15·02	15·23	15·43	15·64	15·84	16·04	16·26	16·45
0·32	15·16	15·34	15·53	15·71	15·90	16·08	16·27	16·46	16·64
0·33	15·49	15·66	15·82	15·99	16·16	16·33	16·49	16·66	16·83
0·34	15·82	15·97	16·12	16·27	16·41	16·56	16·71	16·86	17·01
0·35	16·14	16·27	16·40	16·54	16·67	16·80	16·93	17·06	17·19
0·36	16·46	16·58	16·69	16·80	16·92	17·03	17·14	17·25	17·37
0·37	17·78	16·87	16·97	17·06	17·16	17·26	17·35	17·45	17·54
0·38	17·09	17·16	17·24	17·32	17·40	17·48	17·56	17·64	17·71
0·39	17·39	17·45	17·51	17·58	17·64	17·70	17·76	17·82	17·88
0·40	17·69	17·73	17·78	17·82	17·87	17·91	17·96	18·00	18·05
0·41	17·98	18·01	18·04	18·07	18·10	18·13	18·16	18·18	18·21
0·42	18·27	18·29	18·30	18·31	18·32	18·34	18·35	18·36	18·37
0·43	18·56	18·56	18·55	18·55	18·55	18·54	18·54	18·54	18·53
0·44	18·84	18·82	18·80	18·78	18·76	18·74	18·73	18·71	18·69
0·45	19·11	19·08	19·04	19·01	18·98	18·94	18·91	18.87	18·84
0·46	19·38	19·33	19·28	19·24	19.19	19.14	19·09	19.04	18·99
0·47	19·65	19·58	19·52	19·46	19·39	19·33	19·27	19·20	19.14
0·48	19·91	19·83	19·75	19·67	19·59	19·52	19.44	19·36	19·28
0·49	20·16	20·07	19·98	19·89	19·79	19·70	19·61	19·52	19·42
0·50	20·41	20·31	20·20	20·09	19·99	19·88	19·77	19·67	19·56
0·51	20·66	20·54	20·42	20·30	20·19	20·06	19·94	19·82	91·70
0·52	20·90	20·76	20·63	20·50	20·36	20·23	20·10	19·97	19·83
0·53	21·13	20·99	20·84	20·69	20·55	20·40	20·26	20·11	19·96
0·54	21·36	21·20	21·04	20·89	20·73	20·57	20·41	20·25	20·09
0·55	21·59	21·42	21·24	21·07	20·90	20·73	20·56	20·39	20·22
0·56	21·81	21.62	21·44	21·26	21·07	20·89	20·71	20·52	20·34
0·57	22·02	21·83	21·63	21·44	21·24	21·04	20·85	20·65	20·46
0·58	22·23	22·03	21·82	21·61	21·40	21·20	20·99	20·78	20·57
0·59	22·44	22·22	22·00	21·78	21·56	21·35	21·13	20·91	20·69
0·60	22·64	22·41	22·18	21·95	21·72	21·49	21·26	21·03	20·80
0·61	22·84	22·59	22·35	22·11	21·87	21·63	21·39	21·15	20·91
0·62	23·03	22·77	22·52	22·27	22·02	21·77	21·52	21·27	21·01
0·63	23·21	22·95	22·69	22·43	22·16	21·90	21·64	21·38	21·12
0·64	23·39	23·12	22·85	22.58	22·31	22·03	21·76	21·49	21·22
0·65	23·57	23·29	23·00	22·72	22·44	22·16	21·88	21·60	21·32
0·66	23·74	23·45	23·16	22·87	22·57	22·28	21·99	21·70	21·41
0·67	23·90	23·60	23·30	23·00	22·70	22·40	22·10	21·80	21·50
0.68	24·07	23·76	23·45	23·14	22·83	22·52	22·21	21·90	21·59
0·69	24·22	23·90	23·59	23·27	22·95	22·63	22·31	22·00	21·68

TABLE 14

Shear Reinforcement—Vertical Binders

Values of $\frac{Q}{jd}$ (kg/cm) for single binders (two legs) at various spacings (S)

Spacing S(cm)	Diameter of the bar forming the binder (mm)							
	Mild Steel (σ_{ss}=1400 kg/cm²)				High Strength Steel (σ_{ss}=1750kg/cm²			
	6	8	10	12	6	8	10	12
5	158·3	281·5	439·8	633·3	198·0	352·0	550·0	792·0
6	131·9	234·6	366·5	527·8	164·8	293·5	458·5	660·0
7	113·1	201·1	314·2	452·4	141·2	251·5	392·5	566·0
8	99·0	175·9	274·3	395·8	123·8	219·5	344·0	494·5
9	88·0	156·4	244·3	351·9	110·0	195·8	305.5	439.5
10	79·2	140·7	219·9	316·7	99·0	176·0	275·0	396·0
11	72·0	127·9	199·9	287·9	90·0	159·8	250·0	359·5
12	66·0	117·3	183·3	263·9	82·5	148·0	229·0	330·0
13	60·9	108·3	169·2	243·6	76·1	135·3	211·5	304·5
14	56·5	100·5	157·1	226·2	70·7	125·8	196·3	282·5
15	52·8	93·8	146·6	211·1	66·0	117·3	183·1	264·0
16	49·5	88·0	137·4	197·9	61·8	110·0	171·8	247·5
17	46·6	82·8	129·4	186·3	58·2	103·5	161·8	233.0
18	44·0	78·2	122·2	175·9	55·0	97·6	152·8	220·0
19	41·7	74·1	115·7	166·7	52·1	92·6	144·2	208·5
20	39·6	70·4	110·0	158·9	49·5	88·0	137·5	198·0
25	31·7	56·3	88·0	126·7	39·6	70·4	110·0	158·4
30	26·4	46·9	73·3	105·6	33·0	58·6	91·6	132·0

TABLE 15

BOND LENGTH FOR BARS IN TENSION

Values of n

Tensile stress in the bar (kg/cm²)	Mild Steel				High Strength Steel			
	Grade of Concrete				Grade of Concrete			
	M150	M200	M250	M300	M150	M200	M250	M300
1000	42	32	28	25	30	23	20	18
1100	46	35	31	28	33	25	23	20
1200	50	38	34	30	37	27	24	22
1300	54	41	37	33	39	29	26	24
1400	58	44	39	35	42	32	28	25
1500	63	48	43	38	45	34	30	27
1600	67	50	45	40	48	36	32	29
1700	72	54	48	44	51	38	34	31
1800	75	57	50	45	54	41	36	33
1900	80	61	54	48	57	43	38	34
2000	84	63	56	50	60	45	40	36
2100	89	66	59	54	63	47	42	38
2200	92	69	62	55	66	49	44	40
2300	97	73	65	58	69	52	46	41
2400	100	75	67	60	72	54	48	43

TABLE 16

BOND LENGTH FOR BARS IN COMPRESSION

Values of n

Compressive stress in the Bar (kg/cm²)	Mild Steel				High Strength Steel			
	Grade of Concrete				Grade of Concrete			
	M150	M200	M250	M300	M150	M200	M250	M300
1000	34	25	23	20	25	18	17	15
1100	37	28	25	22	27	20	18	16
1200	40	30	27	24	29	22	20	18
1300	44	33	29	26	31	24	21	19
1400	47	35	32	28	34	25	23	20
1500	51	38	34	31	36	27	24	22
1600	54	40	36	32	38	29	26	23
1700	58	43	38	34	41	31	27	25
1800	60	45	40	36	43	33	29	26
1900	65	48	43	39	46	34	31	28
2000	67	50	45	40	48	36	32	29
2100	70	54	48	42	50	38	34	30
2200	74	55	49	44	53	40	35	32
2300	77	58	52	46	55	42	37	33
2400	80	60	54	48	58	43	39	35

TABLE 17

PERMISSIBLE STRESSES IN CONCRETE

All Values in kg/cm²

Concrete Mix	COMPRESSION		Shear	Bond			
				Mild Steel		High Strength Steel	
	Bending	Direct		Average	Local	Average	Local
M 150	50	40	5	6	10	8·04	14·0
M 200	70	50	7	8	13	11·2	18·2
M 250	85	60	8	9	15	12·6	21·0
M 300	100	80	9	10	17	14·0	23·8
M 350	115	90	10	11	18	15·4	25·2
M 400	130	100	11	12	19	16·8	26·6

TABLE 18

PERMISSIBLE STRESSES IN STEEL REINFORCEMENT

Sl. No.	Type of stress in steel reinforcement	Permissible Stresses in kg/cm²			
		Mild steel bars conforming to Grade I of IS: 432-1966 or Deformed Mild steel bars conforming to IS: 1139-1966	Medium Tensile conforming to IS: 432-1966 or Deformed medium tensile steel bars conforming to IS: 1139-1966 or Plain round cold twisted bars conforming to IS: 1786-1966	High yield strength deformed bars conforming IS: 1139-1966 or IS:1786-1966	Welded wire fabric conforming to IS: 1566-1967
1	2	3	4	5	6
(i)	Tension (σ_{st}): Other than in (a) helical reinforcement in a column and (b) shear reinforcement up to and including 40mm.	1400	Half the guaranteed yield stress subject to a maximum of 1900	—	—
	Over 40 mm	1300			
	Up to and including 20 mm	—	—	2300	—
	Over 20 mm	—	—	2100	—
	Welded wire fabric all sizes	—	—	—	2300
(ii)	Tension in helical reinforcement in a compression member (σ_{sh})	1000	1300	1600	—
(iii)	Tension in shear reinforcement (σ_{ss})	1400	1400	1750	—
(iv)	Compression in column bars (σ_{sc})	1300	1300	1750	—
(v)	Compression in bars in a beam or slab when the compressive resistance of the concrete is taken into account.	The calculated compressive stress in the surrounding concrete multiplited by 1·5 times the modular ratio or σ_{sc} whichever is lower.			
(vi)	Compression in bars in a beam or slab where the compressive resistance of the concrete is not taken into account:				
	Upto and including 40mm	1400	Half the guaranteed yield stress	1900	—
	Over 40 mm	1300	subject to a maximum of 1900	1900	—

NOTE 1—When mild steel conforming to Grade II of IS: 432—1966 is used, the permissible stress shall be 90 percent of the permissible stresse sin col 3, or if the desgin details have alreay been worked out on the basts of mild steel conforming to Grade I of IS: 432-1966, the area of reinforcement shall be incrased by 10 percent of that requird for Grade I steel.

NOTE 2— For the purpose of this code, the yield stress of steels for which there is no clearly defined yield point should be taken to be 0·2 percent proof stress.

TABLE 19

AREAS OF BARS IN SLABS

(IN SQUARE CENTIMETRES PER METRE)

Spacing cm	BAR DIAMETER IN MILLIMETRES											
	6	8	10	12	14	16	18	20	22	25	28	32
5	5·65	10·05	15·71	22·62	30·79	40·21	50·89	62·83	76·03	98·17	123·15	160·85
6	4·71	8·38	13·09	18·85	25·66	33·51	42·41	52·36	63·36	81·81	102·68	134·04
7	4·04	7·18	11·22	16·16	21·99	28·72	36·35	44·88	54·30	70·12	87·96	114·89
8	3·53	6·28	9·82	14·14	19·24	25·13	31·81	39·27	47·52	61·36	76·97	100·53
9	3·14	5·58	8·73	12·57	17·10	22·34	28·27	34·91	42·24	54·54	68·42	89·36
10	2·83	5·03	7·85	11·31	15·39	20·11	25·45	31·42	38·01	49·09	61·57	80·42
11	2·57	4·57	7·14	10·28	13·99	18·28	23·13	28·56	34·56	44·62	55·98	73·11
12	2·36	4·19	6·54	9·42	12·83	16·75	21·21	26·18	31·68	40·91	51·31	67·02
13	2·17	3·87	6·04	8·70	11·84	15·47	19·57	24·17	29·24	37·76	47·37	61·86
14	2·02	3·59	5·61	8·08	11·00	14·36	18·18	22·44	27·15	35·06	43·98	57·45
15	1·88	3·35	5·24	7·54	10·26	13·40	16·96	20·94	25·34	32·72	41·05	53·62
16	1·77	3·14	4·91	7·07	9·62	12·57	15·90	19·63	23·76	30·68	38·48	50·27
17	1·66	2·96	4·62	6·65	9·05	11·83	14·97	18·48	22·36	28·87	36·22	47·31
18	1·57	2·79	4·36	6·28	8·55	11·17	14·44	17·45	21·12	27·27	34·21	44·68
19	1·49	2·65	4·13	5·95	8·10	10·58	13·39	16·53	20·01	25·84	32·41	42·33
20	1·41	2·51	3·93	5·65	7·70	10·05	12·72	15·71	19·01	24·54	30·79	40·21
21	1·35	2·39	3·74	5·39	7·33	9·57	12·12	14·96	18·10	23·37	29·32	38·30
22	1·28	2·28	3·57	5·14	7·00	9·14	11·57	14·28	17·28	22·31	27·99	36·56
23	1·23	2·18	3·41	4·92	6·69	8·74	11·06	13·66	16·53	21·34	26·77	34·97
24	1·18	2·09	3·27	4·71	6·41	8·38	10·60	13·09	15·84	20·54	25·66	33·51
25	1·13	2·01	3·14	4·52	6·16	8·04	10·18	12·57	15·20	19·63	24·63	32·17
26	1·09	1·93	3·02	4·35	5·92	7·73	9·79	12·08	14·62	18·88	23·68	30·93
27	1·05	1·86	2·91	4·19	5·70	7·45	9·42	11·64	14·08	18·18	22·81	29·79
28	1·01	1·79	2·80	4·04	5·50	7·18	9·09	11·22	13·58	17·53	21·99	28·76
29	0·97	1·73	2·71	3·90	5·31	6·93	8·77	10·83	13·11	16·93	21·23	27·73
30	0·94	1·68	2·62	3·77	5·13	6·70	8·48	10·47	12·67	16·36	20·52	26·81
32	0·88	1·57	2·45	3·53	4·81	6·28	7·95	9·82	11·88	15·34	19·24	25·13
34	0·83	1·48	2·31	3·33	4·53	5·91	7·48	9·24	11·18	14·44	18·11	23·65
36	0·78	1·40	2·18	3·14	4·28	5·58	7·77	8·73	10·56	13·63	17·10	22·34
38	0·74	1·32	2·07	2·98	4·05	5·29	6·70	8·27	10·00	12·92	16·20	21·16
40	0·71	1·26	1·96	2·83	3·85	5·03	6·36	7·85	9·50	12·27	15·39	20·11

TABLE 20

AREAS OF GROUPS OF STANDARD BARS (in Square Centimetres)

Number of bars	Bar Diameter in Millimetres								
	12	14	16	18	20	22	25	28	32
1	1·13	1·54	2·01	2·54	3·14	3·80	4·91	6·16	8·04
2	2·26	3·07	4·02	5·08	6·28	7·60	9·81	12·31	16·08
3	3·39	4·61	6·03	7·63	9·42	11·40	14·72	18·47	24·12
4	4·52	6·15	8·04	10·17	12·56	15·20	19·63	24·63	32·17
5	5·65	7·69	10·05	12·72	15·70	19·00	24·54	30·78	40·27
6	6·78	9·23	12·06	15·26	18·85	22·80	29·45	36·94	48·25
7	7·91	10·77	14·07	17·81	21·99	26·60	34·36	43·10	56·29
8	9·04	12·31	16·08	20·35	25·13	30·41	39·27	49·26	64·34
9	10·17	13·85	18·09	22·90	28·27	34·21	44·17	55·41	72·38
10	11·31	15·39	20·10	25·44	31·41	38·01	49·08	61·57	80·42
11	12·44	16·93	22·11	27·99	34·55	41·81	53·99	67·73	88·46
12	13·57	18·47	24·12	30·53	37·69	45·61	58·90	73·89	96·51
13	14·70	20·01	26·13	33·08	40·84	49·41	63·81	80·04	104·55
14	15·83	21·55	28·14	35·62	43·98	53·21	68·72	86·20	112·59
15	16·96	23·09	30·15	38·17	47·12	57·02	73·63	92·36	120·63
16	18·09	24·63	32·17	40·71	50·26	60·82	78·54	98·52	128·68
17	19·22	26·17	34·18	43·26	53·40	64·62	83·44	104·67	136·72
18	20·35	27·70	36·19	45·80	56·54	68·42	88·35	110·83	142·76
19	21·48	29·24	38·20	48·34	59·60	72·22	93·26	116·99	152·80
20	22·62	30·78	40·21	50·89	62·83	76·02	98·17	123·15	160·85

TABLE 21

PERIMETER OF GROUPS OF STANDARD BARS

(In Centimetres)

Number of Bars	Bar Diameter in Millimetres												
	6	8	10	12	14	16	18	20	22	25	28	32	36
1	1.88	2.51	3.14	3.77	4.40	5.03	5.65	6.28	6.91	7.85	8.80	10.05	11.31
2	3.77	5.02	6.28	7.54	8.79	10.05	11.31	12.56	13.82	15.70	17.59	20.10	22.62
3	5.65	7.54	9.42	11.31	13.19	15.08	16.96	18.85	20.73	23.56	26.38	30.15	33.92
4	7.54	10.05	12.56	15.08	17.59	20.10	22.62	25.13	27.64	31.41	35.18	40.21	45.23
5	9.42	12.56	15.70	18.85	21.99	25.13	28.27	31.41	34.55	39.27	43.98	50.26	56.54
6	11.31	15.08	18.85	22.62	26.38	30.15	33.92	37.69	41.46	47.12	52.77	60.31	67.85
7	13.19	17.59	21.99	26.38	30.78	35.18	39.58	43.98	48.38	54.97	61.57	70.37	79.16
8	15.08	20.10	25.13	30.15	35.18	40.21	45.23	50.26	55.29	62.83	70.37	80.42	90.47
9	16.96	22.62	28.27	33.92	39.58	45.23	50.89	56.34	62.20	70.68	79.16	90.47	101.78
10	18.85	25.13	31.41	37.69	43.98	50.26	56.54	62.83	69.11	78.54	87.96	100.53	113.09
11	20.73	27.64	34.55	41.46	48.38	55.29	62·20	69.11	76.02	86.39	96.76	110.58	124.40
12	22.62	30.15	37.69	45.23	52.77	60.31	67.85	75.39	82.93	94.24	105.55	120.63	135.71
13	24.50	32.67	40.84	49.00	37.17	65.34	73.51	81.68	89.85	102.10	114.35	130.69	147.02
14	26.38	35.18	43.98	52.77	61.57	70.37	79.16	87.96	96.76	109.95	123.15	140.74	158.33
15	28.27	37.69	47.12	56.54	65.97	75.39	84.82	94.24	103.67	117.81	131.94	150.79	169.64
16	30.15	40,21	50.26	60.31	70.37	80.42	90.47	100.53	110.58	125.66	140.74	160.85	180.95
17	32.04	42.72	53.40	64.08	74.77	85.45	96.13	106.81	117.49	133.51	149.54	170.90	192.26
18	33.92	45.23	56.54	67.85	79.16	90.47	101.78	113.09	124.40	141.37	158.33	180.95	203.57
19	35.81	47.75	59.69	71.62	83.56	95.50	107.44	119.38	131.31	149.22	167.13	191.00	214.88
20	37.69	50.26	62.83	75.39	87.96	100.53	113.09	125.66	138.23	157.08	175.93	201.06	226.19

TABLE 22

CHARACTERISTICS OF REINFORCEMENT BARS

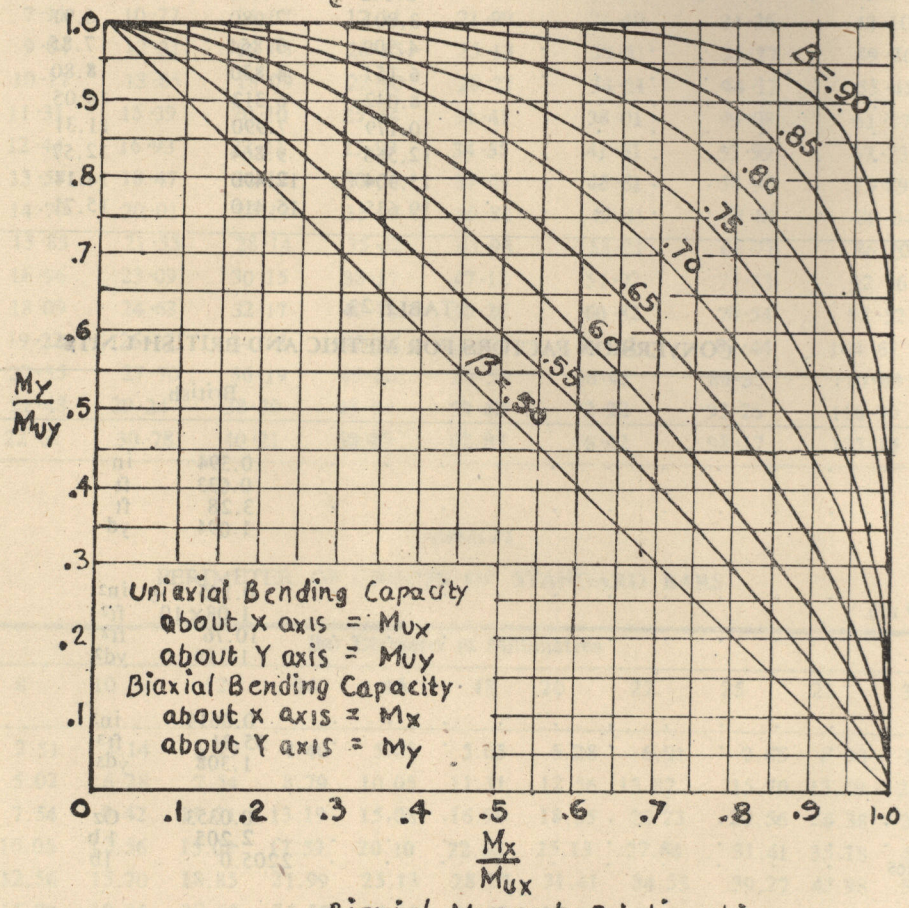
Size (mm)	Area (cm^2)	Wt (Kg/Metre)	Perimeter (cm)	Length (per tonne Metre)
6	0.283	0.222	1.89	4510
8	0.503	0.395	2.51	2532
10	0.785	0.617	3.14	1621
12	1.131	0.888	3.77	1125
14	1.539	1.208	4.40	829
16	2.011	1.578	5.03	633
18	2.545	2.000	5.65	500
20	3.142	2.466	6.28	405
22	3.801	2.980	6.91	336
25	4.909	3.854	7.85	260
28	6.157	4.830	8.80	207
32	8.042	6.313	10.05	159
36	10.179	7.990	11.31	125
40	12.566	9.864	12.57	101
45	15.904	12.490	14.14	80
50	19.635	15.410	15.71	65

TABLE 23

CONVERSION FACTORS FOR METRIC AND BRITISH UNITS

Metric	British		Reciprocal
Length			
1 cm	0.394	in	2.54
	0.033	ft	30.50
1 m	3.28	ft	0.305
	1.094	yd	0.914
Area			
1 cm^2	0.155	in^2	6.452
	1.08×10^{-5}	ft^2	929.0
1 m^2	10.76	ft^2	0.093
	1.196	yd^2	0.838
Volume			
1 cm^3	0.0619	in^3	16.387
1 m^3	35.31	ft^3	0.0283
	1.308	yd^3	0.765
Mass Force			
1 g.	0.0353	Oz	28.35
1 kg.	2.205	1 b	0.454
1 Tonnes	2205.0	1b	0.00045
Density			
1 kg/cm^3	36.1	$1b/in^3$	0.0277
1 kg/m^3	0.0624	$1 b/ft^3$	16.02
Moment of Inertia			
1 cm^4	0.024	in^4	41.623
1 m^4	115.9	ft^4	0.00863
Pressure, Stress			
1 kg/cm	5.6	1b/in	0.178
1 kg/m	0.672	lb/ft	1.488
1 kg/mm^2	1422.0	lb/in^2	0.007
	1.422	kip/in^2	0.703
1fig(cm^2	14.22	lb/in^2	0.0703
1 kg/m^2	0.205	lb/ft^2	4.88
1 $tonne/m^2$	205.0	lb/ft^2	0.0049
Moment, Torque			
1 kg-cm	0.868	in-lb	1.152
	0.0723	ft-lb	13.825
1kg-m	7.231	ft-lb	0.138
1 tonne-cm	868.0	in-lb	0.00115
	72.3	ft-lb	0.0138

Chart 1 (Para 7.5.2)



Biaxial Moment Relationship

Chart 2 (Para 7.5.1)

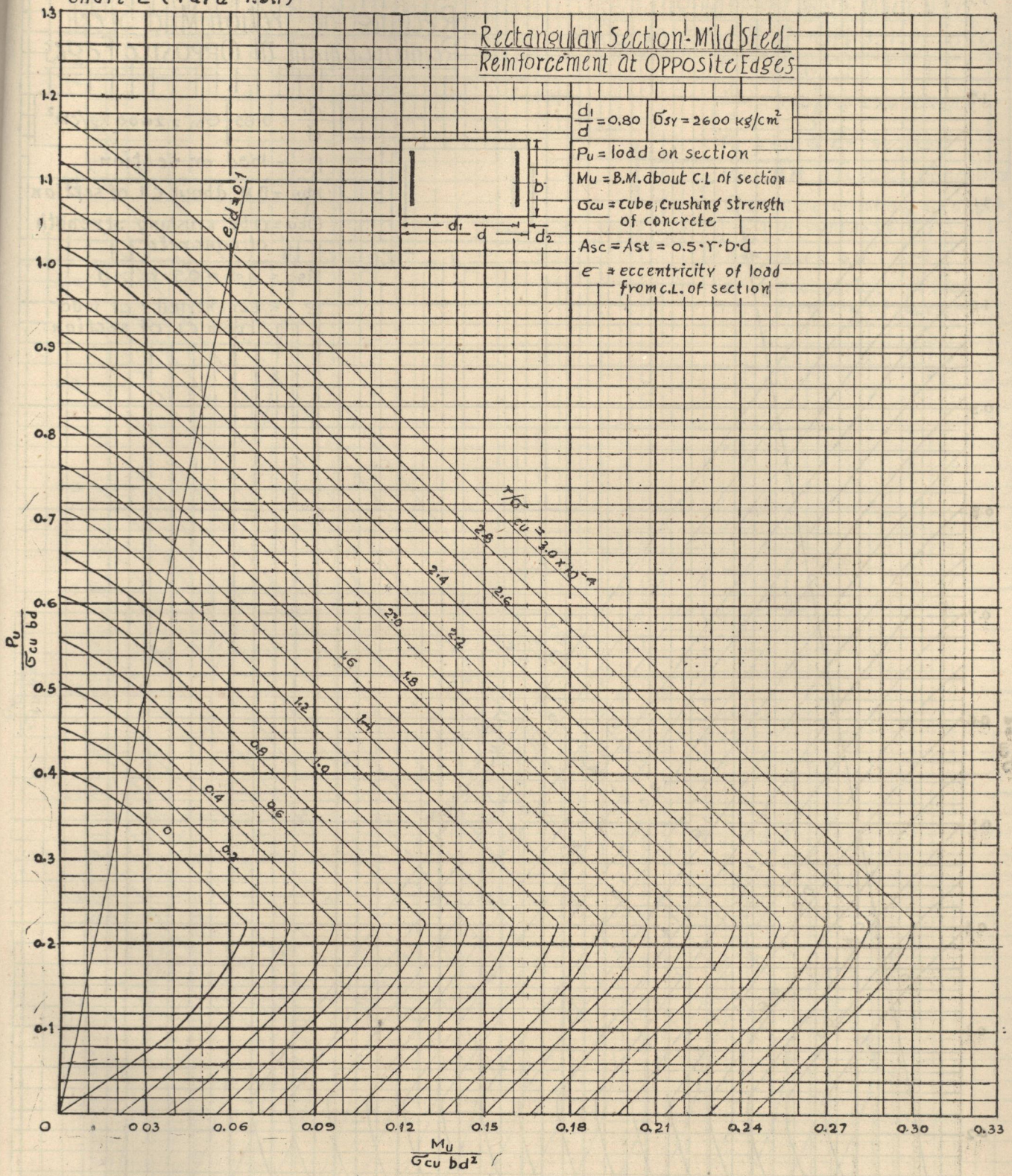


Chart 3 (Para 7.5.1)

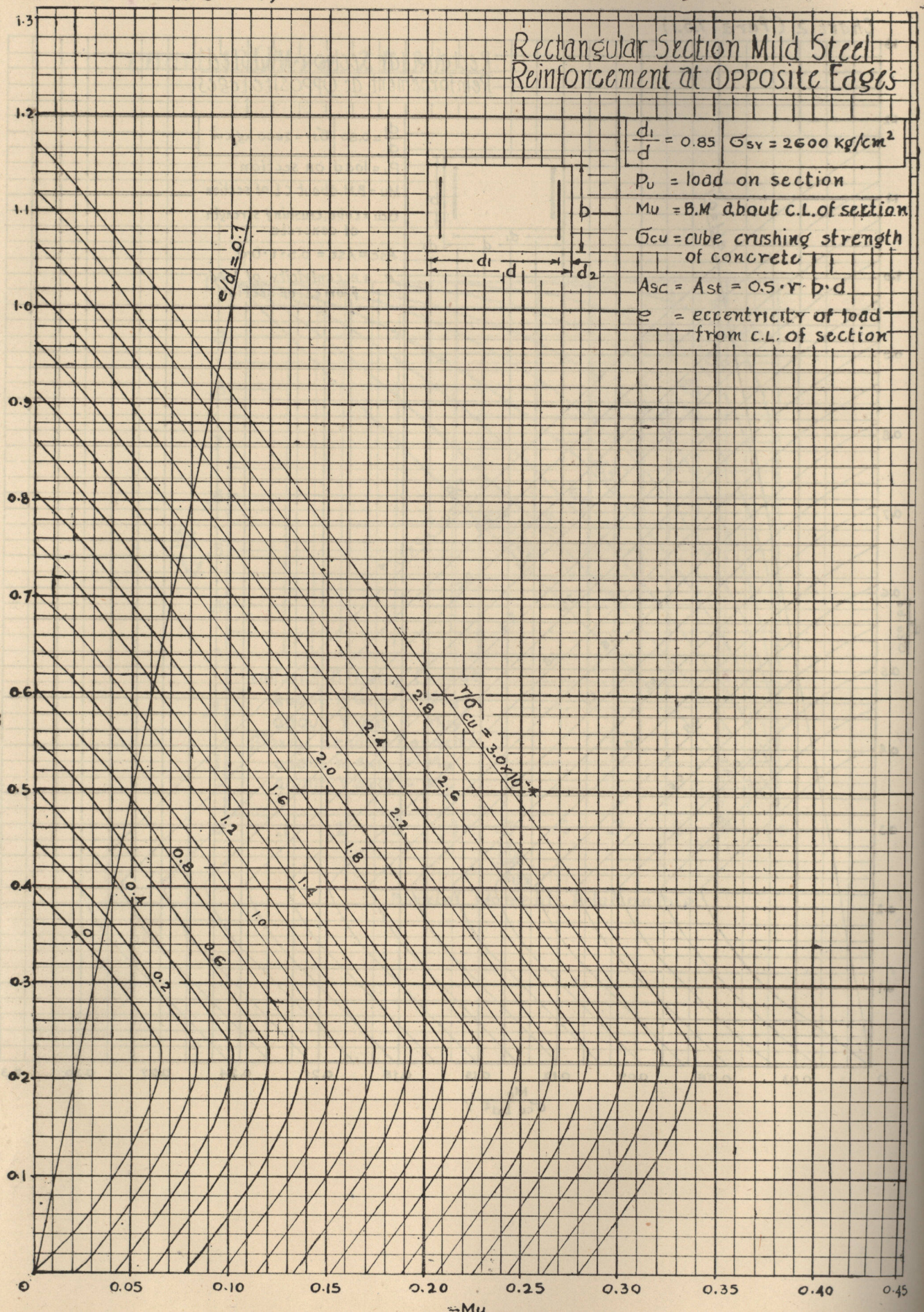


Chart 4 (Para 7.5.1)

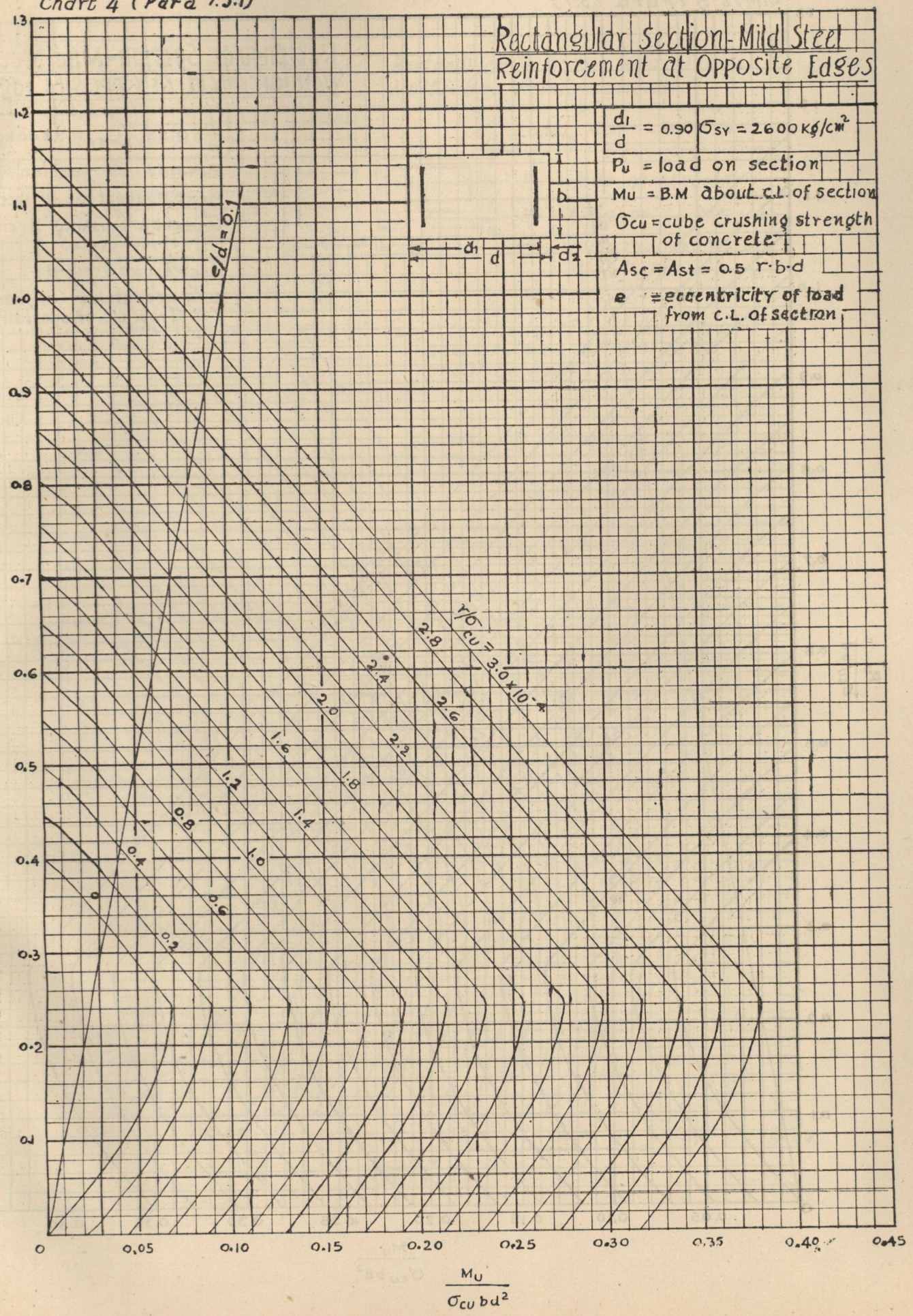


Chart 5 (Para 7.5.1)

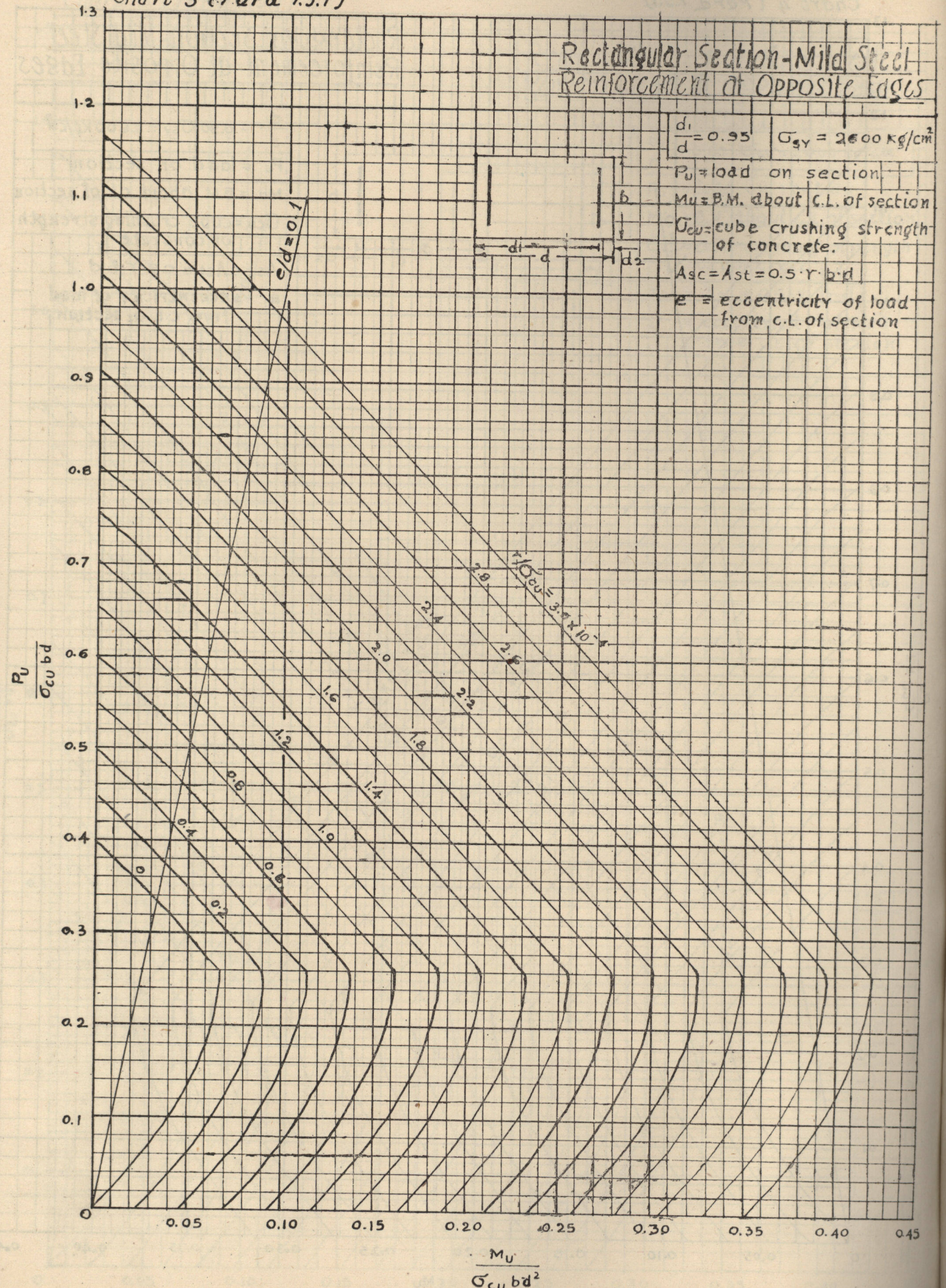


chart 6 (Para 7.5.1)

Rectangular Section- Mild Steel Multi-Bar Reinforcement

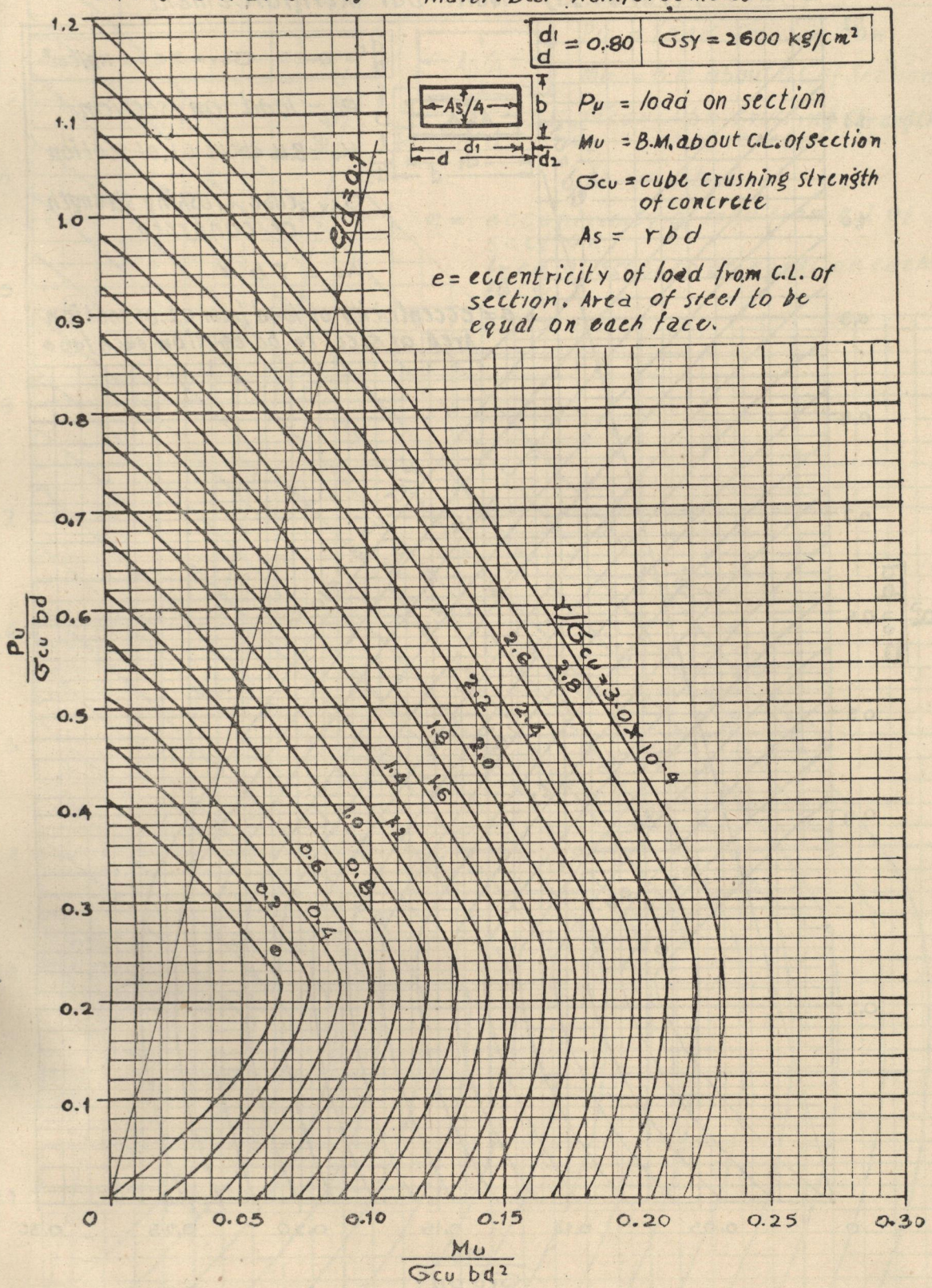


Chart 7 (Para 7.5.1) Rectangular Section - Mild Steel Multi-Bar Reinforcement

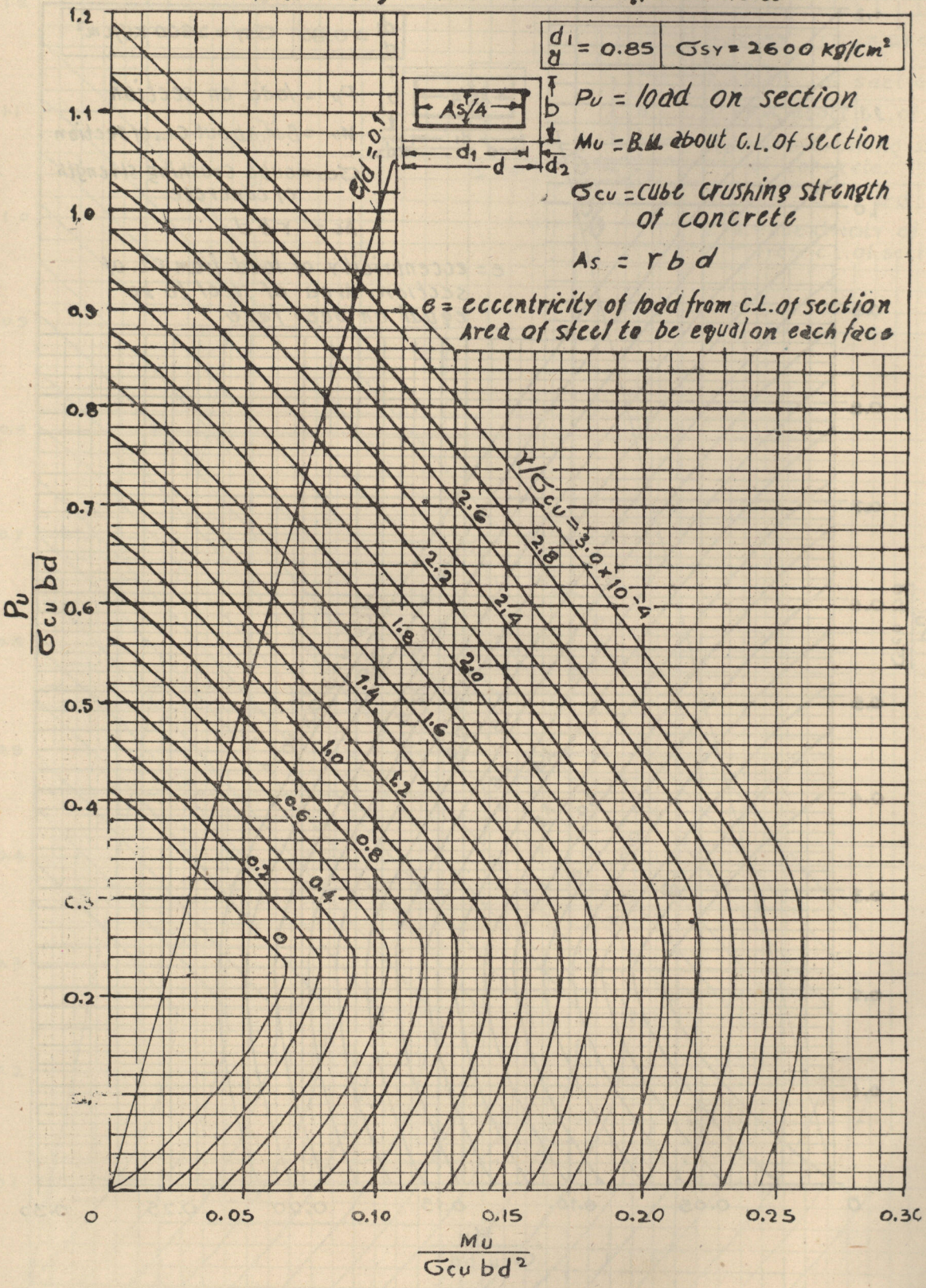
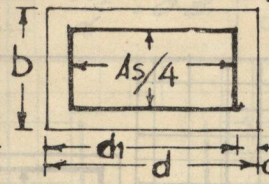


Chart 8 (Para 7.5.1)

Rectangular Section - Mild Steel Multi-Bar Reinforcement

$\frac{d_1}{d} = 0.90$	$\sigma_{sy} = 2600\ kg/cm^2$



Pu = load On Section

Mu = B.M. about C.L. of Section

σ_{cu} = Cube crushing Strength of concrete

$A_s = rbd$

e = eccentricity of load from c.l of section

Area of steel to be equal on each face

Rectangular Section-Mild-Steel Multi-Bar Reinforcement

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Chart 9 (Para 7.5.1)

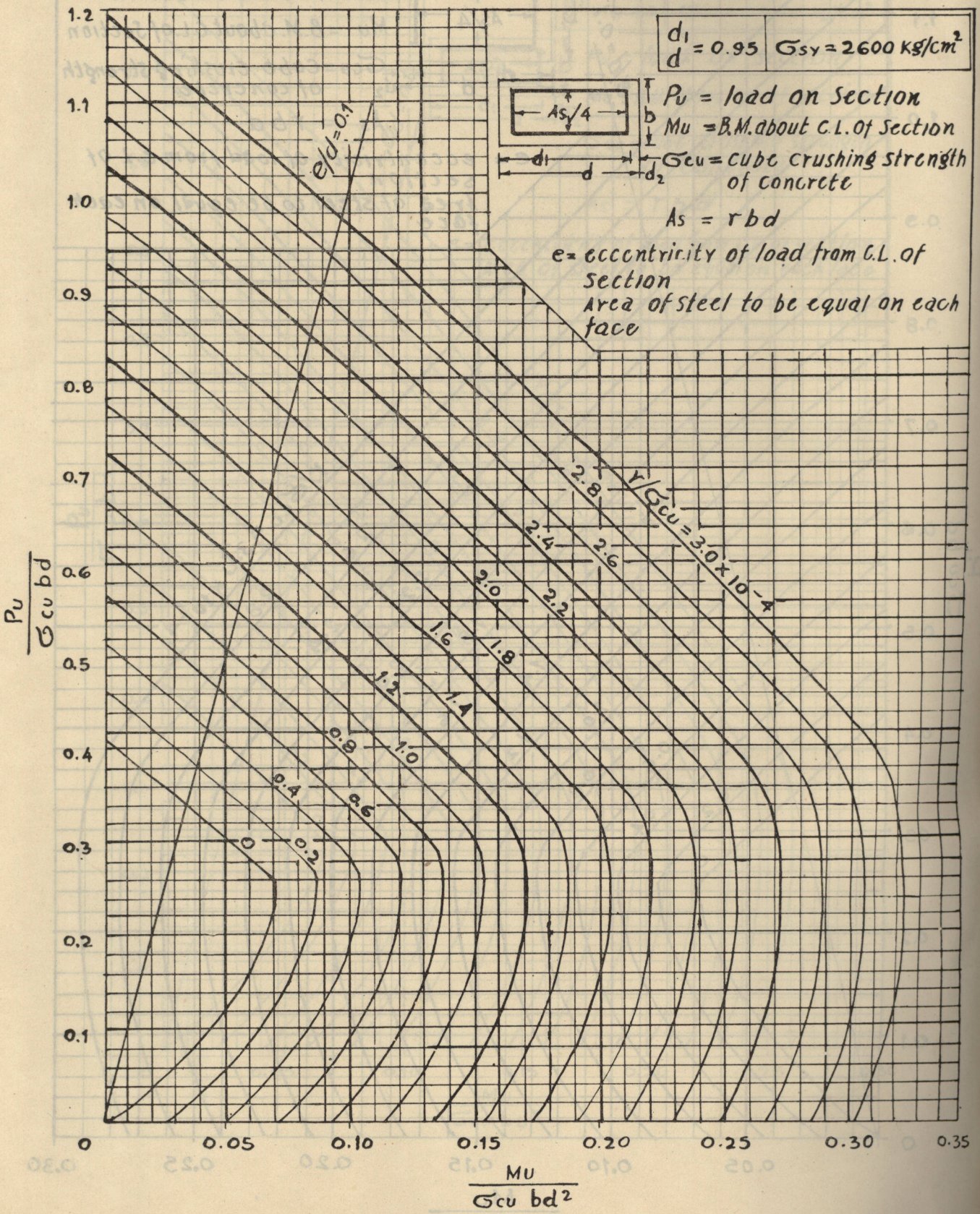
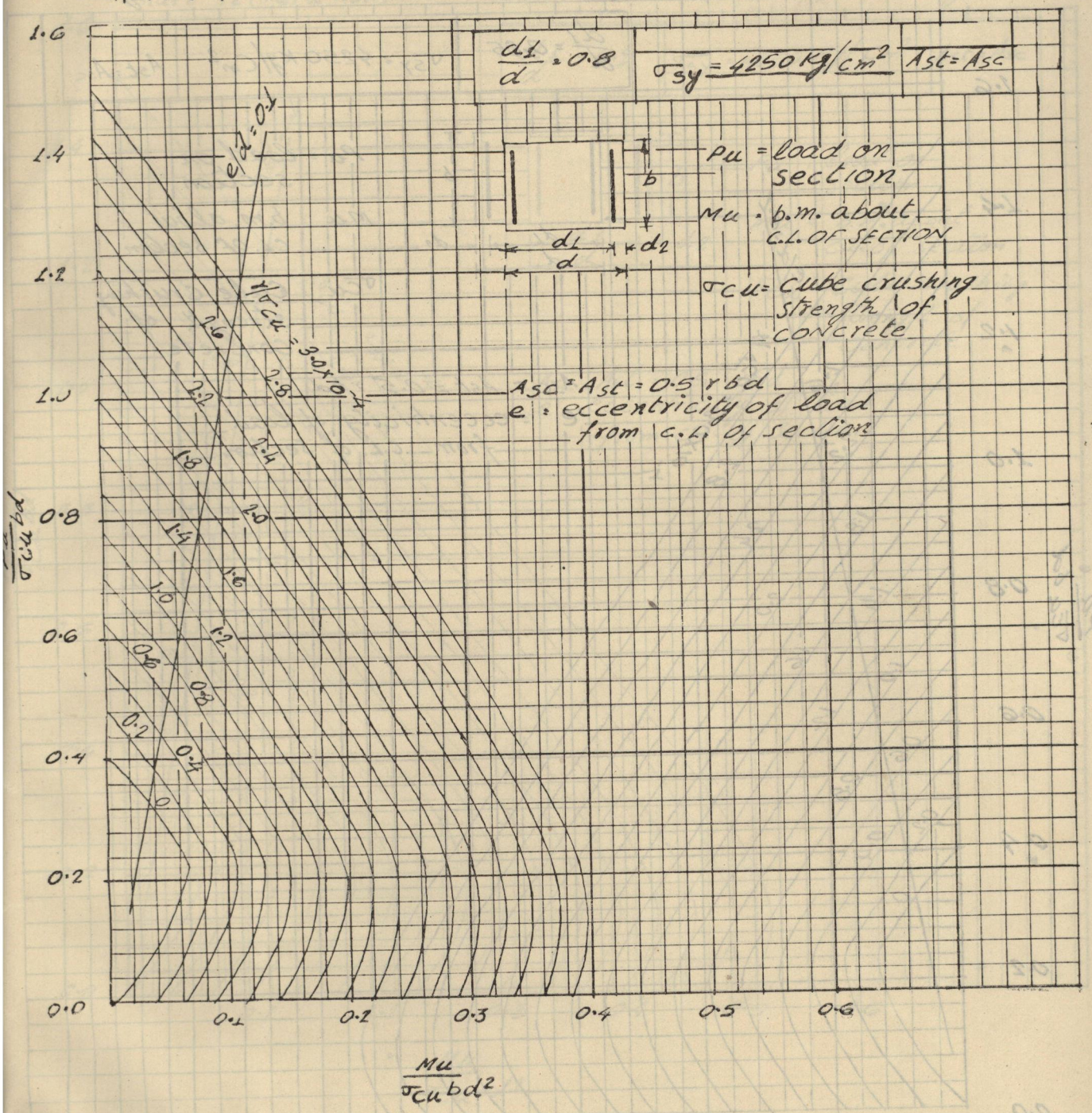


Chart 10 (Para 7.5.1)

Rectangular Section - High Tensile Steel Reinforcement at Opposite Edges.



Rectangular Section - High Tensile Steel Reinforcement at opposite Edges

Chart 11 (Para 7.5.1)

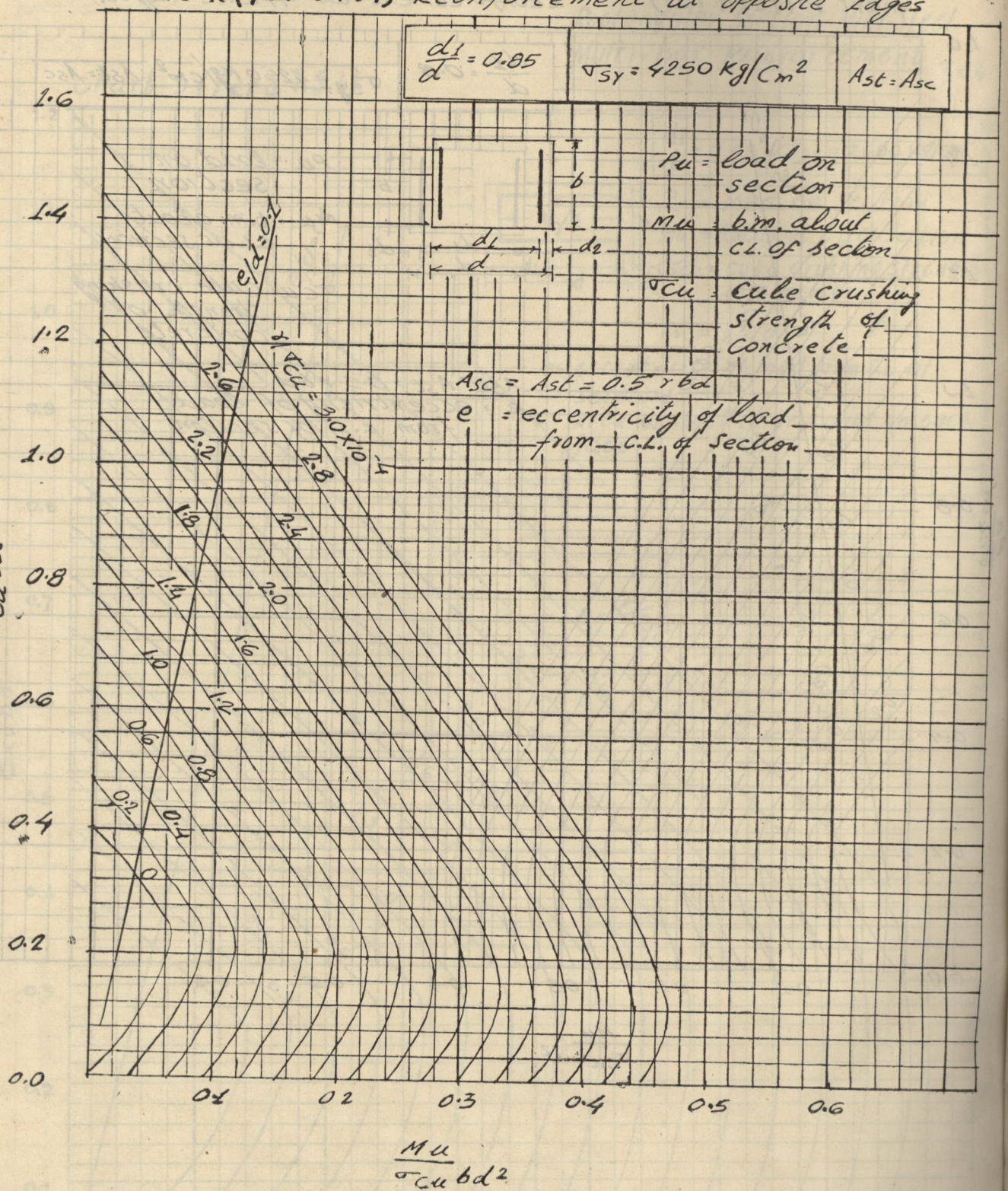


Chart 12 (Para 7.5.1)

Rectangular Section - High Tensile Steel Reinforcement at opposite Edges

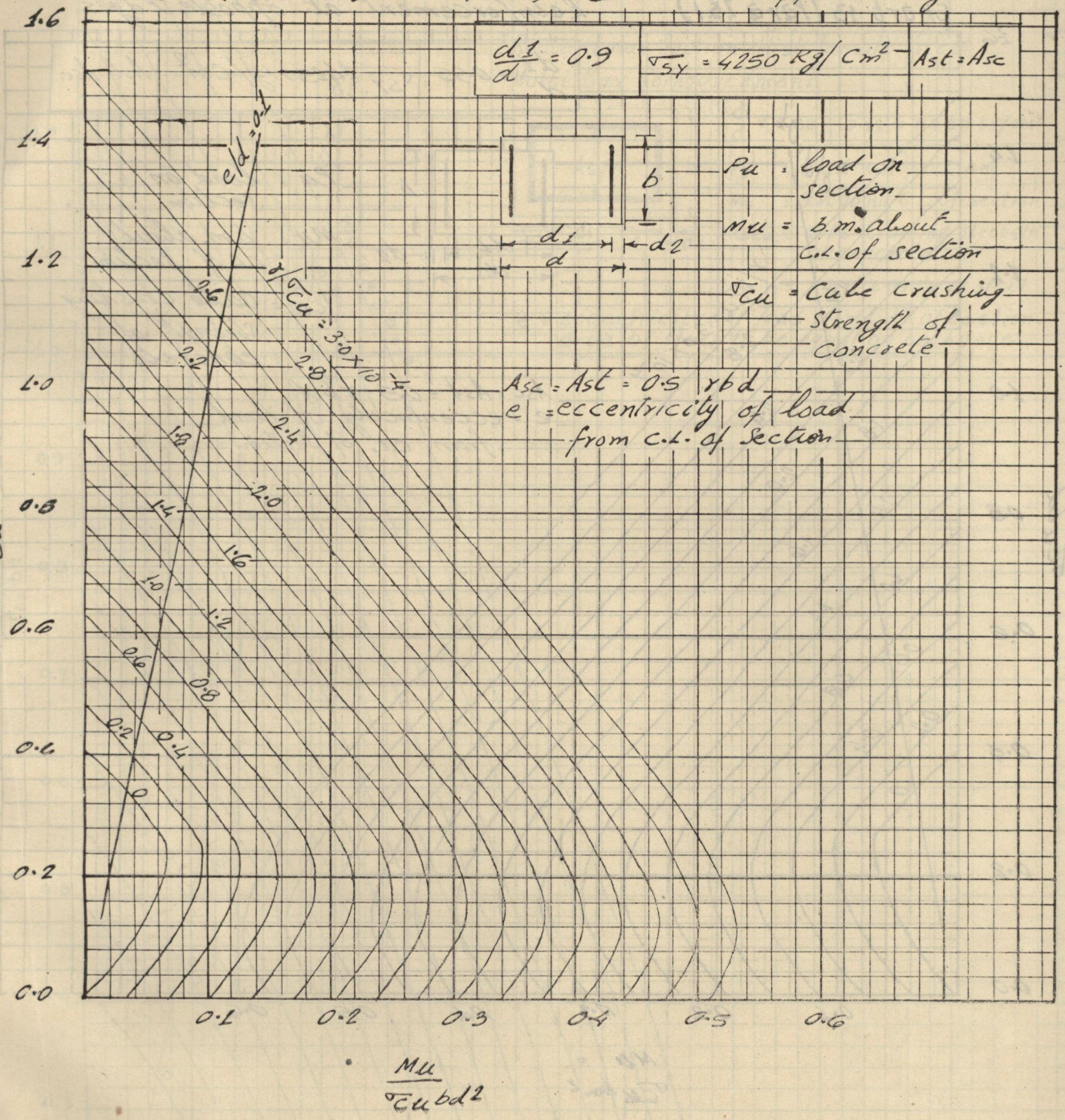


Chart 13 (Para 7.5.1)

Rectangular Section - High Tensile Steel Reinforcement at opposite Edges

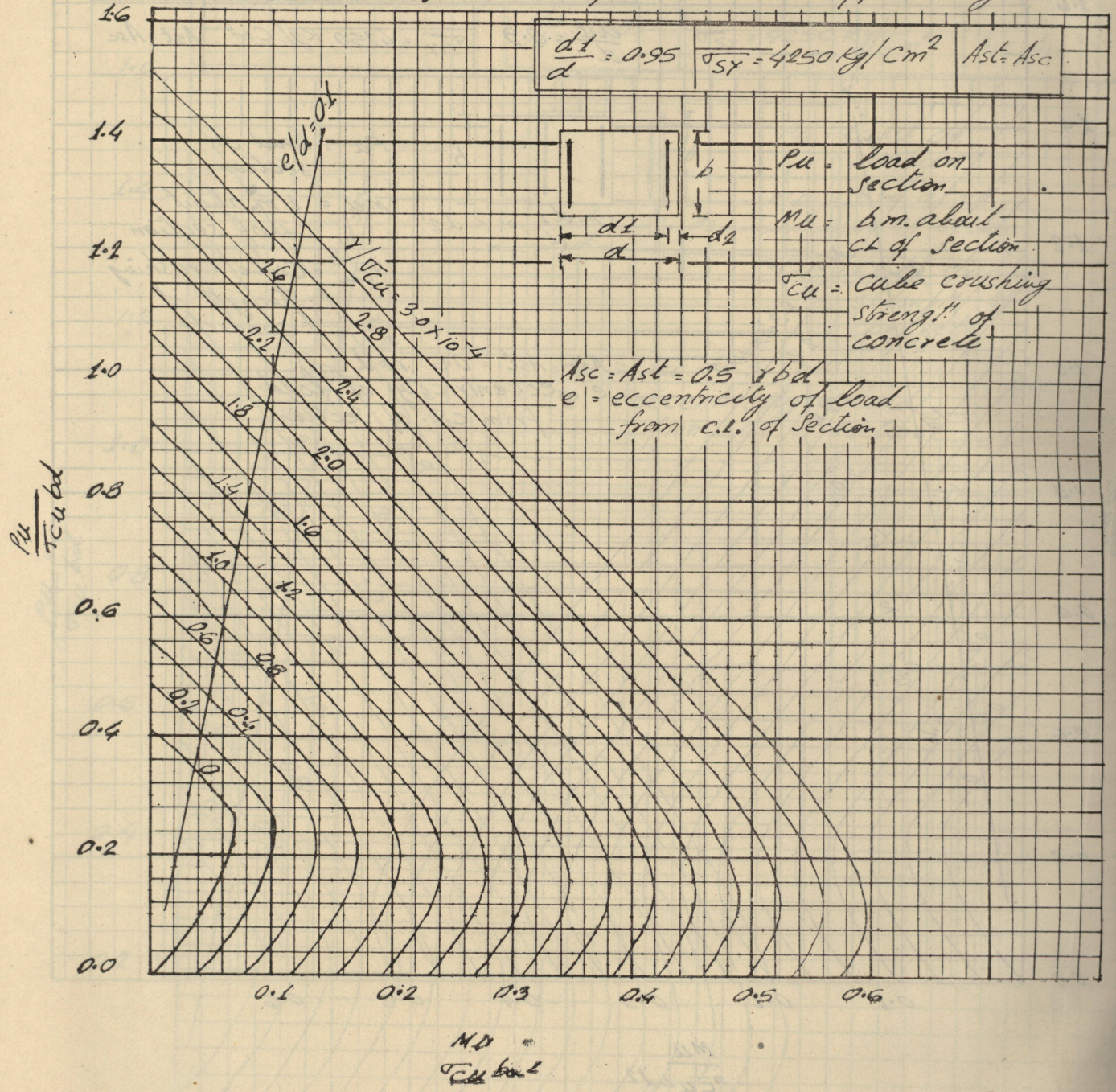


Chart 14 (Para 7.5.1)

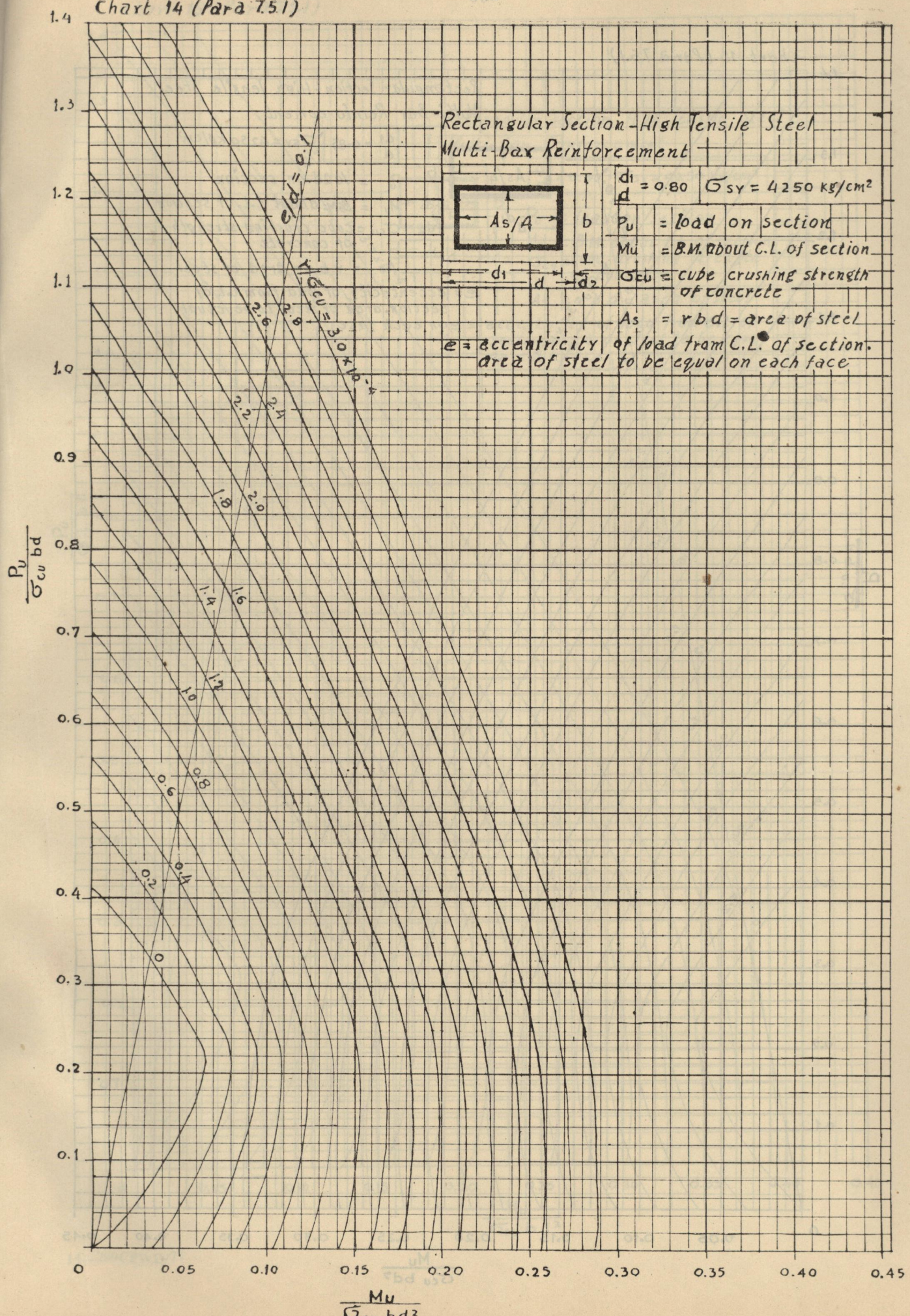


Chart 15 (Para 7.5.1)

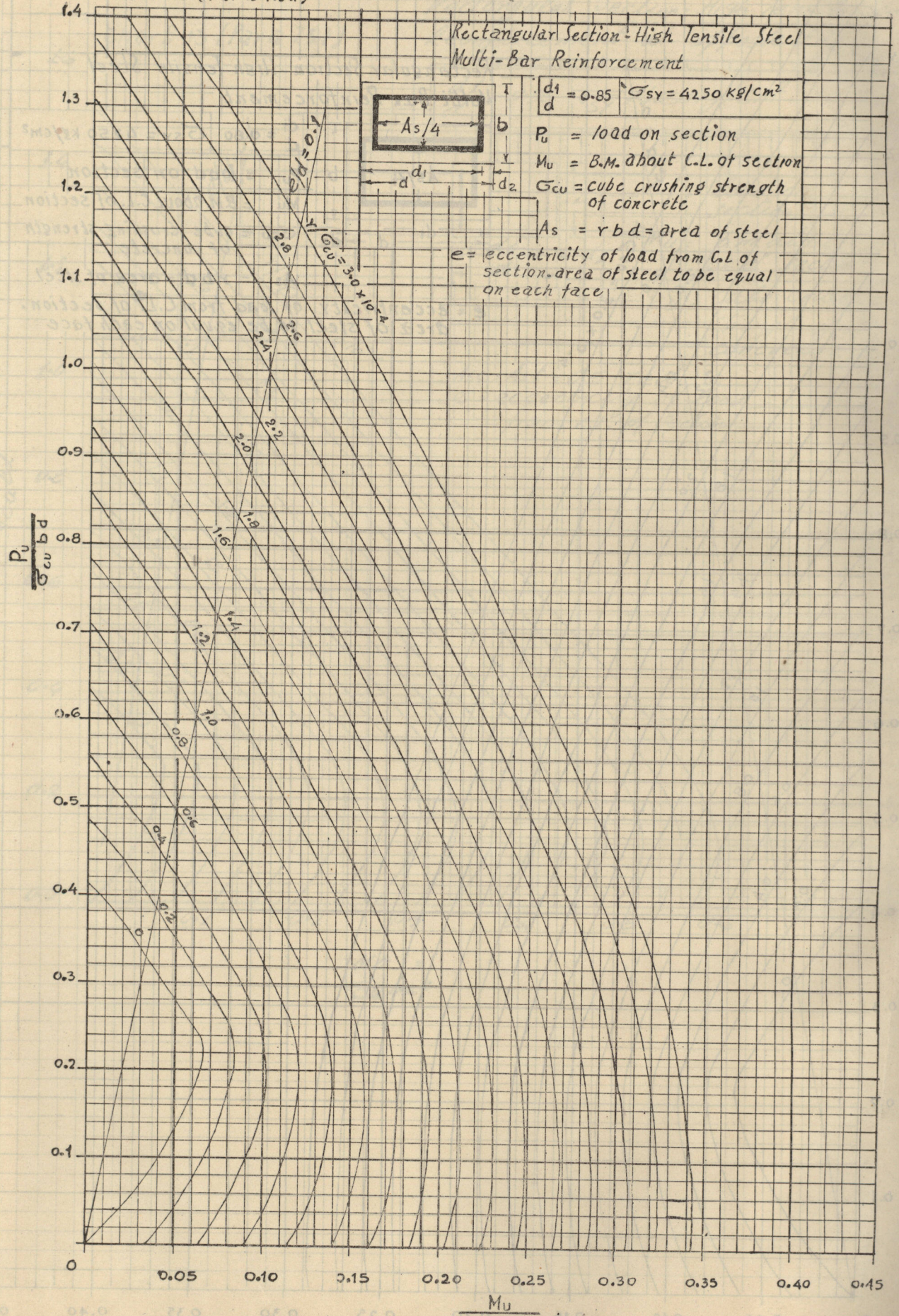


Chart 16 (Para 7.5.1)

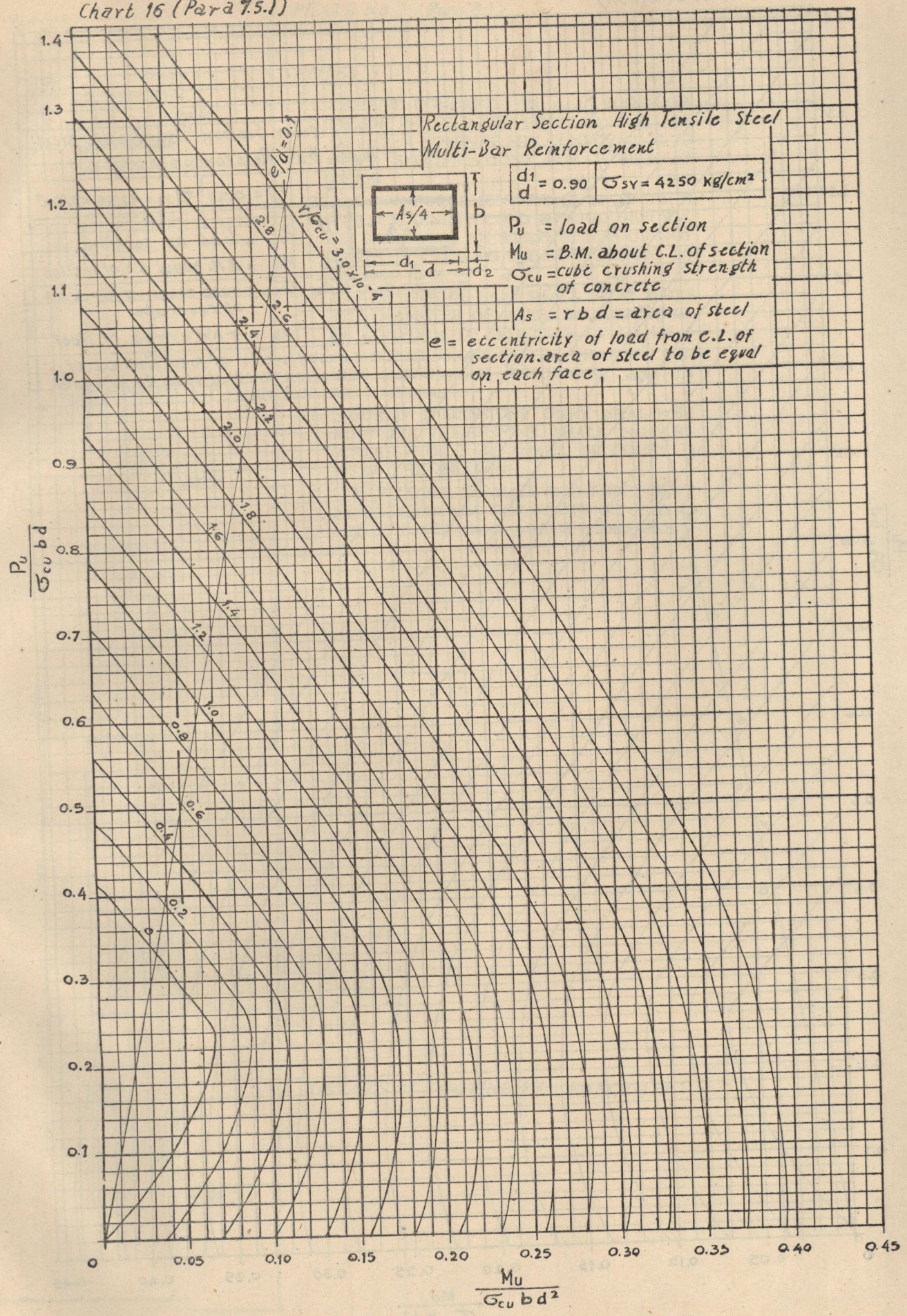
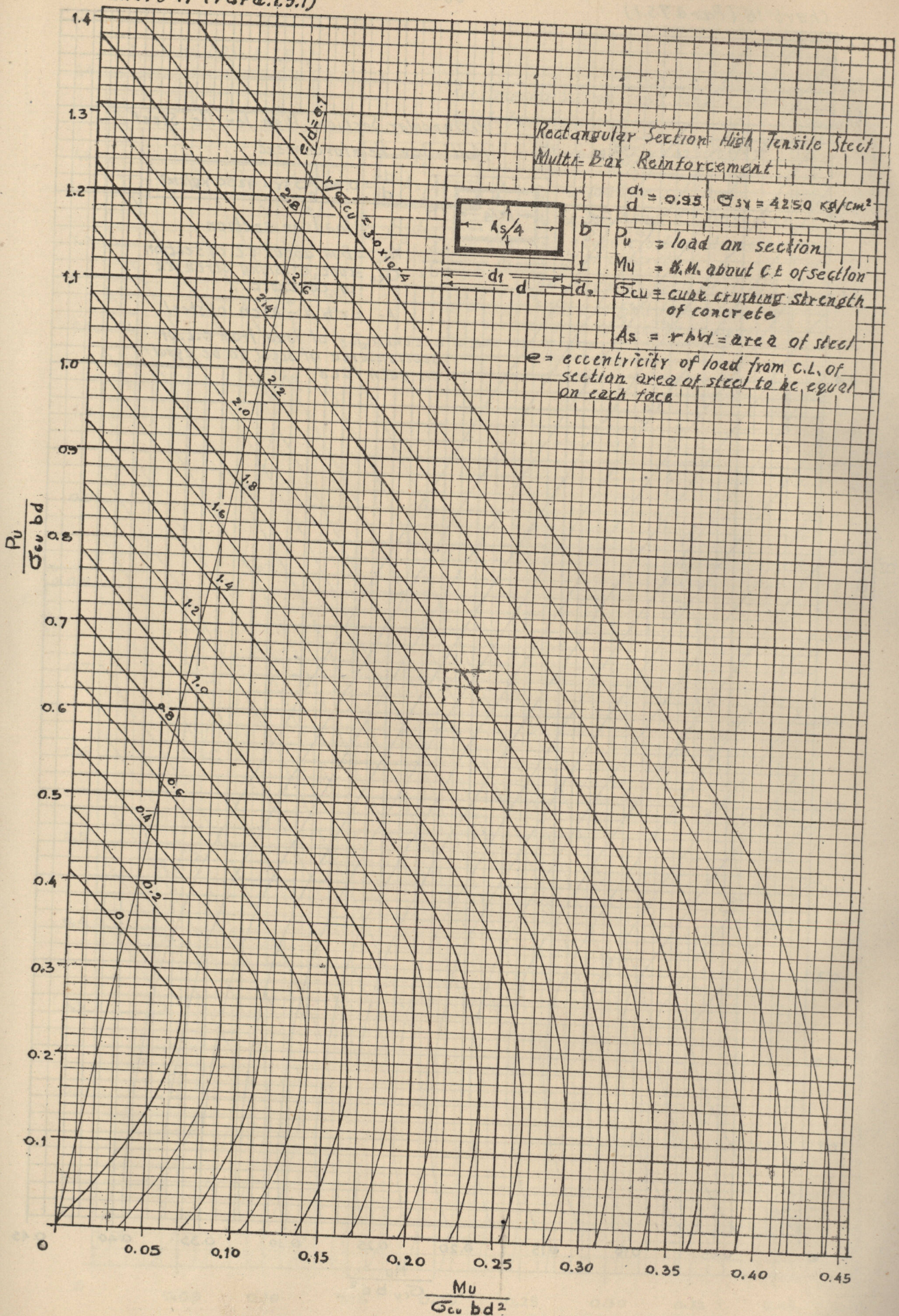


Chart 17 (Para. 7.5.1)



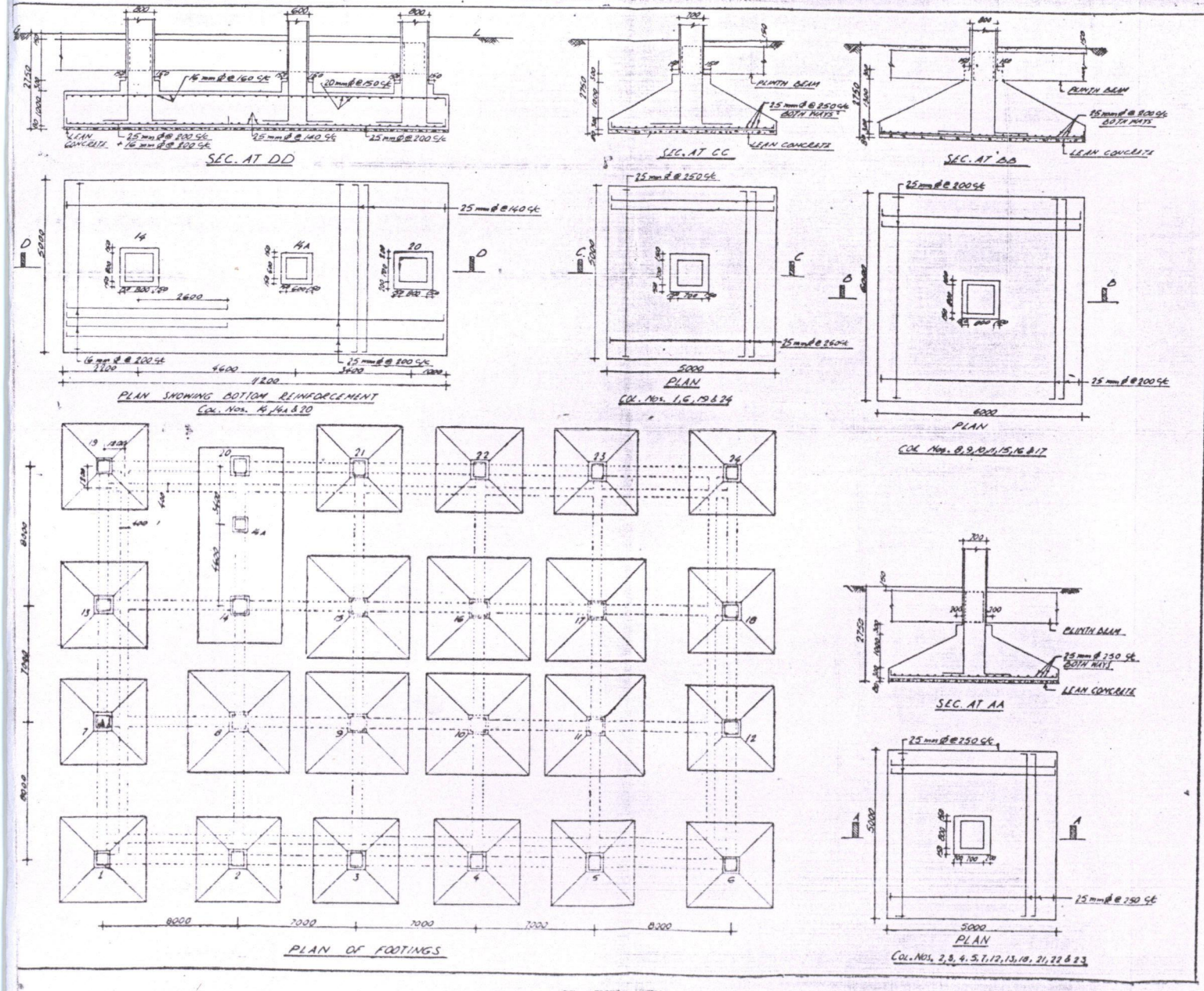


PLATE No 1 (Para 8.1)

NOTES

1. CEMENT CONCRETE IN ALL FOOTINGS, GRADE BEAMS AND PEDESTAL SHALL BE OF 1 : 2 : 4 NOMINAL MIX CONFORMING TO CPWD SPECIFICATIONS FOR WORKS AT DELHI.
2. ALL REINFORCEMENT SHALL BE OF HIGH YIELD STRENGTH COLD TWISTED DEFORMED BARS WITH AN YIELD STRESS OF NOT LESS THAN 4250 KG/CM² CONFORMING TO I.S. 1786—1966.
3. LAP LENGTH OF BARS SHALL BE 69 TIMES THE DIAMETER OF BARS. SPLICES SHALL BE CONFORM TO PROVISION OF I.S. 456.
4. ALL DIAMENSIONS IN THE DRAWING UNLESS, MENTIONED OTHERWISE ARE IN MILLIMETERS.
5. FIGURED DIMENSIONS ONLY SHALL BE ADOPTED. DIMENSIONS SHALL NOT BE SCALED NOR BARS SHALL BE COUNTED FROM THE DRAWING.
6. CLEAR COVER TO MAIN REINFORCEMENT SHALL BE 80 MM. FOR FOOTINGS, AND 40 MM. FOR COLUMNS ABOVE PEDESTALS.

DETAILS OF FOOTING REINFORCEMENT

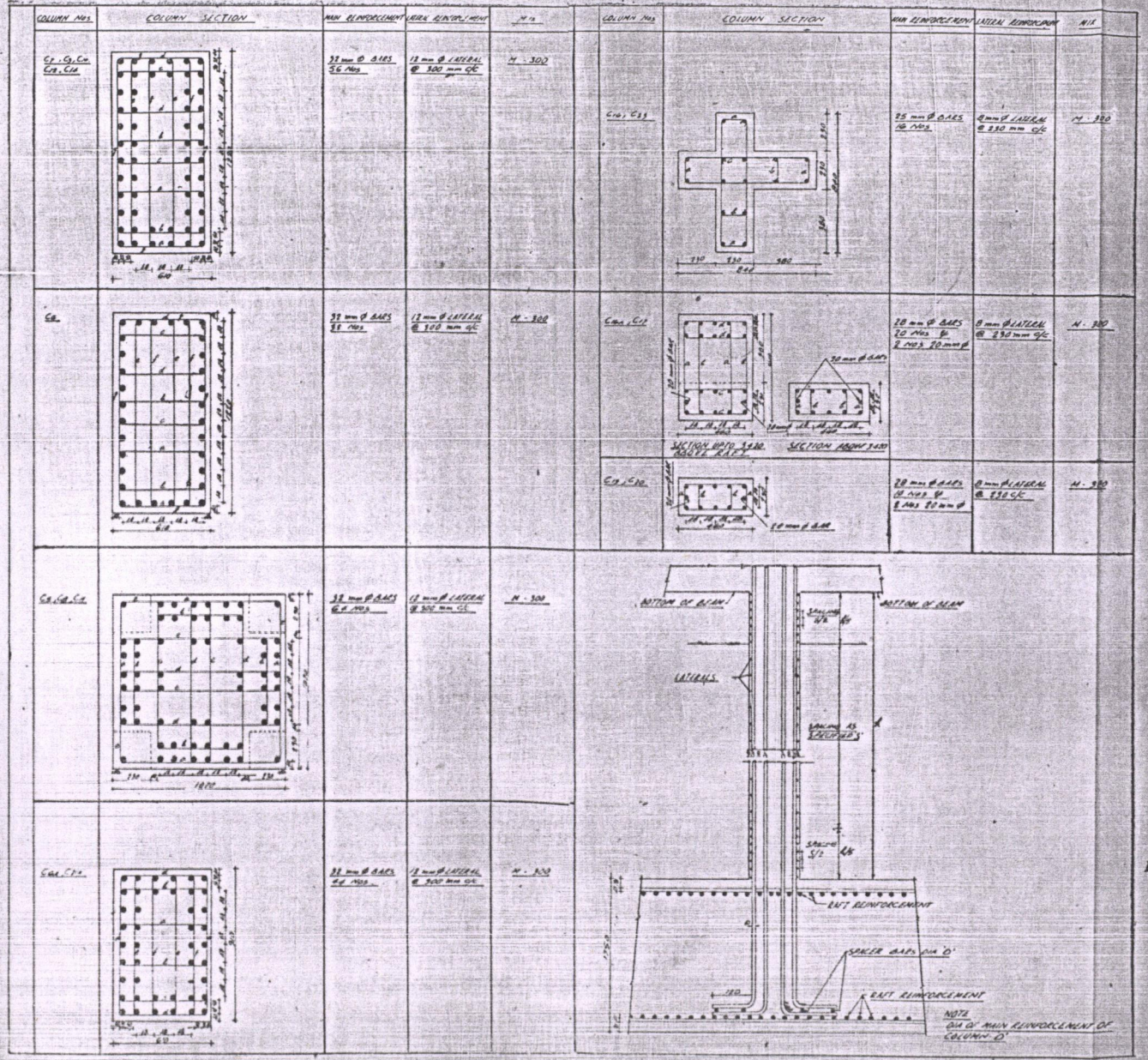


PLATE No 2 (Para 8.1)

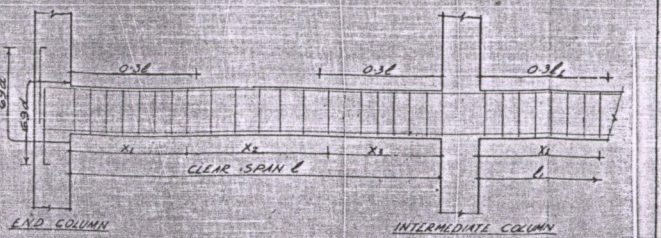
NOTES

1. CEMENT CONCRETE FOR COLUMNS SHALL BE OF 1 : 1 : 2 NOMINAL MIX CONFORMING TO CPWD SPECIFICATIONS FOR WORKS AT DELHI.
2. ALL REINFORCEMENT SHALL BE OF HIGH YIELD STRENGTH COLD TWISTED DEFORMED BARS WITH AN YIELD STRESS OF NOT LESS THAN 4250 KG/CM² CONFORMING TO I.S. 1786—1966.
3. THE SPACING OF BINDERS SHALL BE REDUCED TO HALF THE SPECIFIED SPACING FOR A LENGTH OF 1/5TH HEIGHT OF COLUMN ON BOTH ENDS AT THE JUNCTION OF COLUMNS WITH BEAM OR RAFT.
4. NOT MORE THAN ONE THIRD OF THE TOTAL NUMBER OF MAIN BARS SHALL BE OVERLAPPED AT ANY SECTION OF A COLUMN LAP LENGTH EQUAL TO 41 TIMES DIAMETER OF THE BARS SHALL BE PROVIDED.
5. CLEAR COVER TO MAIN REINFORCEMENT SHALL BE 40 MILLIMETRES.
6. EACH OF THE LATERAL REINFORCEMENT a, b, c, d, e & f SHALL BE PROVIDED AT THE SPACING INDICATED.
7. ALL DIAMENSIONS IN THE DRAWING UNLESS MENTIONED OTHERWISE, ARE IN MILLIMETRES.

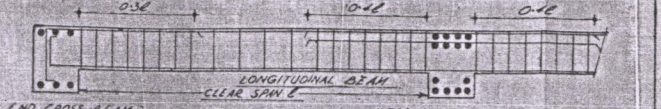
DETAILS OF COLUMN REINFORCEMENT

DETAILS FOR REINFORCEMENT OF BEAMS

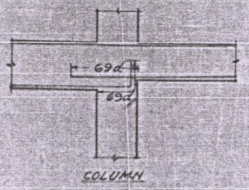
SERIAL NO	BEAMS		CLEAR SPAN (METRES)	WIDTH (CENTIMETRES)	OVERALL DEPTH (CENTIMETRES)	REINFORCEMENT AT 1st SUPPORT TOP No - mm ϕ	AT 1st SUPPORT BOTTOM No - mm ϕ	AT CENTRE TOP No - mm ϕ	AT CENTRE BOTTOM No - mm ϕ	AT 2nd SUPPORT TOP No - mm ϕ	AT 2nd SUPPORT BOTTOM No - mm ϕ	STIRRUPS AT 1st SUPPORT DIA mm ϕ	LEGGED	C/C in Cms	DIST from M Xl	AT CENTRE DIA mm ϕ	LEGGED	C/C in Cms	DIST from M Xl	AT 2nd SUPPORT DIA mm ϕ	LEGGED	C/C in Cms	DIST from M Xl
1	C_1 - C_2		5·07	23	80	2 - 28	2 - 28	2 - 28	2 - 28	2 - 28	2 - 28	10	2	8	2·0	10	2	15	1·07	10	2	8	2·0
						1 - 28	2 - 28	2 - 28	2 - 28	2 - 28	2 - 28											
2	C_2 - C_3		5·07	23	80	2 - 28	2 - 28	2 - 28	2 - 28	2 - 28	2 - 28	10	2	10	2·0	10	2	20	1·01	10	2	10	2·0
									2 - 28	2 - 28	2 - 28											
3	J_{10} - J_{11}		10·355	45	107	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	12	4	10	3·5	12	4	20	3·355	12	4	10	3·5
						5 - 28	3 - 28		3 - 28	5 - 28	3 - 28											
4	J_8 - J_9		10·355	45	107	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	12	4	10	3·5	12	4	20	3·355	12	4	10	3·5
						5 - 28	3 - 28		3 - 28	5 - 28	3 - 28											
5	J_{13} - J_{14}		10·355	45	107	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	10	4	10	2·5	10	4	20	4·355	10	4	10	3·5
							5 - 28		5 - 28	5 - 28	5 - 28											
			10·355	45	107	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	10	4	10	3·5	10	4	20	4·355	10	4	10	2·5
						2 - 28	5 - 28		5 - 28		5 - 28											
			10·125	45	107	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	10	4	10	3·0	10	4	10	4·125	10	4	10	3·0
						5 - 28	3 - 28		3 - 28	5 - 28	3 - 28											
	C_6		10·125	45	107	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	10	4	10	3·0	10	4	20	4·125	10	4	10	3·0
						5 - 28	3 - 28		3 - 28	5 - 28	3 - 28											
8	J_{16} - J_{17}		10·355	45	107	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	10	4	10	2·5	10	4	20	4·355	10	4	10	3·5
							5 - 28		5 - 28	5 - 28	5 - 28											
9	J_{17} - C_{14}		10·355	45	107	5 - 28	5 - 28	5 - 28	5 - 28	4 - 28	5 - 28	10	4	10	3·5	10	4	20	4·355	10	4	10	2·5
						5 - 20				2 - 20												
11	J_9 - J_{19}		10·355	45	107	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	10	4	10	2·5	10	4	20	4·355	10	4	10	3·5
							5 - 28		5 - 28	5 - 28	5 - 28											
12	J_{19} - J_{20}		10·355	45	107	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	10	4	10	3·5	10	4	20	4·355	10	4	10	2·5
						2 - 28				2 - 28												
13	C_7 - C_8		10·125	45	107	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	10	4	10	3·0	10	4	20	4·125	10	4	10	3·0
						5 - 28	3 - 28		3 - 28	5 - 28	3 - 28											
14	C_8 - C_9		10·125	45	107	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	10	4	10	3·0	10	4	20	4·125	10	4	10	3·0
						5 - 28	3 - 28		3 - 28	5 - 28	3 - 28											
15	J_{15} - J_{24}		10·355	45	107	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	10	4	10	2·5	10	4	20	4·355	10	4	10	3·5
							5 - 28		5 - 28	5 - 28	5 - 28											
16	J_{24} - J_{25}		10·355	45	107	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	10	4	10	3·5	10	4	20	4·355	10	4	10	2·5
						5 - 28	5 - 28		5 - 28		5 - 28											
17	J_{26} - J_{27}		10·355	45	107	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	10	4	10	2·5	10	4	20	4·355	10	4	10	2·5
							3 - 28		3 - 28	5 - 28	3 - 28											
18	J_{27} - J_{28}		10·355	45	107	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	10	4	10	2·5	10	4	20	4·355	10	4	10	2·5
						5 - 28	5 - 28		3 - 28		3 - 28											
19	C_{10} - C_{11}		10·125	45	90	5 - 28	5 - 28	3 - 28	5 - 28	5 - 28	5 - 28	10	4	15	3·0	10	4	30	4·125	10	4	15	3·0
						3 - 28				3 - 28												
20	C_{11} - C_{12}		10·125	45	90	5 - 28	5 - 28	3 - 28	5 - 28	5 - 28	5 - 28	10	4	15	3·0	10	4	30	4·125	10	4	15	3·0
						3 - 28				3 - 28												
21	J_{29} - J_{30}		10·355	45	90	3 - 28	5 - 28	3 - 28	5 - 28	5 - 28	5 - 28	10	4	15	3·0	10	4	30	4·355	10	4	15	3·0
										3 - 28												
22	J_{30} - J_{31}		10·355	45	90	5 - 28	5 - 28	3 - 28	5 - 28	3 - 28	5 - 28	10	4	15	3·0	10	4	30	4·355	10	4	15	3·0
						3 - 28																
23	J_{32} - J_{33}		10·355	45	90	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	10	4	15	2·0	10	4	30	6·355	10	4	15	2·0
						3 - 28				3 - 28												
24	J_{33} - J_{34}		10·355	45	90	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	5 - 28	10	4	15	2·0	10	4	30	6·355	10	4	15	2·0
						3 - 28				3 - 28												
25	C_{13} - C_{14}		6·10	VARY	107	3 - 28	3 - 28	3 - 28	3 - 28	3 - 28	3 - 28	8	4	15	1·5	8	4	30	3·1	8	4	15	1·5
						3 - 28	2 - 28		2 - 28	3 - 28	2 - 28											
26	C_{14} - C_{15}		6·10	VARY	107	3 - 28	3 - 28	3 - 28	3 - 28	3 - 28	3 - 28	8	4	15	1·5	8	4	30	3·1	8	4	15	1·5
						3 - 28	2 - 28		2 - 28	3 - 28	2 - 28											
27	C_1 - C_4		3·66	VARY	107	3 - 32	3 - 32	3 - 32	3 - 32	3 - 32	3 - 32	10	4	10	-	10	4	10	-	10	4	10	-
						3 - 32				3 - 32												
28	C_4 - C_7		7·47	VARY	107	3 - 32	3 - 32	3 - 32	3 - 32	5 - 32	4 - 32	10	4	7	-	10	4	7	-	12	4	10	3·5
						2 - 32	2 - 32	3 - 32	2 - 32	5 - 32	2 - 32											
29	C_7 - C_{11A}		4·07	GI	107	5 - 32	4 - 32	5 - 32	4 - 32	5 - 32	4 - 32	10	4	10	-	10	4	10	-	10	4	10	-
30	C_{11A} - C_{10}		2·49	GI	107	5 - 32	4 - 32	5 - 32	4 - 32	5 - 32	4 - 32	10	4	10	-	10	4	10	-	10	4	10	-
31	C_{10} - C_{13}		3·66	VARY	107	3 - 32	3 - 32	3 - 32	3 - 32	3 - 32	3 - 32	10	4	10	-	10	4	10	-	10	4	10	-
						3 - 32				2 - 32												
32	C_2-C_3/C_5-C_6 / C_8-C_9/C_{11}-C_{12}		7·845 / 7·62	GI	107	5 - 32 / 3 - 32	5 - 32 / 3 - 32	5 - 32	5 - 32 / 6 - 32	5 - 32 / 6 - 32	5 - 32 / 5 - 32	12	6	21	-	12	6	20	-	12	6	10	-
33	C_3 - C_6		3·66	VARY	107	3 - 32	3 - 32	3 - 32	3 - 32	3 - 32	3 - 32	10	4	10	-	10	4	10	-	10	4	10	-
						2 - 32				2 - 32												
34	C_6 - C_{11}		1·74	VARY	107	3 - 32	3 - 32	3 - 32	3 - 32	3 - 32	3 - 32	10	4	10	-	10	4	10	-	10	4	10	-
						2 - 32	2 - 32		2 - 32	2 - 32	2 - 32											
35	C_{11} - C_9		1·00	GI	107	5 - 32	5 - 32	5 - 32	5 - 32	5 - 32	5 - 32	10	4	10	-	10	4	10	-	10	4	10	-
36	C_9 - C_{12}		7·47	GI	210	5 - 32	5 - 32	5 - 32	5 - 32	5 - 32	5 - 32	12	4	8	-	12	4	8	-	12	4	8	-
						5 - 32	5 - 32		6 - 32	6 - 32	5 - 32											
37	C_{12} - C_{15}		3·66	VARY	107	3 - 32	3 - 32	3 - 32	3 - 32	3 - 32	3 - 32	10	4	10	-	10	4	10	-	10	4	10	-
										2 - 32												

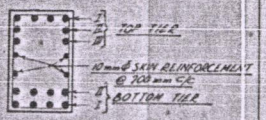


TYPICAL DETAILS FOR REINFORCEMENT OF BEAMS



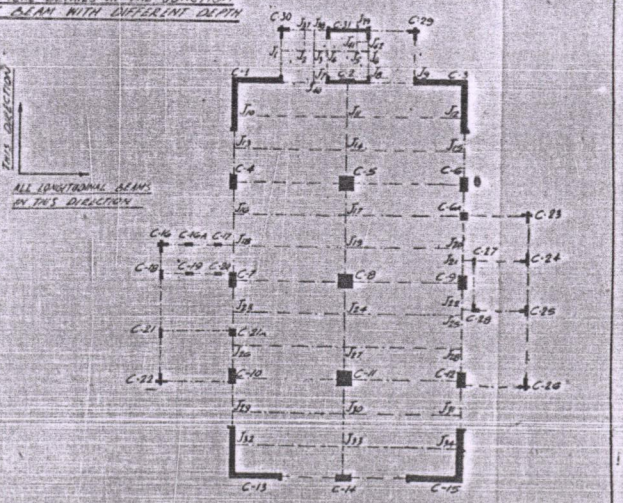
TYPICAL DETAILS AT THE JUNCTION OF BEAM RESTING OVER ANOTHER BEAM





CROSS SECTION OF BEAM.

TYPICAL DETAILS AT THE JUNCTION OF BEAM WITH DIFFERENT DEPTH



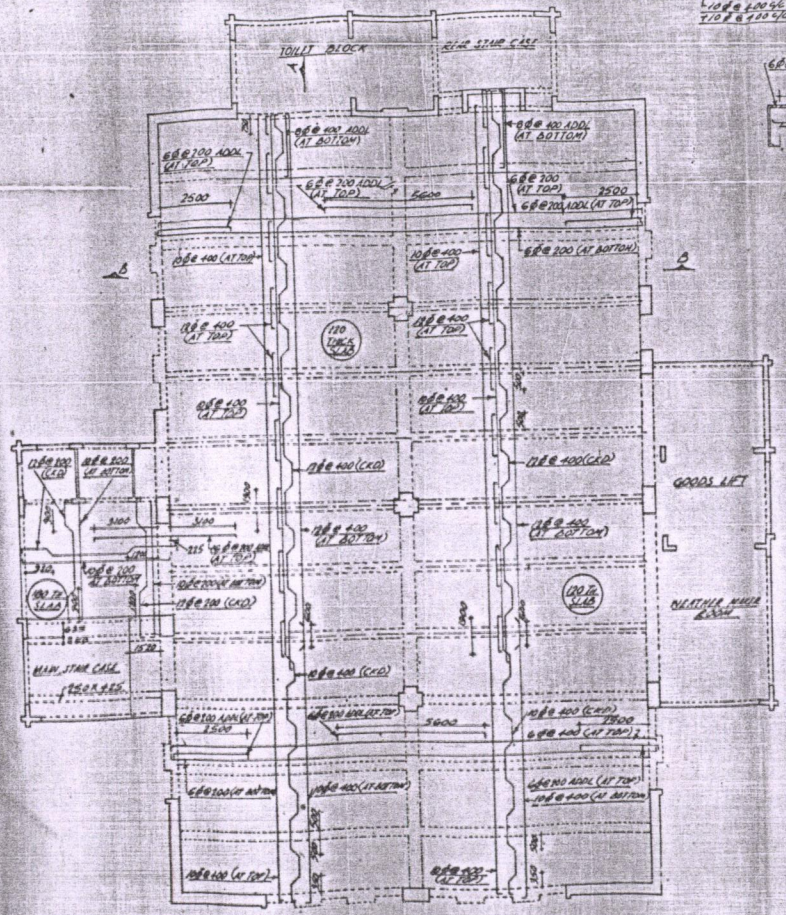
KEY PLAN

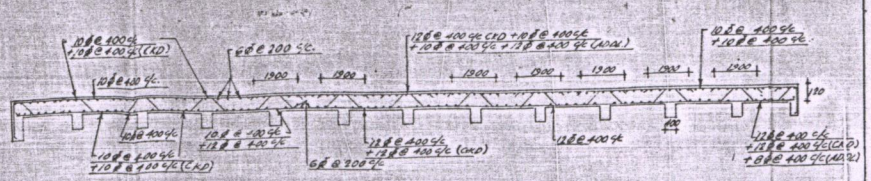
PLTAE No 3 (Para 8.2)

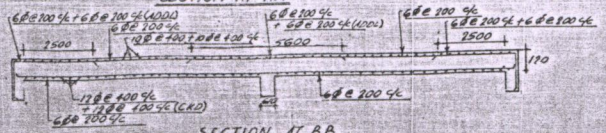
NOTES

1. CEMENT CONCRETE SHALL BE OF 1 : 2 : 4 NOMINAL MIX CONFORMING TO CPWD SPECIFICATIONS FOR WORKS AT DELHI.
2. THE REINFORCEMENT SHALL BE OF HIGH YIELD STRENGTH COLD TWISTED DEFORMED BARS CONFORM TO I.S. 1786—1966 WITH YIELD STRESS OF 0.2 PERCENT PROOF STRESS OF NOT LESS THAN 4250 KG/CM².
3. LAP LENGTH FOR REINFORCEMENT BARS SHALL BE 69 TIMES THE DIAMETER OF THE BAR FOR M-150 GRADE OF CONCRETE AND 52 TIMES THE DIAMETER OF THE BAR FOR M-200 GRADE OF CONCRETE. LAPS SHALL BE STAGGERED AND AVOIDED AT THE POINTS OF MAXIMUM BENDING MOMENT.
4. REINFORCEMENT SHALL HAVE CONCRETE COVER AND THE THICKNESS OF SUCH COVER (EXCLUSIVE OF PLASTER OR OTHER DECORATIVE FINISH) SHALL BE AS FOLLOWS :
   a. AT EACH END OF REINFORCING BAR NOT LESS THAN 25 MM NOR LESS THAN TWICE THE DIAMETER OF SUCH BAR.
   b. FOR LONGITUDINAL REINFORCING BAR IN A BEAM NOT LESS 25 MM NOR LESS THAN THE DIAMETER OF SUCH BAR.
5. THE MINIMUM VERTICAL DISTANCE BETWEEN TWO HORIZONTAL MAIN REINFORCING BARS SHALL BE 15 MM. THE MAXIMUM SIZE OF THE COARSE AGGREGATE OR THE MAXIMUM SIZE OF THE BAR WHICH EVER IS GREATER.
6. SUSPENDER BARS FOR STIRRUPS WHERE REQUIRED SHALL BE 20 MM DIAMETER.
7. CONCRETE OF THE STRENGTH SPECIFIED FOR THE SUPPORTING COLUMNS SHALL BE PROVIDED IN THE BEAMS RESTING ON THE COLUMNS FOR AN AREA FOUR TIMES THE COLUMN AREA ABOUT THE COLUMN, WELL INTEGRATED INTO THE FLOOR CONCRETE AND PLACED MONOLITHICALLY WITH THE SUPPORT.
8. BEAMS C6A-C23, C23-C24-C25-C26, C-12-C26, C25-C28, C27-C24, C27-C28, J21-C27 AND J22-C28 ALONGWITH 1000 MM LENGTH OF FLANGE (SLAB) OVER IT SHALL BE CAST WITH QUALITY CONTROLLED CEMENT CONCRETE WITH A MINIMUM WORK TEST CUBE STRENGTH OF 200 KG/Sq.Cm.

DETAILS OF BEAM REINFORCEMENT







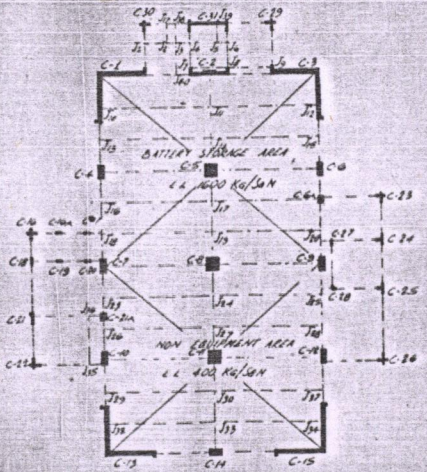


PLATE No 4 (Para 8.2)

NOTES

1. CEMENT CONCRETE FOR BEAMS OF MAIN HALL SHALL BE OF NOMINAL MIX 1:1½:3; FOR SLABS IN THE ENTRANCE BLOCK, TOILET BLOCK, GOODS LIFT AND WEATHER MAKER BLOCK, CONCRETE SHALL BE OF NOMINAL MIX 1:2:4 CONFORMING TO CPWD SPECIFICATIONS FOR WORKS AT DELHI.
2. ALL REINFORCEMENT SHALL BE OF HIGH YIELD STRENGTH COLD TWISTED DEFORMED BARS CONFORMING TO I.S. 1786—1966 WITH YIELD STRESS OR 0.2 PERCENT PROOF STRESS OF NOT LESS THAN 4250 KG/CM².
3. DISTRIBUTION BARS OF 6 MM. DIAMETER AT 20 CMS. CENTRE TO CENTRE IN THE MAIN HALL AND TOILET BLOCK AND 8 MM DIAMETER AT 20 CMS. CENTRE TO CENTRE IN OTHER SLABS SHALL BE PROVIDED WHEREVER NOT SPECIFICALLY SHOWN IN THIS DRAWING.
4. LAP LENGTH FOR REINFORCEMENT BARS SHALL BE 69 TIMES THE DIAMETER OF THE BAR FOR M-150 GRADE OF CONCRETE AND 52 TIMES THE DIAMETER OF THE BAR FOR M-200 GRADE OF CONCRETE. LAPS SHALL BE STAGGERED AND AVOIDED AT THE POINTS OF MAXIMUM BENDING MOMENT.
5. CLEAR COVER IN THE SLAB SHALL BE 13 MM. OR DIAMETER OF REINFORCEMENT WHICHEVER IS MORE.
6. STANDARD 'L' HOOKS SHALL BE PROVIDED AT THE ENDS OF ALL BARS.
7. ALL DIAMENSIONS UNLESS OTHERWISE MENTIONED ARE IN MILLIMETERS.
8. THE PORTION OF MAIN HALL SLAB BOUNDED BY COLUMNS AND SHEAR WALLS C9-C6A-C6-C3-C2-C1-C4-C7 & BEAM C7-C9 IS DESIGNED FOR A LIVE LOAD OF 1600 KG/SQ.M. THE REMAINING PORTION OF THE MAIN HALL SLAB IS DESIGNED FOR A LIVE LOAD OF 400 KG/SQ.M.

DETAILS OF SLAB REINFORCEMENT