SP 16 (1980): Design Aids for Reinforced Concrete to IS 456:1978 [CED 2: Cement and Concrete]

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Design Aids

For Reinforced Concrete

to IS : 456-1978

BUREAU OF INDIAN STANDARDS

DESIGN AIDS FOR REINFORCED CONCRETE TO IS : 456-1978

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Design Aids
For Reinforced Concrete
to IS : 456-1978

BUREAU OF INDIAN STANDARDS
BAHADUR SHAH ZAFAR MARG, NEW DLEHI 110 002

SP 16 : 1980

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FOREWORD

Users of various civil engineering codes have been feeling the need for explanatory handbooks and other compilations based on Indian Standards. The need has been further emphasized in view of the publication of the National Building Code of India 1970 and its implementation. In 1972, the Department of Science and Technology set up an Expert Group on Housing and Construction Technology under the Chairmanship of Maj-Gen Harkirat Singh. This Group carried out in-depth studies in various areas of civil engineering and construction practices. During the preparation of the Fifth Five Year Plan in 1975, the Group was assigned the task of producing a Science and Technology plan for research, development and extension work in the sector of housing and construction technology. One of the items of this plan was the production of design handbooks, explanatory handbooks and design aids based on the National Building Code and various Indian Standards and other activities in the promotion of National Building Code. The Expert Group gave high priority to this item and on the recommendation of the Department of Science and Technology the Planning Commission approved the following two projects which were assigned to the Indian Standards Institution:

a) Development programme on Code implementation for building and civil engineering construction, and

b) Typification for industrial buildings.

A Special Committee for Implementation of Science and Technology Projects (SCIP) consisting of experts connected with different aspects (*see* page viii) was set up in 1974 to advise the ISI Directorate General in identification and for guiding the development of the work under the Chairmanship of Maj-Gen Harkirat Singh, Retired Engineer-in-Chief, Army Headquarters and formerly Adviser (Construction) Planning Commission, Government of India. The Committee has so far identified subjects for several explanatory handbooks/compilations covering appropriate Indian Standards/Codes/Specifications which include the following:

Functional Requirements of Buildings
Functional Requirements of Industrial Buildings
Summaries of Indian Standards for Building Materials
Building Construction Practices
Foundation of Buildings
Explanatory Handbook on Earthquake Resistant Design and Construction (IS : 1893 IS : 4326)
Design Aids for Reinforced Concrete to IS : 456-1978
Explanatory Handbook on Masonry Code
Commentary on Concrete Code (IS : 456)
Concrete Mixes
Concrete Reinforcement
Form Work
Timber Engineering
Steel Code (IS : 800)
Loading Code
Fire Safety
Prefabrication
Tall Buildings
Design of Industrial Steel Structures
Inspection of Different Items of Building Work
Bulk Storage Structures in Steel
Bulk Storage Structures in Concrete
Liquid Retaining Structures

Construction Safety Practices
Commentaries on Finalized Building Bye-laws
Concrete Industrial Structures

One of the explanatory handbooks identified is on IS : 456-1978 Code of practice for plain and reinforced concrete (*third revision*). This explanatory handbook which is under preparation would cover the basis/source of each clause; the interpretation of the clause and worked out examples to illustrate the application of the clauses. However, it was felt that some design aids would be of help in designing as a supplement to the explanatory handbook. The objective of these design aids is to reduce design time in the use of certain clauses in the Code for the design of beams, slabs and columns in general building structures.

For the preparation of the design aids a detailed examination of the following handbooks was made :

a) CP : 110 : Part 2 : 1972 Code of practice for the structural use of concrete : Part 2 Design charts for singly reinforced beams, doubly reinforced beams and rectangular columns. British Standards Institution.
b) ACI Publication SP-17(73) Design Handbook in accordance with the strength design methods of ACI 318-71, Volume 1 (Second Edition). 1973. American Concrete Institute.
c) Reynolds (Charles E) and Steadman (James C). Reinforced Concrete Designer's Handbook. 1974. Ed. 8. Cement and Concrete Association, UK.
d) Fintel (Mark), Ed. Handbook on Concrete Engineering. 1974. Published by Van Nostrand Reinhold Company, New York.

The charts and tables included in the design aids were selected after consultation with some users of the Code in India.

The design aids cover the following:

a) Material Strength and Stress-Strain Relationships;
b) Flexural Members (Limit State Design);
c) Compression Members (Limit State Design);
d) Shear and Torsion (Limit State Design);
e) Development Length and Anchorage (Limit State Design);
f) Working Stress Method;
g) Deflection Calculation; and
h) General Tables.

The format of these design aids is as follows:

a) Assumptions regarding material strength;
b) Explanation of the basis of preparation of individual sets of design aids as related to the appropriate clauses in the Code; and
c) Worked example illustrating the use of the design aids.

Some important points to be noted in the use of the design aids are:

a) The design units are entirely in SI units as per the provisions of IS : 456-1978.
b) It is assumed that the user is well acquainted with the provisions of IS : 456-1978 before using these design aids.
c) Notations as per IS : 456-1978 are maintained here as far as possible.
d) Wherever the word 'Code' is used in this book, it refers to IS : 456-1978 Code of practice for plain and reinforced concrete (*third revision*).
e) Both charts and tables are given for flexural members. The charts can be used conveniently for preliminary design and for final design where greater accuracy is needed, tables may be used.

f) Design of columns is based on uniform distribution of steel on two faces or on four faces.

g) Charts and tables for flexural members do not take into consideration crack control and are meant for strength calculations only. Detailing rules given in the Code should be followed for crack control.

h) If the steel being used in the design has a strength which is slightly different from the one used in the Charts and Tables, the Chart or Table for the nearest value may be used and area of reinforcement thus obtained modified in proportion to the ratio of the strength of steels.

j) In most of the charts and tables, colour identification is given on the right/left-hand corner along with other salient values to indicate the type of steel; in other charts/tables salient values have been given.

These design aids have been prepared on the basis of work done by Shri P. Padmanabhan, Officer on Special Duty, ISI. Shri B. R. Narayanappa, Assistant Director, ISI was also associated with the work. The draft Handbook was circulated for review to Central Public Works Department, New Delhi; Cement Research Institute of India, New Delhi; Metallurgical and Engineering Consultants (India) Limited, Ranchi, Central Building Research Institute, Roorkee; Structural Engineering Research Centre, Madras; M/s C. R. Narayana Rao, Madras; and Shri K. K. Nambiar, Madras and the views received have been taken into consideration while finalizing the Design Aids.

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LIST OF CHARTS

LIST OF TABLES

SYMBOLS

A_c = Area of concrete
A_g = Gross area of section
A_s = Area of steel in a column or in a singly reinforced beam or slab
A_{sc} = Area of compression steel
A_{sv} = Area of stirrups
A_{st_2} = Area of additional tensile reinforcement
a_{cc} = Deflection due to creep
a_{cs} = Deflection due to shrinkage
b = Breadth of beam or shorter dimensions of a rectangular column
b_f = Effective width of flange in a T-beam
b_w = Breadth of web in a T-beam
b_1 = Centre-to-centre distance between corner bars in the direction of width
D = Overall depth of beam or slab or diameter of column or larger dimension in a rectangular column or dimension of a rectangular column in the direction of bending
D_f = Thickness of flange in a T-beam
d = Effective depth of a beam or slab
d',d^1 = distance of centroid of compression reinforcement from the extreme compression fibre of the concrete section
d_1 = Centre to centre distance between corner bars in the direction of depth
E_c = Modulus of elasticity of concrete
E_s = Modulus of elasticity of steel
e_{ax} = Eccentricity with respect to major axis (xx-axis)
e_{ay} = Eccentricity with respect to minor axis (yy-axis)
e_{min} = Minimum eccentricity
f_{cc} = Compressive stress in concrete at the level of centroid of compression reinforcement
f_{ck} = Characteristic compressive strength of concrete
f_{cr} = Flexural tensile strength (modulus of rupture) of concrete
f_s = Stress in steel
f_{sc} = Compressive stress in steel corresponding to a strain of 0·002
f_{st} = Stress in the reinforcement nearest to the tension face of a member subjected to combined axial load and bending
f_y = Characteristic yield strength of steel
f_{yd} = Design yield strength of steel
I_{eff} = Effective moment of inertia
I_{gr} = Moment of inertia of the gross section about centroidal axis, neglecting reinforcement
I_r = Moment of inertia of cracked section
K_b = Flexural stiffness of beam
K_c = Flexural stiffness of column
k = Constant or coefficient or factor
L_d = Development length of bar
l = Length of column or span of beam
l_{ex} = Effective length of a column, bending about xx-axis
l_{ey} = Effective length of a column, bending about yy-axis
M = Maximum moment under service loads
M_r = Cracking moment
M_u = Design moment for limit state Design (factored moment)
$M_{u,lim}$ = Limiting moment of resistance of a singly reinforced rectangular beam
M_{ux} = Design moment about xx-axis
M_{uy} = Design moment about yy-axis
M_{ux1} = Maximum uniaxial moment capacity of the section with axial load, bending about xx-axis

M_{uy1} = Maximum uniaxial moment capacity of the section with axial load, bending about *yy*-axis

M_{e1} = Equivalent bending moment

M_{u2} = Additional moment, $M_u - M_{u,lim}$ in doubly reinforced beams

$M_{u,lim,T}$ = Limiting moment of resistance of a T-beam

m = Modular ratio

P = Axial load

P_b = Axial load corresponding to the condition of maximum compressive strain of 0·003 5 in concrete and 0·002 in the outermost layer of tension steel in a compression member

P_u = Design axial load for limit state design (factored load)

p = Percentage of reinforcement

p_c = Percentage of compression reinforcement, $100\ A_{sc}/bd$

p_t = Percentage of tension reinforcement, $100\ A_{st}/bd$

p_{t2} = Additional percentage of tensile reinforcement in doubly reinforced beams, $100\ A_{st2}/bd$

s_v = Spacing of stirrups

T_u = Torsional moment due to factored loads

V = Shear force

V_s = Strength of shear reinforcement (working stress design)

V_u = Shear force due to factored loads

V_{us} = Strength of shear reinforcement (limit state design)

x = Depth of neutral axis at service loads

x_1 = Shorter dimension of the stirrup

x_u = Depth of neutral axis at the limit state of collapse

$x_{u,max}$ = Maximum depth of neutral axis in limit state design

y_t = Distance from centroidal axis of gross section, neglecting reinforcement, to extreme fibre in tension

y_1 = Longer dimension of stirrup

Z = Lever arm

α = Angle

γ_f = Partial safety factor for load

γ_m = Partial safety factor for material strength

ξ_{cc} = Creep strain in concrete

σ_{cbc} = Permissible stress in concrete in bending compression

σ_{cc} = Permissible stress in concrete in direct compression

σ_s = Stress in steel bar

σ_{sc} = Permissible stress in steel in compression

σ_{st} = Permissible stress in steel in tension

σ_{sv} = Permissible stress in shear reinforcement

τ_v = Nominal shear stress

τ_{bd} = Design bond stress

τ_c = Shear stress in concrete

τ_{ve} = Equivalent shear stress

$\tau_{c,max}$ = Maximum shear stress in concrete with shear reinforcement

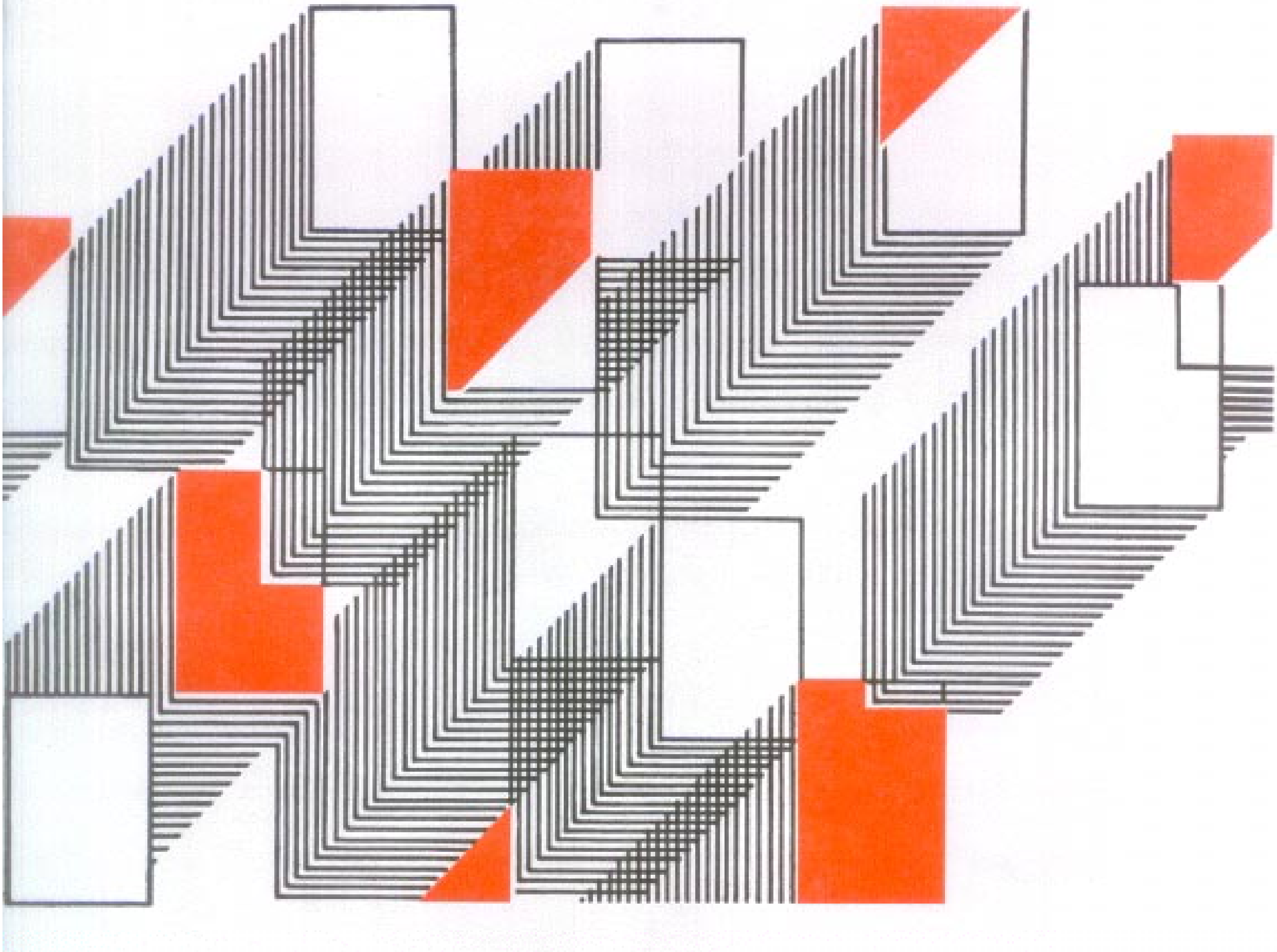
θ = Creep coefficient

ϕ = Diameter of bar

CONVERSION FACTORS

To Convert	*into*	*Multiply by*	*Conversely Multiply by*
(1)	(2)	(3)	(4)
Loads and Forces			
Newton	kilogram	0·102 0	9·807
Kilonewton	Tonne	0·102 0	9·807
Moments and Torques			
Newton metre	kilogram metre	0·102 0	9·807
Kilonewton metre	Tonne metre	0·102 0	9·807
Stresses			
Newton per mm^2	kilogram per mm^2	0·102 0	9·807
Newton per mm^2	kilogram per cm^2	10·20	0·0981

MATERIAL STRENGTH AND STRESS-STRAIN RELATIONSHIPS



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1. MATERIAL STRENGTHS AND STRESS-STRAIN RELATIONSHIPS

1.1 GRADES OF CONCRETE

The following six grades of concrete can be used for reinforced concrete work as specified in Table 2 of the Code (IS : 456-1978*):

M 15, M 20, M 25, M 30, M 35 and M 40.

The number in the grade designation refers to the characteristic compressive strength, f_{ck}, of 15 cm cubes at 28 days, expressed in N/mm^2; the characteristic strength being defined as the strength below which not more than 5 percent of the test results are expected to fall.

*Code of practice for plain and reinforced concrete (*third revision*).

1.1.1 Generally, Grades M 15 and M 20 are used for flexural members. Charts for flexural members and tables for slabs are, therefore, given for these two grades only. However, tables for design of flexural members are given for Grades M 15, M 20, M 25 and M 30.

1.1.2 The charts for compression members are applicable to all grades of concrete.

1.2 TYPES AND GRADES OF REINFORCEMENT BARS

The types of steel permitted for use as reinforcement bars in *4.6* of the Code and their characteristic strengths (specified minimum yield stress or 0·2 percent proof stress) are as follows:

Type of Steel	*Indian Standard*	*Yield Stress or 0·2 Percent Proof Stress*
Mild steel (plain bars)	IS : 432 (Part I)-1966*	26 kgf/mm^2 for bars up to 20 mm dia 24 kgf/mm^2 for bars over 20 mm dia
Mild steel (hot-rolled deformed bars)	IS : 1139-1966†	
Medium tensile steel (plain bars)	IS : 432 (Part I)-1966*	36 kgf/mm^2 for bars up to 20 mm dia 34·5 kgf/mm^2 for bars over 20 mm dia up to 40 mm dia 33 kgf/mm^2 for bars over 40 mm dia
Medium tensile steel (hot-rolled deformed bars)	IS : 1139-1966†	
High yield strength steel (hot-rolled deformed bars)	IS : 1139-1966†	42·5 kgf/mm^2 for all sizes
High yield strength steel (cold-twisted deformed bars)	IS : 1786-1979‡	415 N/mm^2 for all bar sizes 500 N/mm^2 for all bar sizes
Hard-drawn steel wire fabric	IS : 1566-1967§ and IS : 432 (Part II)-1966‖	49 kgf/mm^2

NOTE—SI units have been used in IS: 1786-1979‡; in other Indian Standards, SI units will be adopted in their next revisions.

*Specification for mild steel and medium tensile steel bars and hard-drawn steel wire for concrete reinforcement: Part I Mild steel and medium tensile steel bars (*second revision*).

†Specification for hot rolled mild steel, medium tensile steel and high yield strength steel deformed bars for concrete reinforcement (*revised*).

‡Specification for cold-worked steel high strength deformed bars for concrete reinforcement (*second revision*).

§Specification for hard-drawn steel wire fabric for concrete reinforcement (*first revision*).

‖Specification for mild steel and medium tensile steel bars and hard-drawn steel wire for concrete reinforcement: Part II Hard drawn steel wire (*second revision*).

Taking the above values into consideration, most of the charts and tables have been prepared for three grades of steel having characteristic strength f_y equal to 250 N/mm², 415 N/mm² and 500 N/mm².

1.2.1 If the steel being used in a design has a strength which is slightly different from the above values, the chart or table for the nearest value may be used and the area of reinforcement thus obtained be modified in proportion to the ratio of the strengths.

1.2.2 Five values of f_y (including the value for hard-drawn steel wire fabric) have been included in the tables for singly reinforced sections.

1.3 STRESS-STRAIN RELATIONSHIP FOR CONCRETE

The Code permits the use of any appropriate curve for the relationship between the compressive stress and strain distribution in concrete, subject to the condition that it results in the prediction of strength in substantial agreement with test results [*37.1*(c) of the Code]. *An acceptable stress-strain curve* (see *Fig. 1*) *given in Fig. 20 of the Code will form the basis for the design aids in this publication. The compressive strength of concrete in the structure is assumed to be* $0{\cdot}67 f_{ck}$. *With a value of 1·5 for the partial safety factor* γ_m *for material strength (35.4.2.1 of the Code), the maximum compressive stress in concrete for design purpose is* $0{\cdot}446\ f_{ck}$ (see *Fig. 1*).

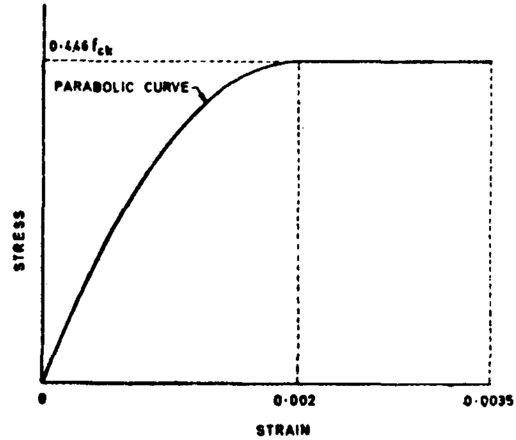


FIG. 1 DESIGN STRESS-STRAIN CURVE FOR CONCRETE

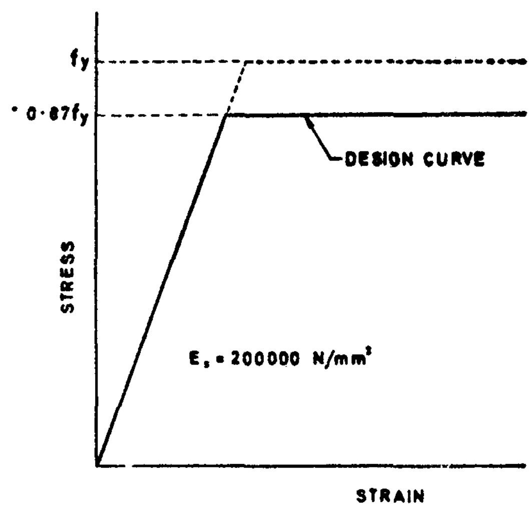


FIG. 2 STRESS-STRAIN CURVE FOR MILD STEEL

1.4 STRESS-STRAIN RELATIONSHIP FOR STEEL

The modulus of elasticity of steel, E_s, is taken as 200 000 N/mm² (*4.6.2* of the Code). This value is applicable to all types of reinforcing steels.

The design yield stress (or 0·2 percent proof stress) of steel is equal to f_y/γ_m. With a value of 1·15 for γ_m (*35.4.2.1* of the Code), the design yield stress f_{yd} becomes $0{\cdot}87\,f_y$. *The stress-strain relationship for steel in tension and compression is assumed to be the same.*

For mild steel, the stress is proportional to strain up to yield point and thereafter the strain increases at constant stress (*see* Fig. 2). For cold-worked bars, the stress-strain relationship given in Fig. 22 of the Code will be adopted. According to this, the stress is proportional to strain up to a stress of $0{\cdot}8\ f_y$. Thereafter, the stress-strain curve is defined as given below:

Stress	*Inelastic strain*
$0{\cdot}80 f_y$	Nil
$0{\cdot}85 f_y$	0·000 1
$0{\cdot}90 f_y$	0·000 3
$0{\cdot}95 f_y$	0·000 7
$0{\cdot}975 f_y$	0·001 0
$1{\cdot}0 f_y$	0·002 0

The stress-strain curve for design purposes is obtained by substituting f_{yd} for f_y in the above. For two grades of cold-worked bars with 0·2 percent proof stress values of 415 N/mm² and 500 N/mm² respectively, the values of total strains and design stresses corresponding to the points defined above are given in Table A (*see* page 6). The stress-strain curves for these two grades of cold-worked bars have been plotted in Fig. 3.

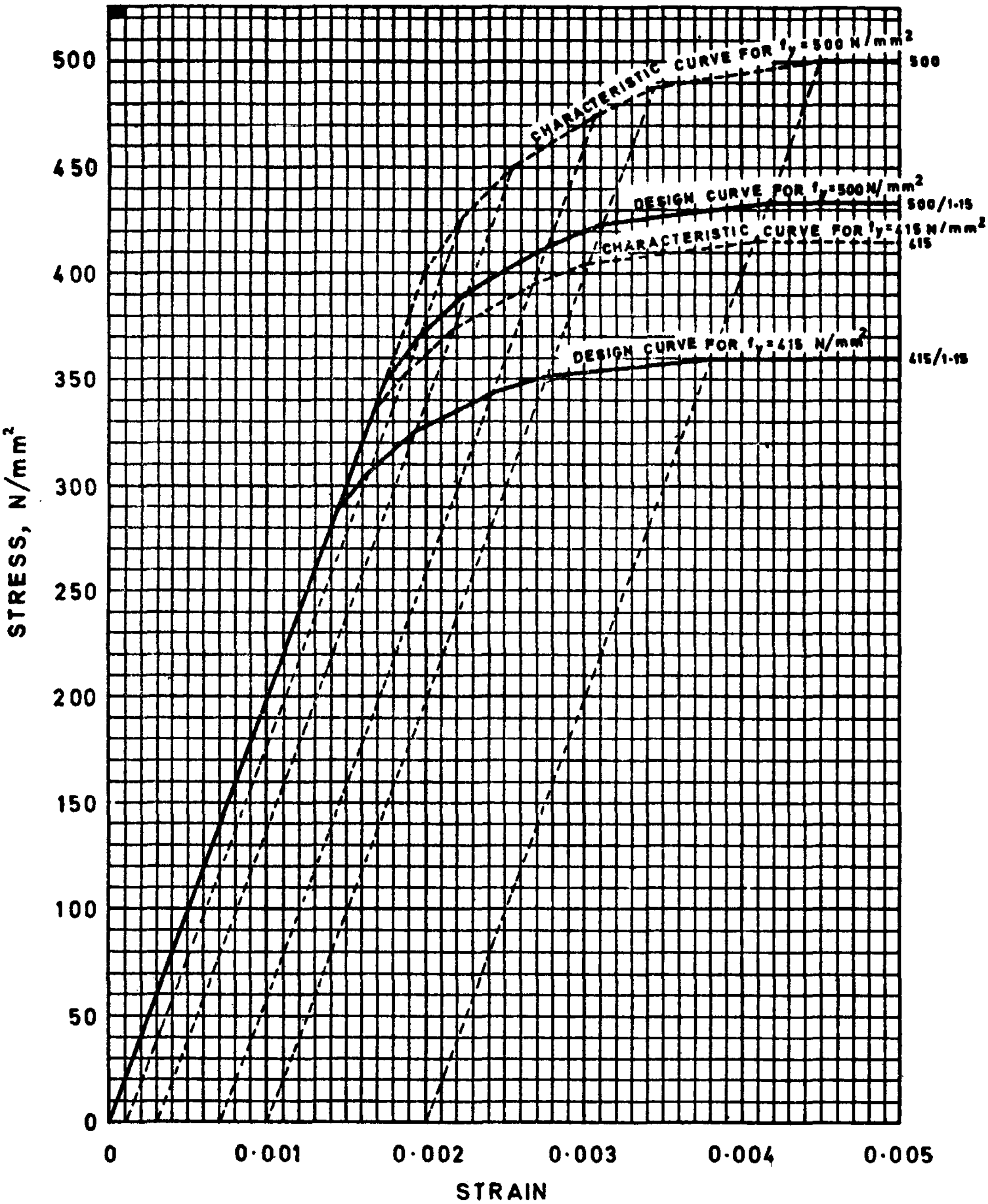


FIG. 3 STRESS-STRAIN CURVES FOR COLD-WORKED STEELS

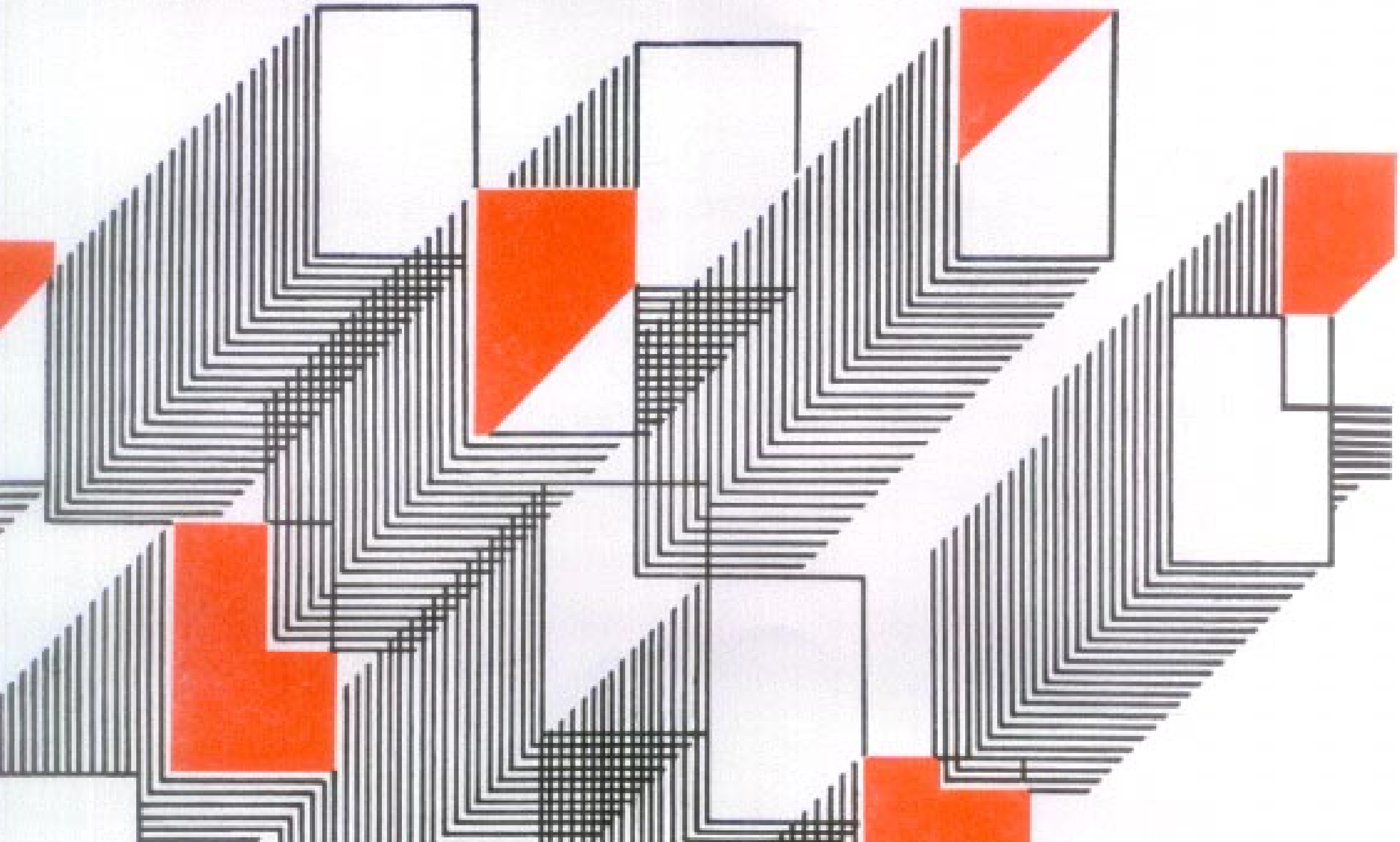
TABLE A SALIENT POINTS ON THE DESIGN STRESS-STRAIN CURVE FOR COLD-WORKED BARS

(*Clause 1.4*)

STRESS LEVEL	f_y = 415 N/mm²		f_y = 500 N/mm²	
	Strain	Stress	Strain	Stress
(1)	(2)	(3) N/mm²	(4)	(5) N/mm²
0·80 f_{yd}	0·001 44	288·7	0·001 74	347·8
0·85 f_{yd}	0·001 63	306·7	0·001 95	369·6
0·90 f_{yd}	0·001 92	324·8	0·002 26	391·3
0·95 f_{yd}	0·002 41	342·8	0·002 77	413·0
0·975 f_{yd}	0·002 76	351·8	0·003 12	423·9
1·0 f_{yd}	0·003 80	360·9	0·004 17	434·8

NOTE — Linear interpolation may be done for intermediate values.

FLEXURAL MEMBERS



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2. FLEXURAL MEMBERS

2.1 ASSUMPTIONS

The basic assumptions in the design of flexural members for the limit state of collapse are given below (*see 37.1* of the Code):

a) Plane sections normal to the axis of the member remain plane after bending. This means that the strain at any point on the cross section is directly proportional to the distance from the neutral axis.

b) The maximum strain in concrete at the outermost compression fibre is 0·003 5.

c) The design stress-strain relationship for concrete is taken as indicated in Fig. 1.

d) The tensile strength of concrete is ignored.

e) The design stresses in reinforcement are derived from the strains using the stress-strain relationships given in Fig. 2 and 3.

f) The strain in the tension reinforcement is to be not less than

$$\frac{0{\cdot}87 f_y}{E_s} + 0{\cdot}002.$$

This assumption is intended to ensure ductile failure, that is, the tensile reinforcement has to undergo a certain degree of inelastic deformation before the concrete fails in compression.

2.2 MAXIMUM DEPTH OF NEUTRAL AXIS

Assumptions (b) and (f) govern the maximum depth of neutral axis in flexural members. The strain distribution across a member corresponding to those limiting conditions is shown in Fig. 4. The maximum depth of neutral axis $x_{u,\,max}$ is obtained directly from the strain diagram by considering similar triangles.

$$\frac{x_{u,max}}{d} = \frac{0{\cdot}003\,5}{(0{\cdot}005\,5 + 0{\cdot}87 f_y/E_s)}$$

The values of $\frac{x_{u,max}}{d}$ for three grades of reinforcing steel are given in Table B.

TABLE B VALUES OF $\frac{x_{u,max}}{d}$ FOR DIFFERENT GRADES OF STEEL

(*Clause 2.2*)

f_y, N/mm²	250	415	500
$\frac{x_{u,max}}{d}$	0·531	0·479	0·456

2.3 RECTANGULAR SECTIONS

The compressive stress block for concrete is represented by the design stress-strain curve as in Fig. 1. It is seen from this stress block (*see* Fig. 4) that the centroid of compressive force in a rectangular section lies

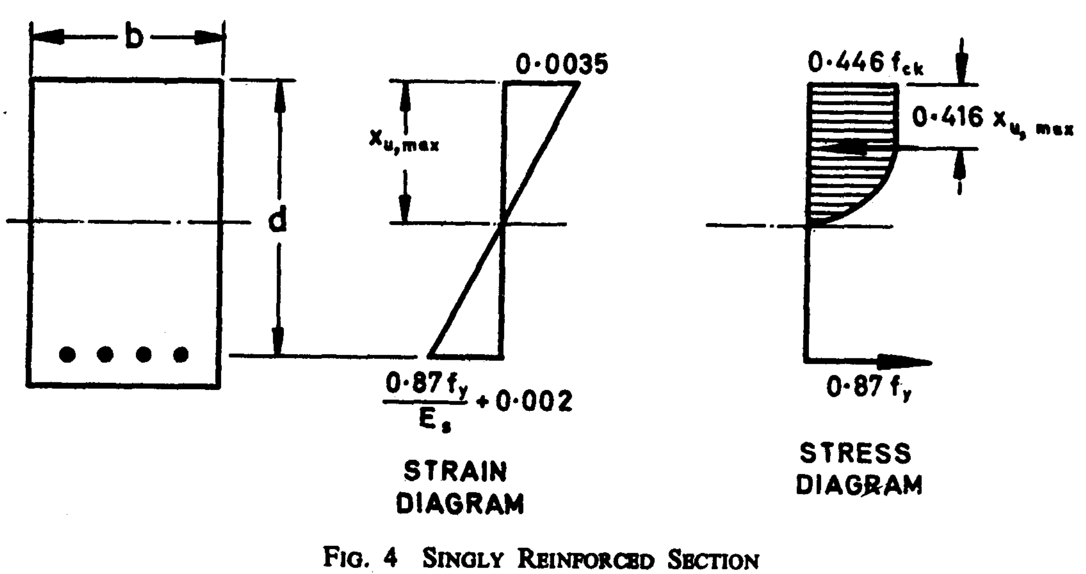

FIG. 4 SINGLY REINFORCED SECTION

at a distance of 0·416 x_u (which has been rounded off to 0·42 x_u in the code) from the extreme compression fibre; and the total force of compression is 0·36 $f_{ck}\,bx_u$. The lever arm, that is, the distance between the centroid of compressive force and centroid of tensile force is equal to $(d - 0{\cdot}416\ x_u)$. Hence the upper limit for the moment of resistance of a singly reinforced rectangular section is given by the following equation:

$$M_{u,lim} = 0{\cdot}36 f_{ck}\, b x_{u,max} \times (d - 0{\cdot}416\ x_{u,max})$$

Substituting for $x_{u,max}$ from Table B and transposing $f_{ck}\,bd^2$, we get the values of the limiting moment of resistance factors for singly reinforced rectangular beams and slabs. These values are given in Table C. The tensile reinforcement percentage, $p_{t,lim}$ corresponding to the limiting moment of resistance is obtained by equating the forces of tension and compression.

$$\frac{p_{t,lim}\, bd\,(0{\cdot}87 f_y)}{100} = 0{\cdot}36 f_{ck}\, b x_{u,max}$$

Substituting for $x_{u,max}$ from Table B, we get the values of $p_{t,lim}\ f_y/f_{ck}$ as given in Table C.

TABLE C LIMITING MOMENT OF RESISTANCE AND REINFORCEMENT INDEX FOR SINGLY REINFORCED RECTANGULAR SECTIONS

(*Clause 2.3*)

f_y, N/mm²	250	415	500
$\frac{M_{u,lim}}{f_{ck}\,bd^2}$	0·149	0·138	0·133
$\frac{p_{t,lim}\, f_y}{f_{ck}}$	21·97	19·82	18·87

The values of the limiting moment of resistance factor M_u/bd^2 for different grades of concrete and steel are given in Table D. The corresponding percentages of reinforcements are given in Table E. These are the maximum permissible percentages for singly reinforced sections.

TABLE D LIMITING MOMENT OF RESISTANCE FACTOR $M_{u,lim}/bd^2$, N/mm² FOR SINGLY REINFORCED RECTANGULAR SECTIONS

(*Clause 2.3*)

f_{ck}, N/mm²	f_y, N/mm² 250	415	500
15	2·24	2·07	2·00
20	2·98	2·76	2·66
25	3·73	3·45	3·33
30	4·47	4·14	3·99

TABLE E MAXIMUM PERCENTAGE OF TENSILE REINFORCEMENT $p_{t,lim}$ FOR SINGLY REINFORCED RECTANGULAR SECTIONS

(*Clause 2.3*)

f_{ck}, N/mm²	f_y, N/mm² 250	415	500
15	1·32	0·72	0·57
20	1·76	0·96	0·76
25	2·20	1·19	0·94
30	2·64	1·43	1·13

2.3.1 *Under-Reinforced Sections*

Under-reinforced section means a singly reinforced section with reinforcement percentage not exceeding the appropriate value given in Table E. For such sections, the depth of neutral axis x_u will be smaller than $x_{u,max}$. The strain in steel at the limit state of collapse will, therefore, be more than $\frac{0{\cdot}87 f_y}{E_s} + 0{\cdot}002$ and, the design stress in steel will be $0{\cdot}87 f_y$. The depth of neutral axis is obtained by equating the forces of tension and compression.

$$\frac{p_t\, bd}{100}(0{\cdot}87 f_y) = 0{\cdot}36 f_{ck}\, b\, x_u$$

$$\frac{x_u}{d} = \left(\frac{p_t}{100}\right)\frac{0{\cdot}87 f_y}{0{\cdot}36 f_{ck}}$$

The moment of resistance of the section is equal to the product of the tensile force and the lever arm.

$$M_u = \frac{p_t\, bd}{100}(0{\cdot}87 f_y)(d - 0{\cdot}416\ x_u)$$

$$= 0{\cdot}87 f_y\left(\frac{p_t}{100}\right)\left(1 - 0{\cdot}416\frac{x_u}{d}\right)bd^2$$

Substituting for $\frac{x_u}{d}$ we get

$$M_u = 0{\cdot}87 f_y\left(\frac{p_t}{100}\right) \times \left[1 - 1{\cdot}005\frac{f_y}{f_{ck}}\left(\frac{p_t}{100}\right)\right]bd^2$$

2.3.1.1 *Charts 1* to *18* have been prepared by assigning different values to M_u/b and plotting *d versus* p_t. The moment values in the charts are in units of kN.m per metre width. Charts are given for three grades of steel and two grades of concrete, namely M 15 and M 20, which are most commonly used for flexural members. Tables 1 to 4 cover a wider range, that is, five values of f_y and four grades of concrete up to M 30. In these tables, the values of percentage of reinforcement p_t have been tabulated against M_u/bd^2.

2.3.1.2 The moment of resistance of slabs, with bars of different diameters and spacings are given in Tables 5 to 44. Tables are given for concrete grades M 15 and M 20, with two grades of steel. Ten different thicknesses ranging from 10 cm to 25 cm, are included. These tables take into account *25.5.2.2* of the Code, that is, the maximum bar diameter does not exceed one-eighth the thickness of the slab. Clear cover for reinforcement has been taken as 15 mm or the bar diameter, whichever is greater [*see 25.4.1*(d) of the Code]. In these tables, the zeros at the top right hand corner indicate the region where the reinforcement percentage would exceed $p_{t,lim}$; and the zeros at the lower left hand corner indicate the region where the reinforcement is less than the minimum according to *25.5.2.1* of the Code.

Example 1 Singly Reinforced Beam

Determine the main tension reinforcement required for a rectangular beam section with the following data:

Size of beam	30 × 60 cm
Concrete mix	M 15
Characteristic strength of reinforcement	415 N/mm²
*Factored moment	170 kN.m

Assuming 25 mm dia bars with 25 mm clear cover,

$$\text{Effective depth} = 60 - 2{\cdot}5 - \frac{2{\cdot}5}{2} = 56{\cdot}25 \text{ cm}$$

From Table D, for $f_y = 415$ N/mm² and $f_{ck} = 15$ N/mm²

$$M_{u,lim}/bd^2 = 2{\cdot}07 \text{ N/mm}^2 = \frac{2{\cdot}07}{1\,000} \times (1\,000)^2 = 2{\cdot}07 \times 10^3 \text{ kN/m}^2$$

$$\therefore \quad M_{u,lim} = 2{\cdot}07 \times 10^3 bd^2 = 2{\cdot}07 \times 10^3 \times \frac{30}{100} \times \left(\frac{56{\cdot}25}{100}\right)^2 = 196{\cdot}5 \text{ kN.m}$$

Actual moment of 170 kN.m is less than $M_{u,lim}$. The section is therefore to be designed as a singly reinforced (under-reinforced) rectangular section.

METHOD OF REFERRING TO FLEXURE CHART

For referring to Chart, we need the value of moment per metre width.

$$M_u/b = \frac{170}{0{\cdot}3} = 567 \text{ kN.m per metre width.}$$

*The term 'factored moment' means the moment due to characteristic loads multiplied by the appropriate value of partial safety factor γ_f.

Referring to *Chart 6*, corresponding to $M_u/b = 567$ kN.m and $d = 56{\cdot}25$ cm,

$$\text{Percentage of steel } p_t = \frac{100 A_s}{bd} = 0{\cdot}6$$

$$\therefore \; A_s = \frac{0{\cdot}6\, bd}{100} = \frac{0{\cdot}6 \times 30 \times 56{\cdot}25}{100} = 10{\cdot}1 \text{ cm}^2$$

METHOD OF REFERRING TO TABLES

For referring to Tables, we need the value of $\frac{M_u}{bd^2}$

$$\frac{M_u}{bd^2} = \frac{170 \times 10^6}{30 \times 56{\cdot}25 \times 56{\cdot}25 \times 10^3} = 1{\cdot}79 \text{ N/mm}^2$$

From *Table 1*,

Percentage of reinforcement, $p_t = 0{\cdot}594$

$$\therefore \; A_s = \frac{0{\cdot}594 \times 30 \times 56{\cdot}25}{100} = 10{\cdot}02 \text{ cm}^2$$

Example 2 Slab

Determine the main reinforcement required for a slab with the following data:

Factored moment	9·60 kN.m per metre width
Depth of slab	10 cm
Concrete mix	M 15
Characteristic strength of reinforcement	a) 415 N/mm² b) 250 N/mm²

METHOD OF REFERRING TO TABLES FOR SLABS

Referring to *Table 15* (for $f_y = 415$ N/mm²), directly we get the following reinforcement for a moment of resistance of 9·6 kN.m per metre width:

8 mm dia at 13 cm spacing
or 10 mm dia at 20 cm spacing

Reinforcement given in the tables is based on a cover of 15 mm or bar diameter whichever is greater.

METHOD OF REFERRING TO FLEXURE CHART

Assume 10 mm dia bars with 15 mm cover,

$$d = 10 - 1{\cdot}5 - \frac{1{\cdot}0}{2} = 8 \text{ cm}$$

a) For $f_y = 415$ N/mm²
From Table D, $M_{u,lim}/bd^2 = 2{\cdot}07$ N/mm²

$$\therefore \; M_{u,lim} = 2{\cdot}07 \times 10^3 \times \frac{100}{100} \times \left(\frac{8}{100}\right)^2 = 13{\cdot}25 \text{ kN.m}$$

Actual bending moment of 9·60 kN.m is less than the limiting bending moment.

Referring to *Chart 4*, reinforcement percentage, $p_t = 0{\cdot}475$

Referring to *Chart 90*, provide
8 mm dia at 13 cm spacing
or 10 mm dia at 20 cm spacing.

Alternately,

$$A_s = 0{\cdot}475 \times 100 \times \frac{8}{100} = 3{\cdot}8 \text{ cm}^2 \text{ per}$$

metre width.

From *Table 96*, we get the same reinforcement as before.

b) For $f_y = 250$ N/mm²

From Table D, $M_{u,lim}/bd^2 = 2{\cdot}24$ N/mm²

$$M_{u,lim} = 2{\cdot}24 \times 10^3 \times 1 \times \left(\frac{8}{100}\right)^2$$
$$= 14{\cdot}336 \text{ kN.m}$$

Actual bending moment of 9·6 kN.m is less than the limiting bending moment.

Referring to *Chart 1*, reinforcement percentage, $p_t = 0{\cdot}78$

Referring to *Chart 90*, provide 10 mm dia at 13 cm spacing.

2.3.2 Doubly Reinforced Sections — Doubly reinforced sections are generally adopted when the dimensions of the beam have been predetermined from other considerations and the design moment exceeds the moment of resistance of a singly reinforced section. The additional moment of resistance needed is obtained by providing compression reinforcement and additional tensile reinforcement. The moment of resistance of a doubly reinforced section is thus the sum of the limiting moment of resistance $M_{u,lim}$ of a singly reinforced section and the additional moment of resistance M_{u2}. Given the values of M_u which is greater than $M_{u,lim}$, the value of M_{u2} can be calculated.

$$M_{u2} = M_u - M_{u,lim}$$

The lever arm for the additional moment of resistance is equal to the distance between centroids of tension reinforcement and compression reinforcement, that is $(d - d')$ where d' is the distance from the extreme compression fibre to the centroid of compression reinforcement. Therefore, considering the moment of resistance due to the additional tensile reinforcement and the compression reinforcement we get the following:

$$M_{u2} = A_{st2}\,(0{\cdot}87\,f_y)\,(d - d')$$

also, $M_{u2} = A_{sc}\,(f_{sc} - f_{cc})\,(d - d')$

where

- A_{st2} is the area of additional tensile reinforcement,
- A_{sc} is the area of compression reinforcement,
- f_{sc} is the stress in compression reinforcement, and
- f_{cc} is the compressive stress in concrete at the level of the centroid of compression reinforcement.

Since the additional tensile force is balanced by the additional compressive force,

$$A_{sc}\,(f_{sc} - f_{cc}) = A_{st2}\,(0{\cdot}87\,f_y)$$

Any two of the above three equations may be used for finding A_{st2} and A_{sc}. The total tensile reinforcement A_{st} is given by,

$$A_{st} = p_{t,lim}\,\frac{bd}{100} + A_{st2}$$

It will be noticed that we need the values of f_{sc} and f_{cc} before we can calculate A_{sc}. The approach given here is meant for design of sections and not for analysing a given section. The depth of neutral axis is, therefore, taken as equal to $x_{u,max}$. As shown in Fig. 5, strain at the level of the compression reinforcement will be equal to $0{\cdot}003\,5\left(1 - \frac{d'}{x_{u,max}}\right)$

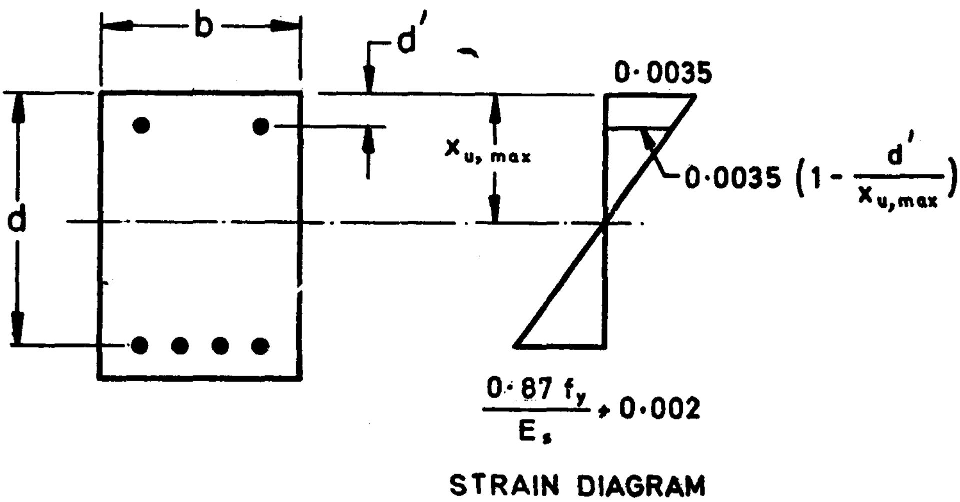

FIG. 5 DOUBLY REINFORCED SECTION

For values of d'/d up to 0·2, f_{cc} is equal to 0·446 f_{ck}; and for mild steel reinforcement f_{sc} would be equal to the design yield stress of 0·87 f_y. When the reinforcement is cold-worked bars, the design stress in compression reinforcement f_{sc} for different values of d'/d up to 0·2 will be as given in Table F.

TABLE F STRESS IN COMPRESSION REINFORCEMENT f_{sc}, N/mm² IN DOUBLY REINFORCED BEAMS WITH COLD-WORKED BARS

(*Clause 2.3 2*)

f_y, N/mm²	d'/d			
	0·05	0·10	0·15	0·20
415	355	353	342	329
500	424	412	395	370

2.3.2.1 A_{st_2} has been plotted against $(d - d')$ for different values of M_{u2} in *Charts 19* and *20*. These charts have been prepared for $f_s = 217·5$ N/mm² and it is directly applicable for mild steel reinforcement with yield stress of 250 N/mm². Values of A_{st_2} for other grades of steel and also the values of A_{sc} can be obtained by multiplying the value read from the chart by the factors given in Table G. The multiplying factors for A_{sc}, given in this Table, are based on a value of f_{cc} corresponding to concrete grade M 20, but it can be used for all grades of concrete with little error.

TABLE G MULTIPLYING FACTORS FOR USE WITH CHARTS 19 AND 20

(*Clause 2.3.2.1*)

f_y, N/mm²	Factor for A_{st_2}	Factor for A_{sc} for d'/d			
		0·05	0·10	0·15	0·20
250	1·00	1·04	1·04	1·04	1·04
415	0·60	0·63	0·63	0·65	0·68
500	0·50	0·52	0·54	0·56	0·60

2.3.2.2 The expression for the moment of resistance of a doubly reinforced section may also be written in the following manner:

$$M_u = M_{u,lim} + \frac{p_{t2}bd}{100}(0·87 f_y)(d - d')$$

$$\frac{M_u}{bd^2} = \frac{M_{u,lim}}{bd^2} + \frac{p_{t2}}{100}(0·87 f_y)\left(1 - \frac{d'}{d}\right)$$

where

p_{t2} is the additional percentage of tensile reinforcement.

$$p_t = p_{t,lim} + p_{t2}$$

$$p_c = p_{t2}\left[\frac{0·87 f_y}{f_{sc} - f_{cc}}\right]$$

The values of p_t and p_c for four values of d'/d up to 0·2 have been tabulated against M_u/bd^2 in Tables 45 to 56. Tables are given for three grades of steel and four grades of concrete.

Example 3 Doubly Reinforced Beam

Determine the main reinforcements required for a rectangular beam section with the following data:

Size of beam	30 × 60 cm
Concrete mix	M 15
Characteristic strength of reinforcement	415 N/mm²
Factored moment	320 kN.m

Assuming 25 mm dia bars with 25 mm clear cover,

$$d = 60 - 2·5 - \frac{2·5}{2} = 56·25 \text{ cm}$$

From Table D, for $f_y = 415$ N/mm² and $f_{ck} = 15$ N/mm²

$M_{u,lim}/bd^2 = 2·07$ N/mm² $= 2·07 \times 10^3$ kN/m²

$\therefore$ $M_{u,lim} = 2·07 \times 10^3\, bd^2$

$$= 2·07 \times 10^3 \times \frac{30}{100} \times \frac{56·25}{100} \times \frac{56·25}{100}$$

$$= 196·5 \text{ kN.m}$$

Actual moment of 320 kN.m is greater than $M_{u,lim}$

$\therefore$ The section is to be designed as a doubly reinforced section.

Reinforcement from Tables

$$\frac{M_u}{bd^2} = \frac{320}{0·3 \times (0·562\,5)^2 \times 10^3} = 3·37 \text{ N/mm}^2$$

$$d'/d = \left(\frac{2·5 + 1·25}{56·25}\right) = 0·07$$

Next higher value of $d'/d = 0·1$ will be used for referring to Tables.

Referring to *Table 49* corresponding to

$$M_u/bd^2 = 3·37 \text{ and } \frac{d'}{d} = 0·1,$$

$p_t = 1·117$, $p_c = 0·418$

$\therefore$ $A_{st} = 18·85$ cm², $A_{sc} = 7·05$ cm²

Reinforcement from Charts

$(d - d') = (56·25 - 3·75) = 52·5$ cm

$M_{u2} = (320 - 196·5) = 123·5$ kN.m

Chart is given only for $f_y = 250$ N/mm²; therefore use *Chart 20* and modification factors according to *Table G*.

Referring to *Chart 20*,

A_{st_2} (for $f_y = 250$ N/mm²) $= 10·7$ cm²

Using modification factors given in *Table G* for $f_y = 415$ N/mm^2,

$$A_{st_2} = 10{\cdot}7 \times 0{\cdot}60 = 6{\cdot}42 \text{ cm}^2$$

$$A_{sc} = 10{\cdot}7 \times 0{\cdot}63 = 6{\cdot}74 \text{ cm}^2$$

Referring to *Table E*,

$$p_{t,lim} = 0{\cdot}72$$

$$\therefore \quad A_{st,lim} = 0{\cdot}72 \times \frac{56{\cdot}25 \times 30}{100} = 12{\cdot}15 \text{ cm}^2$$

$$A_{st} = 12{\cdot}15 + 6{\cdot}42 = 18{\cdot}57 \text{ cm}^2$$

These values of A_{st} and A_{sc} are comparable to the values obtained from the table.

2.4 T-SECTIONS

The moment of resistance of a T-beam can be considered as the sum of the moment of resistance of the concrete in the web of width b_w and the contribution due to flanges of width b_f.

The maximum moment of resistance is obtained when the depth of neutral axis is $x_{u,max}$. When the thickness of flange is small, that is, less than about 0·2 d, the stress in the flange will be uniform or nearly uniform (*see* Fig. 6) and the centroid of the compressive force in the flange can be taken at $D_f/2$ from the extreme compression fibre. Therefore, the following expression is obtained for the limiting moment of resistance of T-beams with small values of D_f/d.

$$M_{u,lim,T} = M_{u,lim,web} + 0{\cdot}446 f_{ck} \times (b_f - b_w)\, D_f \left(d - \frac{d_f}{2} \right)$$

where $M_{u,lim,web}$

$= 0{\cdot}36\, f_{ck}\, b_w x_{u,max}\, (d - 0{\cdot}416\, x_{u,max})$.

The equation given in *E-2.2* of the Code is the same as above, with the numericals rounded off to two decimals. When the flange thickness is greater than about 0·2 d, the above expression is not correct because the stress distribution in the flange would not be uniform. The expression given in *E-2.2.1* of the Code is an approximation which makes allowance for the variation of stress in the flange. This expression is obtained by substituting y_f for D_f in the equation of *E-2.2* of the Code; y_f being equal to $(0{\cdot}15\ x_{u,max} + 0{\cdot}65\, D_f)$ but not greater than D_f. With this modification,

$$M_{u,lim,T} = M_{u,lim,web} + 0{\cdot}446\, f_{ck} \times (b_f - b_w) y_f \left(d - \frac{y_f}{2} \right)$$

Dividing both sides by $f_{ck}\, b_w\, d^2$,

$$\frac{M_{u,lim,T}}{f_{ck}\, b_w\, d^2} = \frac{M_{u,lim,web}}{f_{ck}\, b_w\, d^2} + 0{\cdot}446 \times \left(\frac{b_f}{b_w} - 1 \right) \frac{y_f}{d} \left(1 - \frac{y_f}{2d} \right)$$

where

$$\frac{y_f}{d} = 0{\cdot}15 \frac{x_{u,max}}{d} + 0{\cdot}65 \frac{D_f}{d}$$

$$\text{but } \frac{y_f}{d} \le \frac{D_f}{d}$$

Using the above expression, the values of the moment of resistance factor $M_{u,lim,T}/f_{ck}\, b_w d^2$ for different values of b_f/b_w and D_f/d have been worked out and given in Tables 57 to 59 for three grades of steel.

2.5 CONTROL OF DEFLECTION

2.5.1 The deflection of beams and slabs would generally be within permissible limits if the ratio of span to effective depth of the member does not exceed the values obtained in accordance with *22.2.1* of the Code. The following basic values of span to effective depth are given:

Simply supported	20
Continuous	26
Cantilever	7

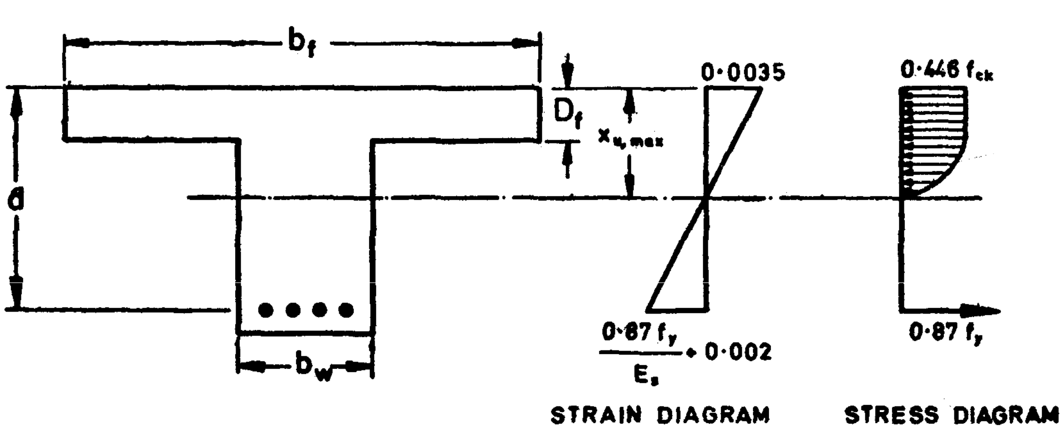

FIG. 6 T-SECTION

Further modifying factors are given in order to account for the effects of grade and percentage of tension reinforcement and percentage of compression reinforcement.

2.5.2 In normal designs where the reinforcement provided is equal to that required from strength considerations, the basic values of span to effective depth can be multiplied by the appropriate values of the modifying factors and given in a form suitable for direct reference. Such charts have been prepared as explained below:

a) The basic span to effective depth ratio for simply supported members is multiplied by the modifying factor for tension reinforcement (Fig. 3 of the Code) and plotted as the base curve in the chart. A separate chart is drawn for each grade of steel. In the chart, span to effective depth ratio is plotted on the vertical axis and the tensile reinforcement percentage is plotted on the horizontal axis.

b) When the tensile reinforcement exceeds $p_{t,lim}$ the section will be doubly reinforced. The percentage of compression reinforcement is proportional to the additional tensile reinforcement $(p_t - p_{t,lim})$ as explained in *2.3.2*. However, the value of $p_{t,lim}$ and p_c will depend on the grade of concrete also. Therefore, the values of span to effective depth ratio according to base curve is modified as follows for each grade of concrete:

   1) For values of p_t greater than the appropriate value of $p_{t,lim}$, the value of $(p_t - p_{t,lim})$ is calculated and then the percentage of compression reinforcement p_c required is calculated. Thus, the value of p_c corresponding to a value of p_t is obtained. (For this purpose d'/d has been assumed as 0·10 but the chart, thus obtained can generally be used for all values of d'/d in the normal range, without significant error in the value of maximum span to effective depth ratio.)

   2) The value of span to effective depth ratio of the base curve is multiplied by the modifying factor for compression reinforcement from Fig. 4 of the Code.

   3) The value obtained above is plotted on the same Chart in which the base curve was drawn earlier. Hence the span to effective depth ratio for doubly reinforced section is plotted against the tensile reinforcement percentage p_t without specifically indicating the value of p_c on the Chart.

2.5.3 The values read from these Charts are directly applicable for simply supported members of rectangular cross section for spans up to 10 m. For simply supported or continuous spans larger than 10 m, the values should be further multiplied by the factor (10/span in metres). For continuous spans or cantilevers, the values read from the charts are to be modified in proportion to the basic values of span to effective depth ratio. The multiplying factors for this purpose are as follows:

Continuous spans	1·3
Cantilevers	0·35

In the case of cantilevers which are longer than 10 m the Code recommends that the deflections should be calculated in order to ensure that they do not exceed permissible limits.

2.5.4 For flanged beams, the Code recommends that the values of span to effective depth ratios may be determined as for rectangular sections, subject to the following modifications:

a) The reinforcement percentage should be based on the area $b_f d$ while referring the charts.

b) The value of span to effective depth ratio obtained as explained earlier should be reduced by multiplying by the following factors:

b_f/b_w	*Factor*
1·0	1·0
$>$3·33	0·8

For intermediate values, linear interpolation may be done.

NOTE — The above method for flanged beams may sometimes give anomalous results. If the flanges are ignored and the beam is considered as a rectangular section, the value of span to effective depth ratio thus obtained (percentage of reinforcement being based on the area $b_w d$) should always be on the safe side.

2.5.5 In the case of two way slabs supported on all four sides, the shorter span should be considered for the purpose of calculating the span to effective depth ratio (*see* Note 1 below *23.1* of the Code).

2.5.6 In the case of flat slabs the longer span should be considered (*30.2.1* of the Code). When drop panels conforming to *30.2.2* of the Code are not provided, the values of span to effective depth ratio obtained from the Charts should be multiplied by 0·9.

Example 4 Control of Deflection

Check whether the depth of the member in the following cases is adequate for controlling deflection:

a) Beam of Example 1, as a simply supported beam over a span of 7·5 m

b) Beam of Example 3, as a cantilever beam over a span of 4·0 m

c) Slab of Example 2, as a continuous slab spanning in two directions the shorter and longer spans being, 2·5 m and 3·5 m respectively. The moment given in Example 2 corresponds to shorter span.

a) Actual ratio of $\frac{\text{Span}}{\text{Effective depth}}$

$$= \frac{7{\cdot}5}{(56{\cdot}25/100)} = 13{\cdot}33$$

Percentage of tension reinforcement required,

$p_t = 0{\cdot}6$

Referring to *Chart 22,* value of Max $\left(\frac{\text{Span}}{d}\right)$ corresponding to $p_t = 0{\cdot}6$, is 22·2.

Actual ratio of span to effective depth is less than the allowable value. Hence the depth provided is adequate for controlling deflection.

b) Actual ratio of $\frac{\text{Span}}{\text{Effective depth}}$

$$= \left(\frac{4{\cdot}0}{56{\cdot}25/100}\right) = 7{\cdot}11$$

Percentage of tensile reinforcement, $p_t = 1{\cdot}117$

Referring to *Chart 22,*

Max value of $\left(\frac{\text{Span}}{d}\right) = 21{\cdot}0$

For cantilevers, values read from the Chart are to be multiplied by 0·35.

$$\left.\begin{array}{l}\therefore \text{Max value of}\\ l/d \text{ for}\\ \text{cantilever}\end{array}\right\} = 21{\cdot}0 \times 0{\cdot}35 = 7{\cdot}35$$

∴ The section is satisfactory for control of deflection.

c) Actual ratio of $\frac{\text{Span}}{\text{Effective depth}}$

$$= \frac{2{\cdot}5}{0{\cdot}08} = 31{\cdot}25$$

(for slabs spanning in two directions, the shorter of the two is to be considered)

(i) For $f_y = 415\ \text{N/mm}^2$

$p_t = 0{\cdot}475$

Referring to *Chart 22,*

$$\text{Max}\left(\frac{\text{Span}}{d}\right) = 23{\cdot}6$$

For continuous slabs the factor obtained from the Chart should be multiplied by 1·3.

$$\therefore \text{Max}\ \frac{\text{Span}}{d}\ \text{for continuous slab}$$

$$= 23{\cdot}6 \times 1{\cdot}3 = 30{\cdot}68$$

Actual ratio of span to effective depth is slightly greater than the allowable. Therefore the section may be slightly modified or actual deflection calculations may be made to ascertain whether it is within permissible limits.

(ii) For $f_y = 250\ \text{N/mm}^2$

$p_t = 0{\cdot}78$

Referring to *Chart 21,*

$$\text{Max}\left(\frac{\text{Span}}{d}\right) = 31{\cdot}3$$

∴ For continuous slab,

$$\text{Max}\ \frac{\text{Span}}{d} = 31{\cdot}3 \times 1{\cdot}3 = 40{\cdot}69$$

Actual ratio of span to effective depth is less than the allowable value. Hence the section provided is adequate for controlling deflection.

Chart 1 FLEXURE — Singly Reinforced Section

Moment of Resistance kN.m per Metre Width

$f_y = 250$ N/mm^2 $f_{ck} = 15$ N/mm^2

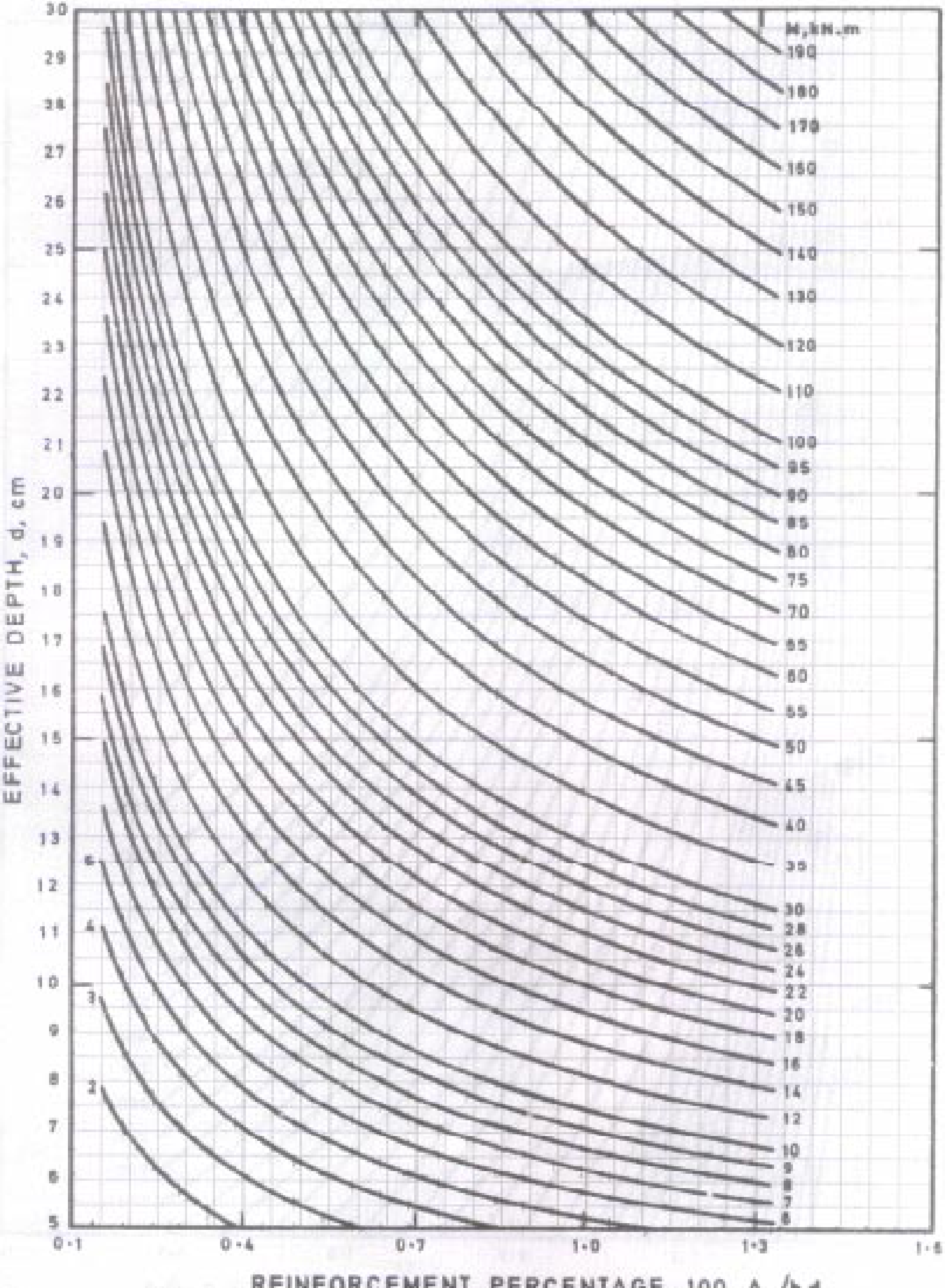

Chart 2 FLEXURE — Singly Reinforced Section

Moment of Resistance kN.m per Metre Width

$f_y = 250\ N/mm^2$ $f_{ck} = 15\ N/mm^2$

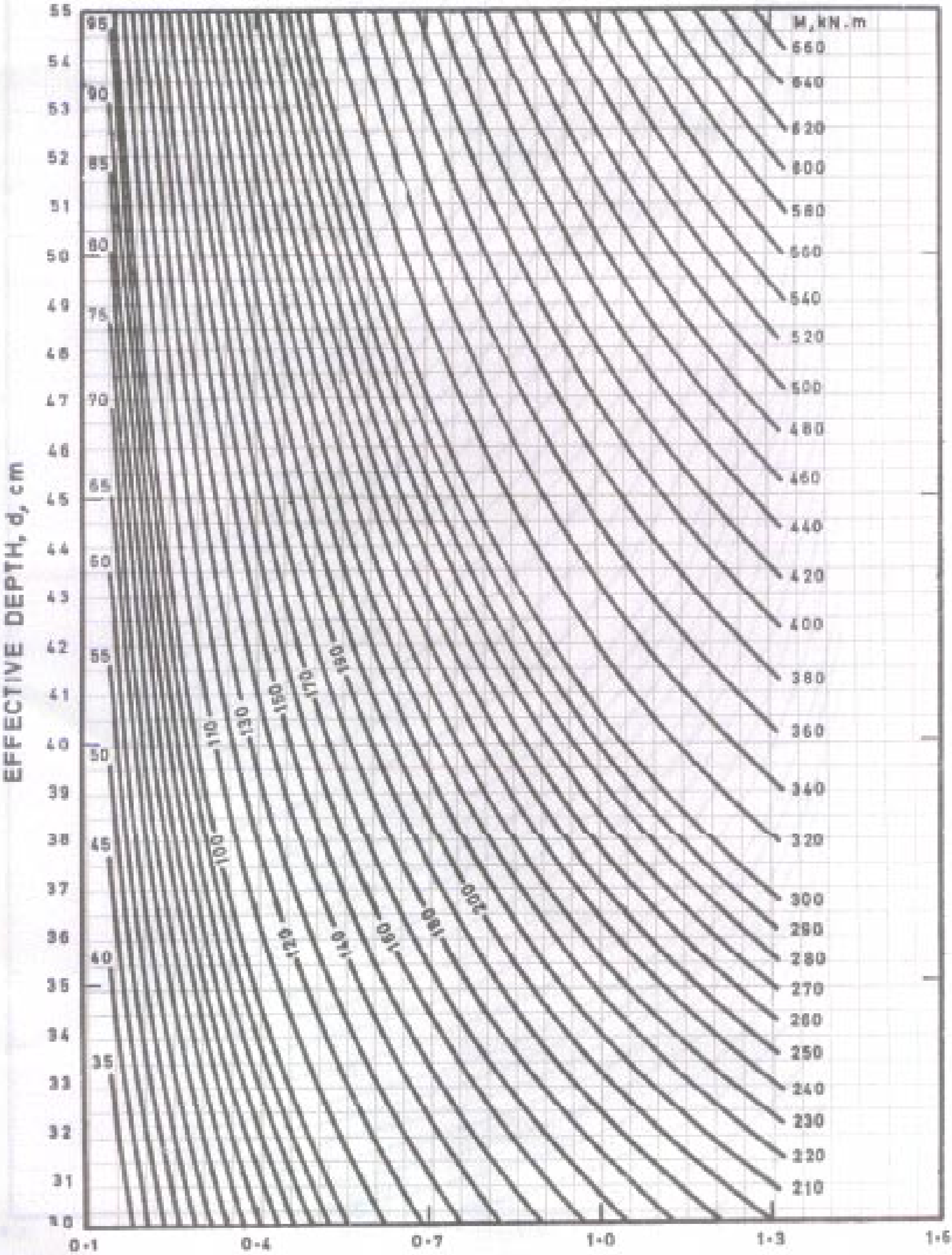

Chart 3 FLEXURE — Singly Reinforced Section

Moment of Resistance kN.m per Metre Width

f_y = 250 N/mm^2 f_{ck} = 15 N/mm^2

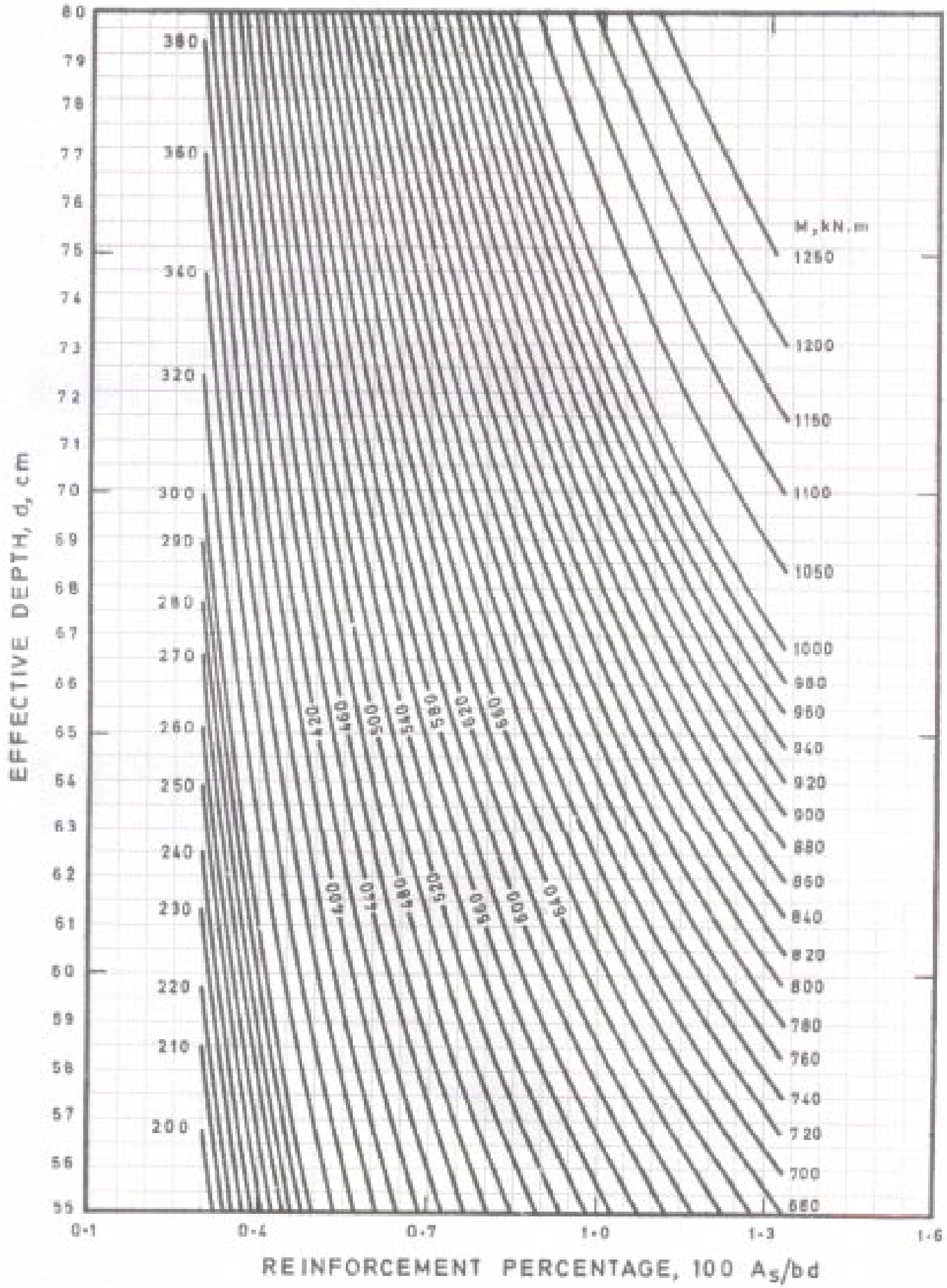

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Chart 4 FLEXURE — Singly Reinforced Section

Moment of Resistance kN.m per Metre Width

$f_y = 415\ N/mm^2$ $f_{ck} = 15\ N/mm^2$

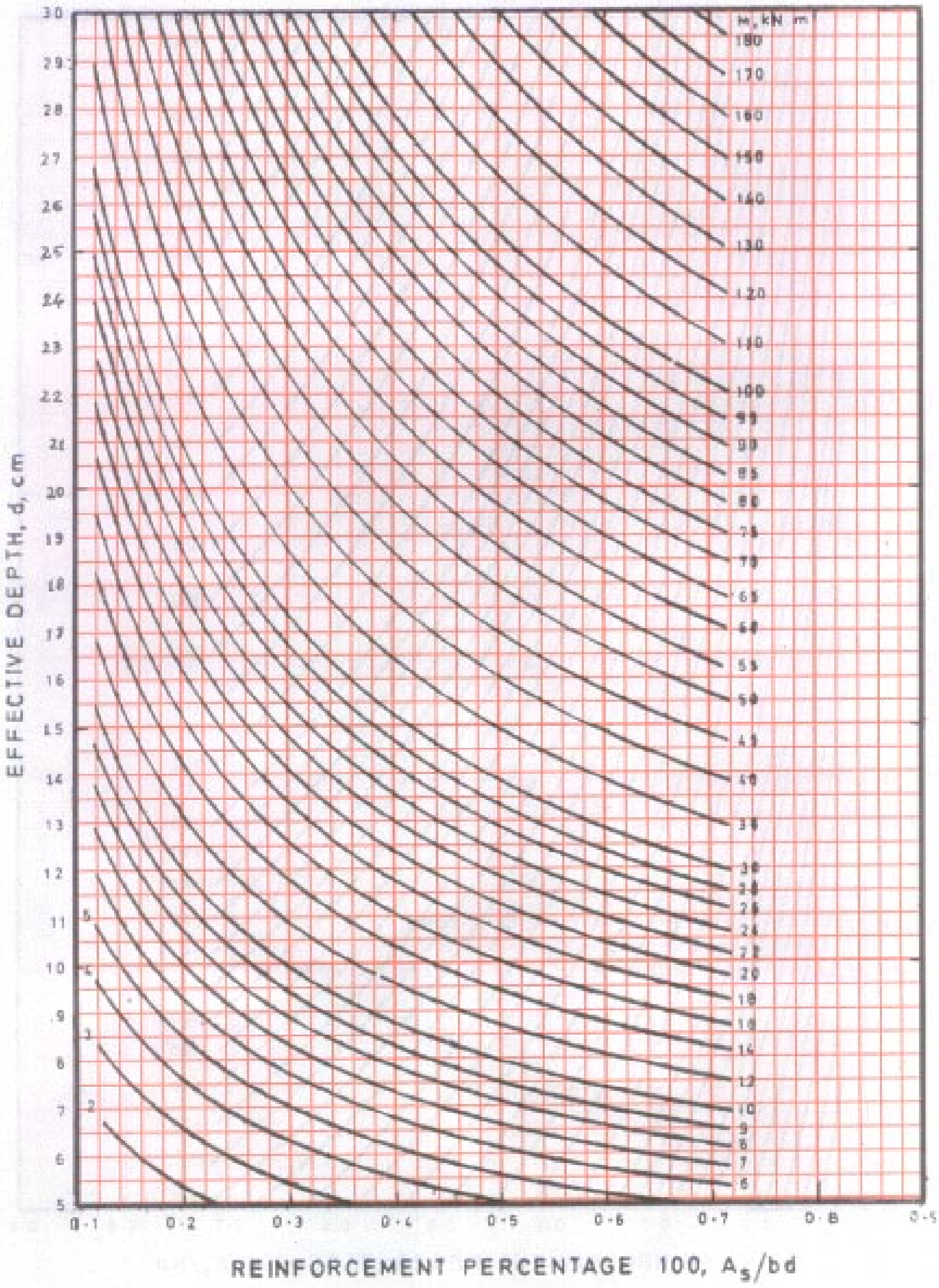

Chart 5 FLEXURE — Singly Reinforced Section

Moment of Resistance kN.m per Metre Width

f_y=415 N/mm^2 f_{ck}=15 N/mm^2

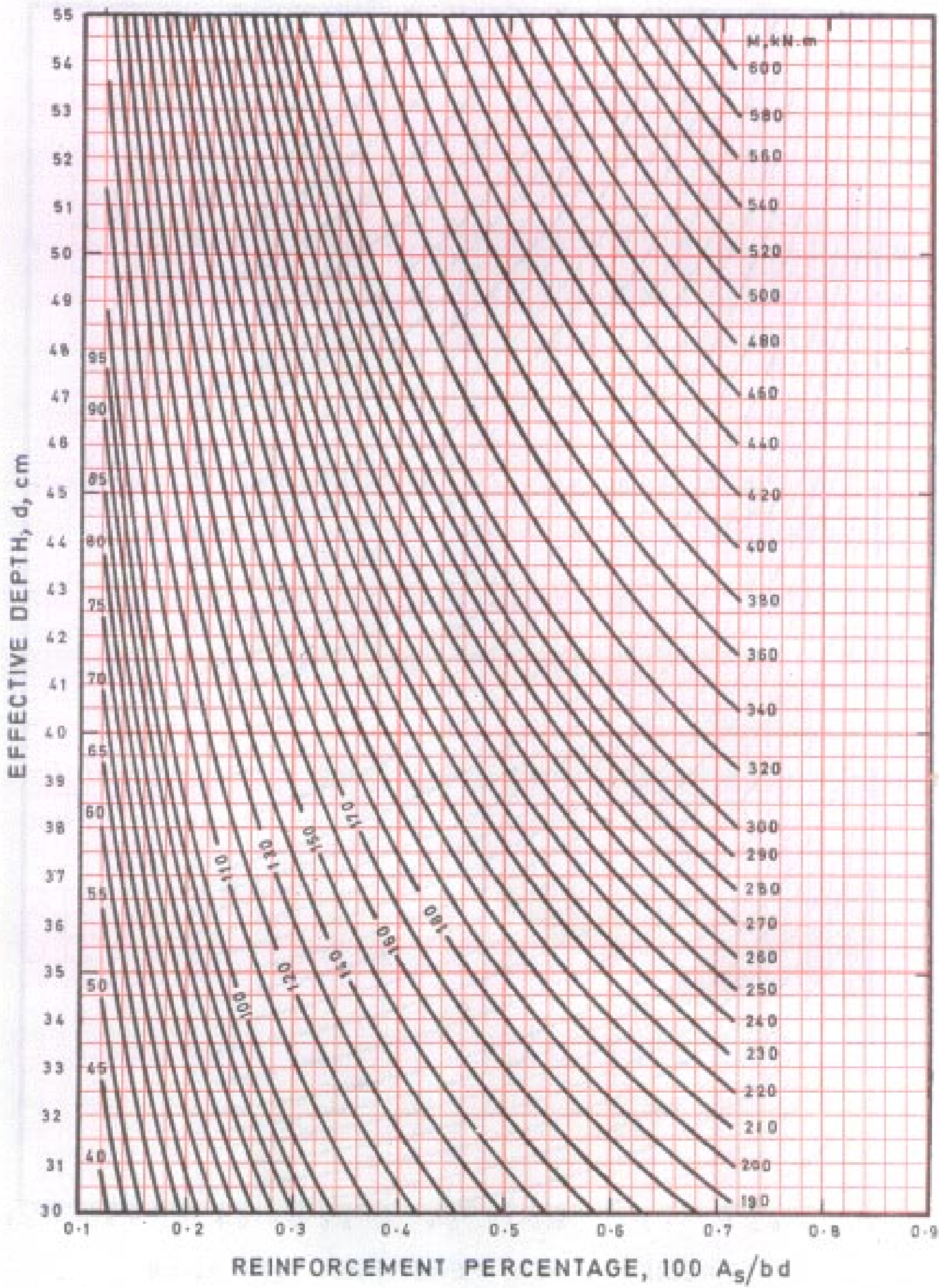

Chart 6 FLEXURE — Singly Reinforced Section

Moment of Resistance kN.m per Metre Width

$f_y = 415\ N/mm^2$ $f_{ck} = 15\ N/mm^2$

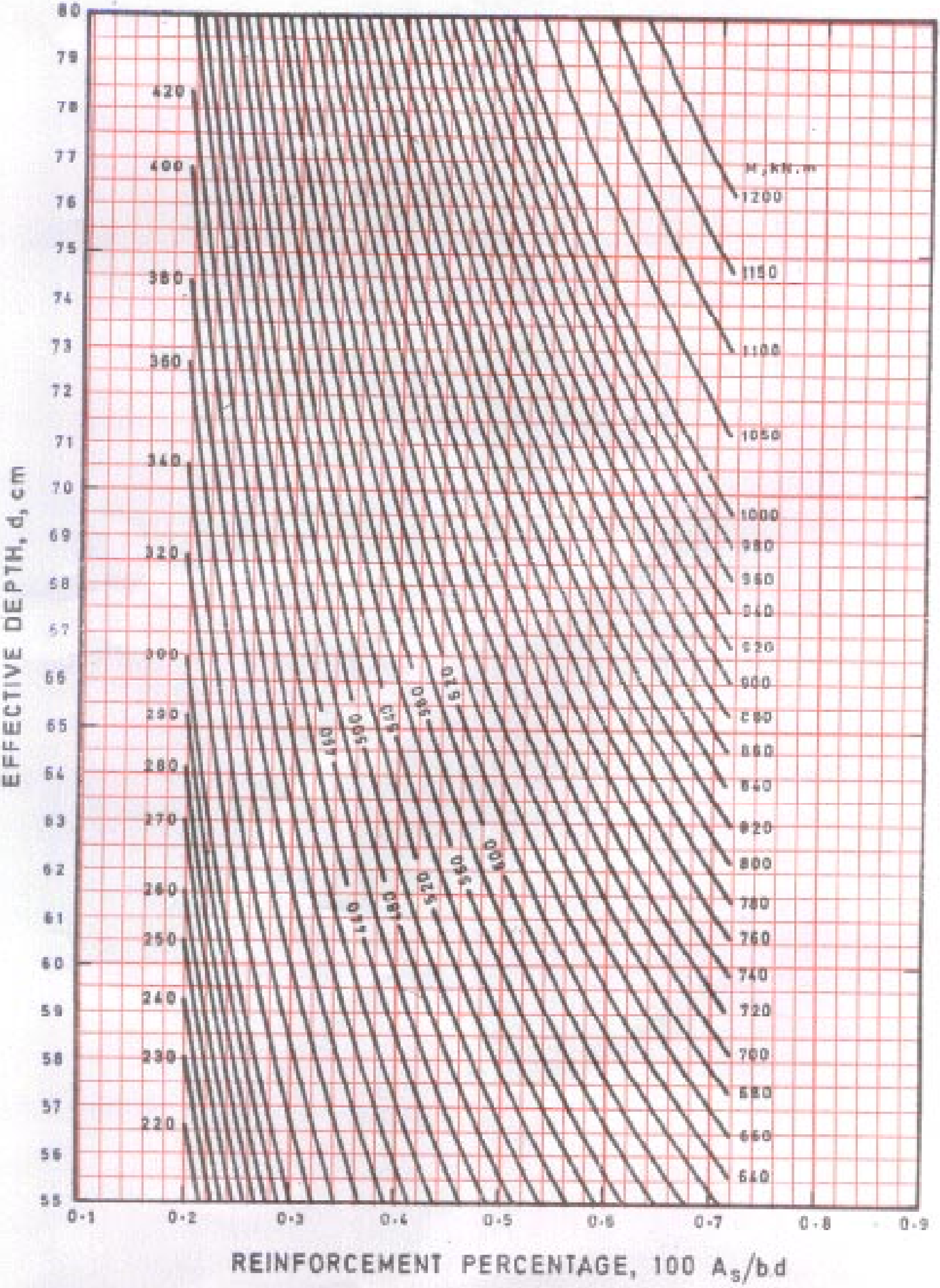

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Chart 7 FLEXURE — Singly Reinforced Section

Moment of Resistance kN.m per Metre Width

$f_y = 500\ N/mm^2$ $f_{ck} = 15\ N/mm^2$

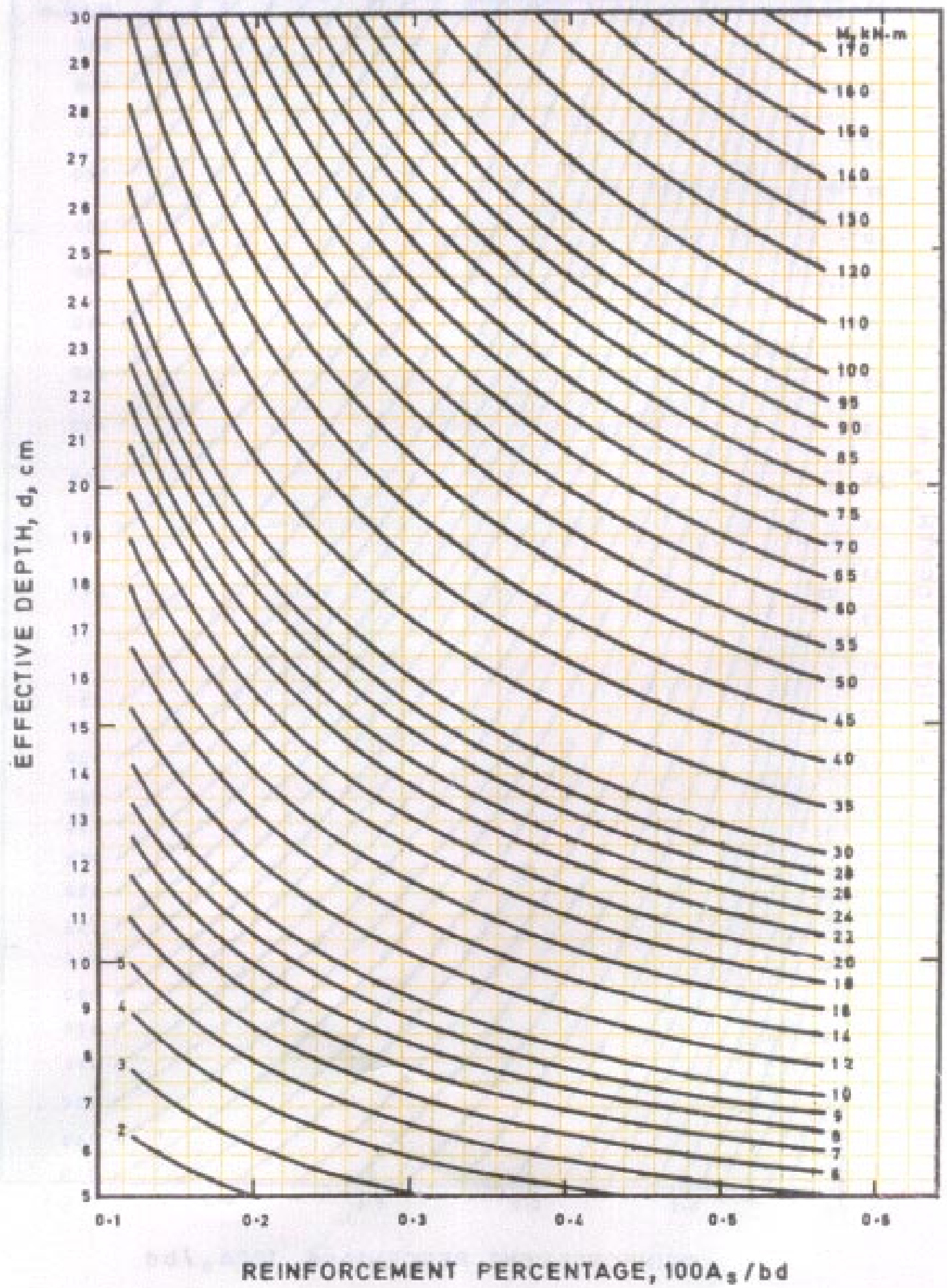


Chart 8 FLEXURE — Singly Reinforced Section

Moment of Resistance kN.m per Metre Width

$f_y = 500$ N/mm^2 $f_{ck} = 15$ N/mm^2

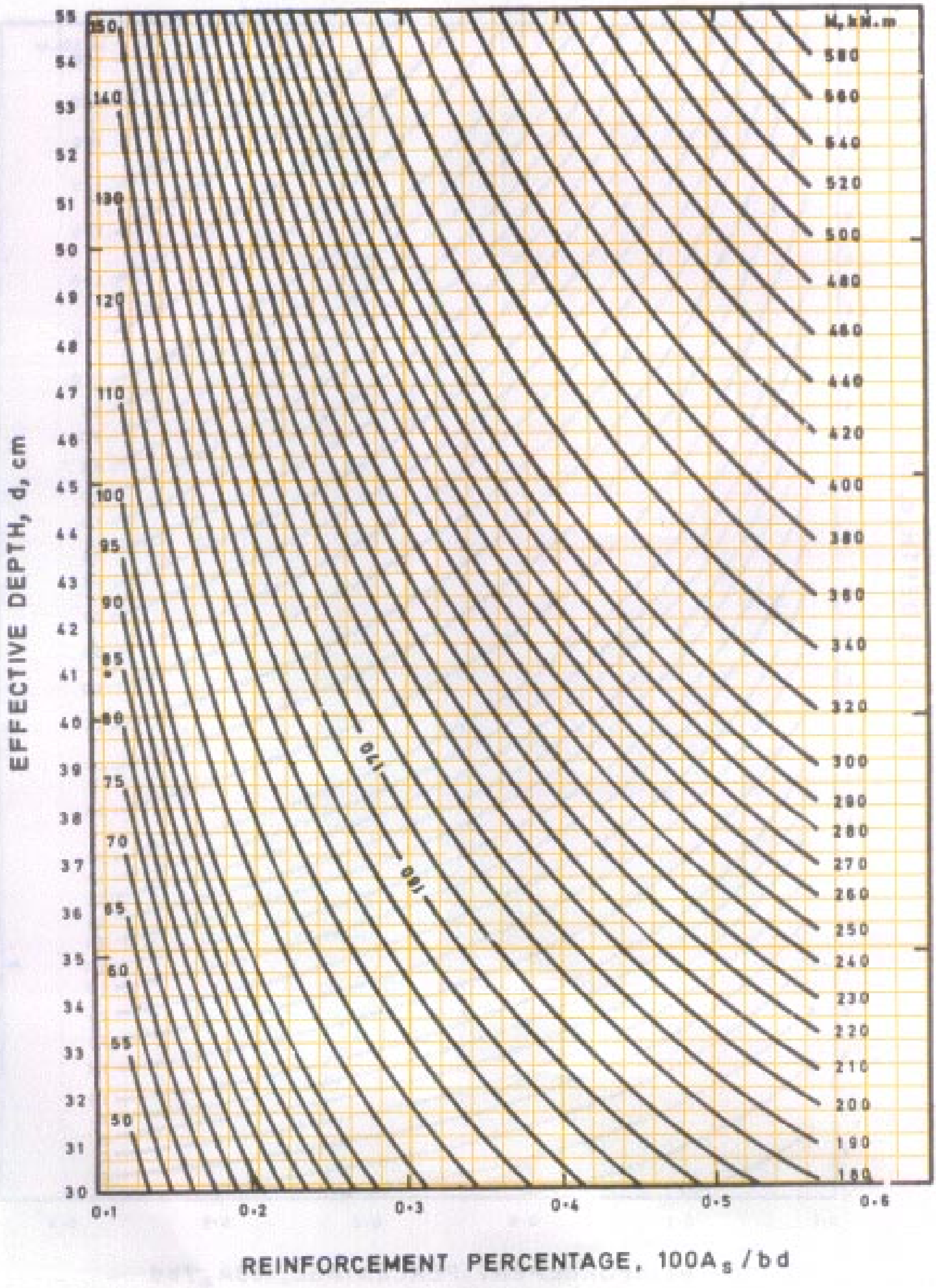
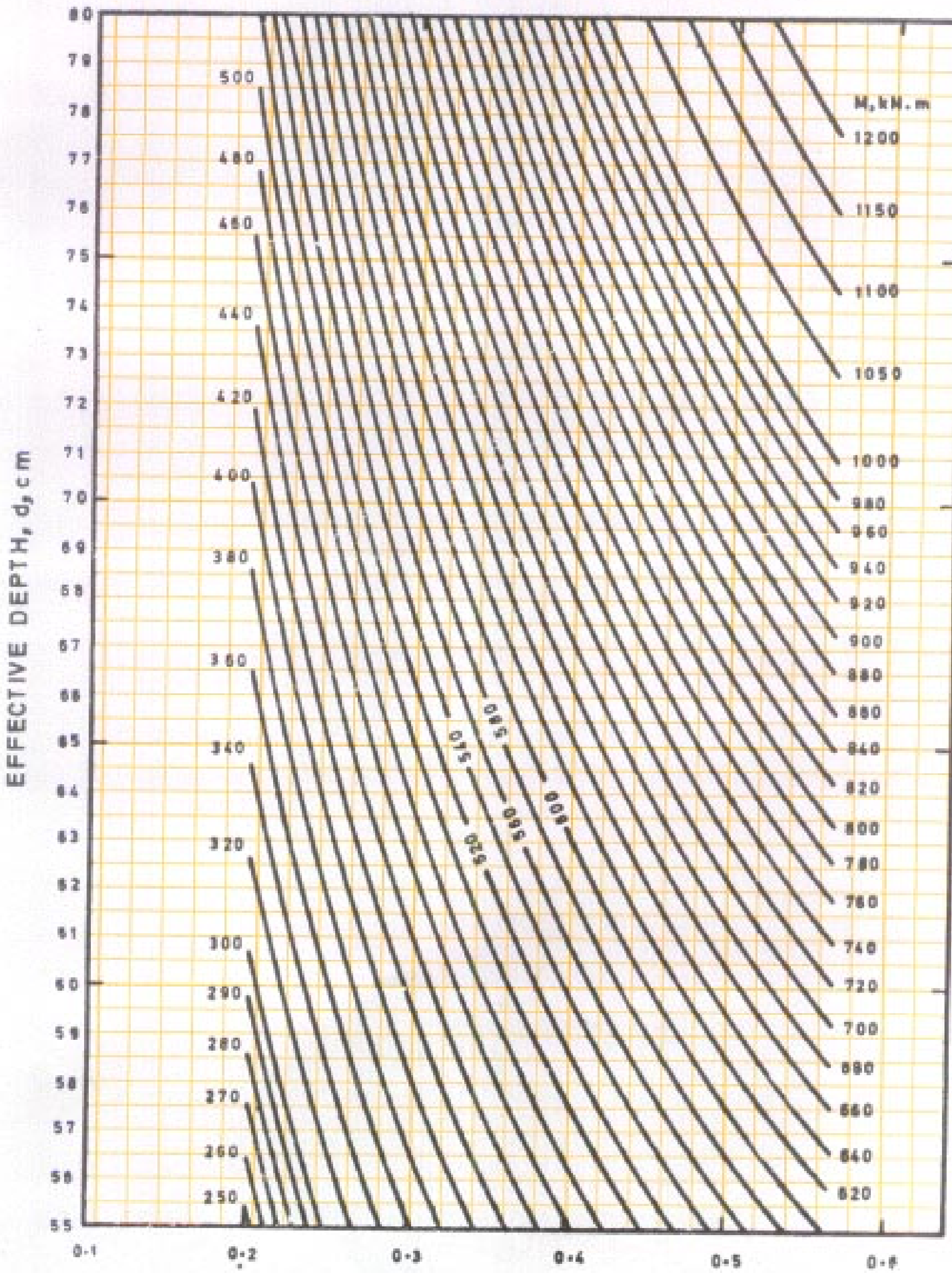


Chart 9 FLEXURE — Singly Reinforced Section

Moment of Resistance kN.m per Metre Width

$f_y = 500\ N/mm^2$ $f_{ck} = 15\ N/mm^2$



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f_y 250

f_{ck} 20

d 5–30

Chart 10 FLEXURE — Singly Reinforced Section

Moment of Resistance kN.m per Metre Width

$f_y = 250\ N/mm^2$ $f_{ck} = 20\ N/mm^2$

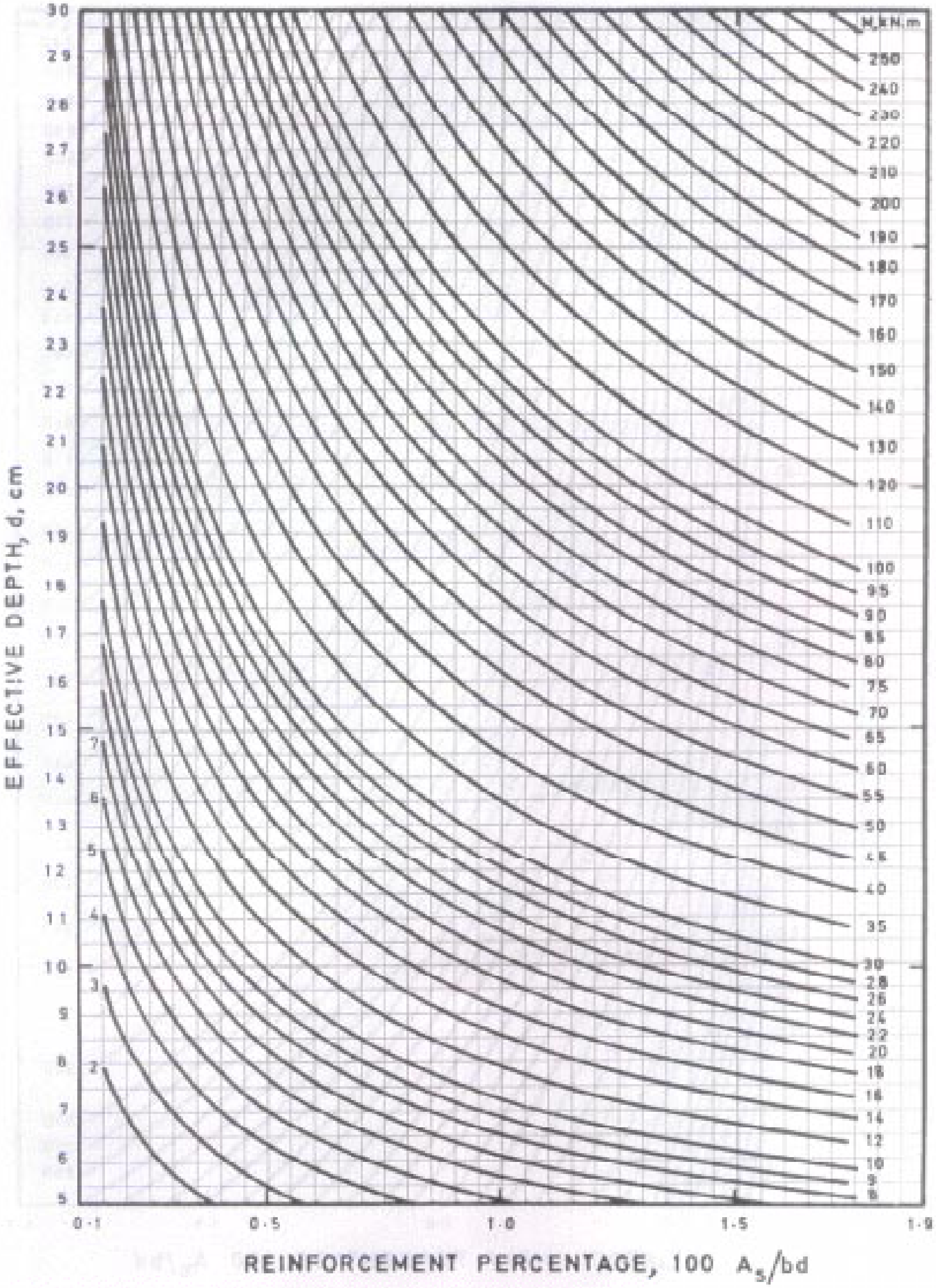
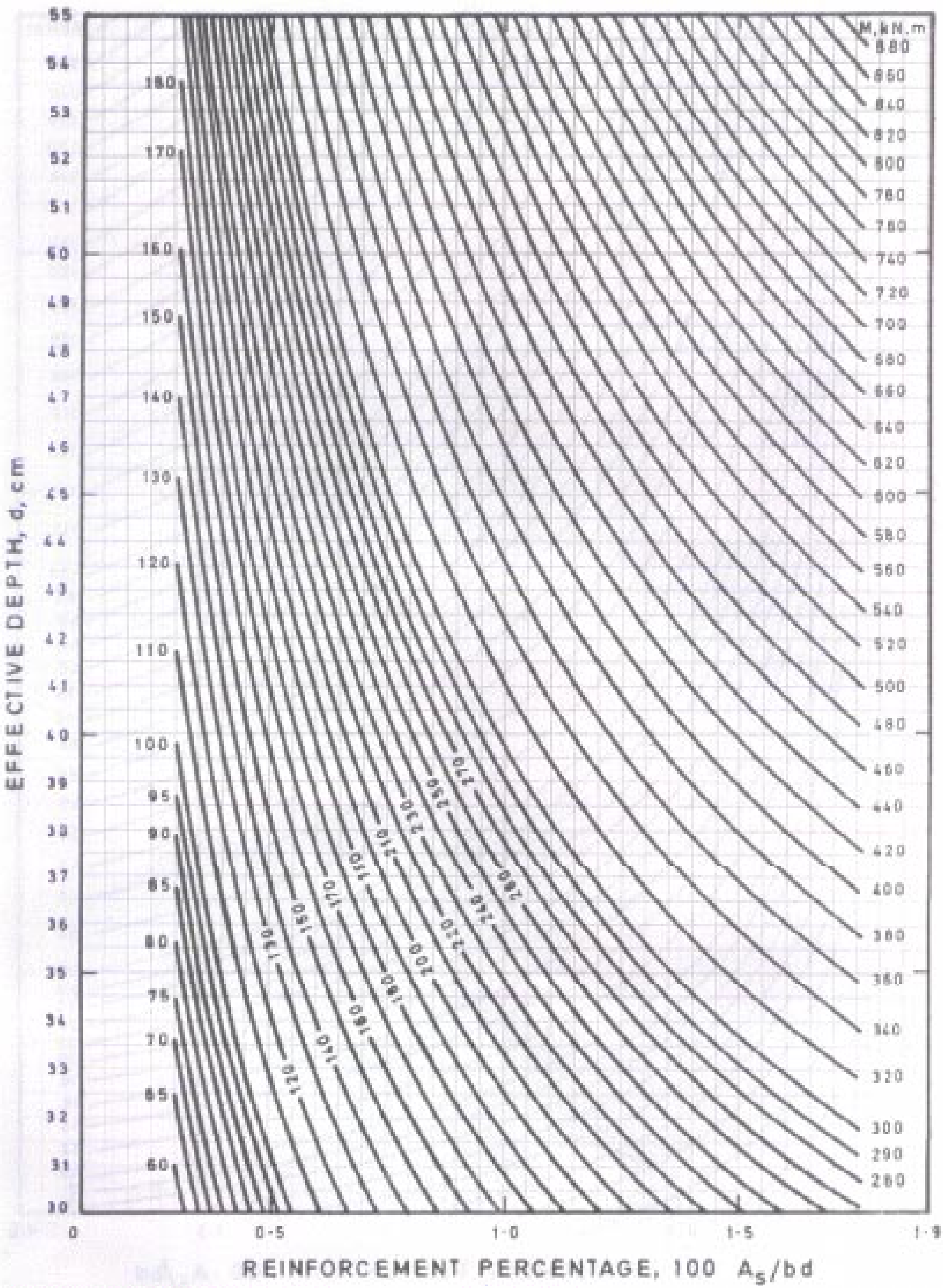


Chart 11 FLEXURE — Singly Reinforced Section

Moment of Resistance kN.m per Metre Width

$f_y = 250$ N/mm^2 $f_{ck} = 20$ N/mm^2

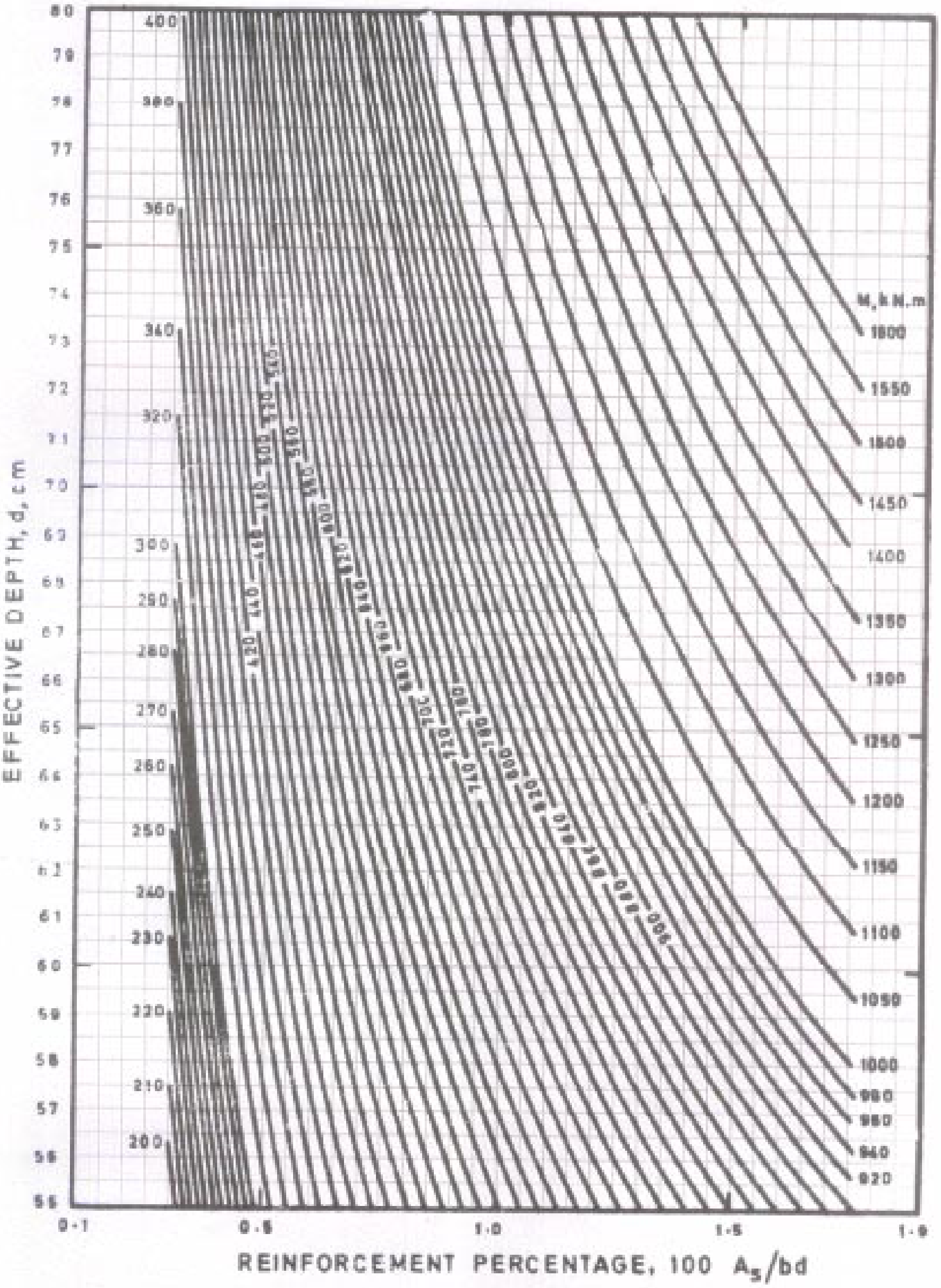


f_y
250
f_{ck}
20
d
55-80

Chart 12 FLEXURE — Singly Reinforced Section

Moment of Resistance kN.m per Metre Width

$f_y = 250\ N/mm^2$ $f_{ck} = 20\ N/mm^2$



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f_y
415
f_{ck}
20
d
5-30

Chart 13 FLEXURE — Singly Reinforced Section

Moment of Resistance kN.m per Metre Width

$f_y = 415$ N/mm^2 $f_{ck} = 20$ N/mm^2

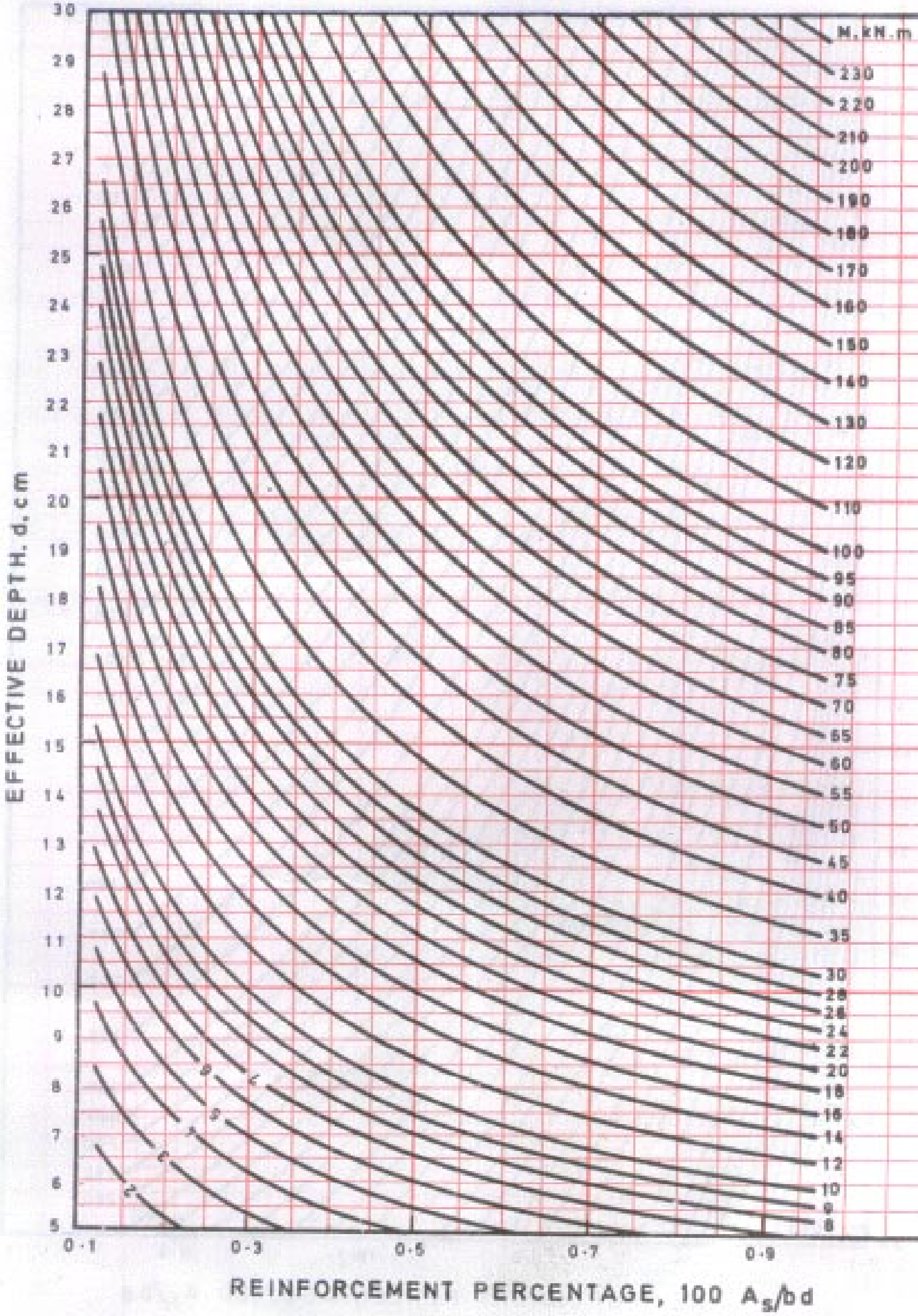

Chart 14 FLEXURE — Singly Reinforced Section

Moment of Resistance kN.m per Metre Width

$f_y = 415$ N/mm^2 $f_{ck} = 20$ N/mm^2

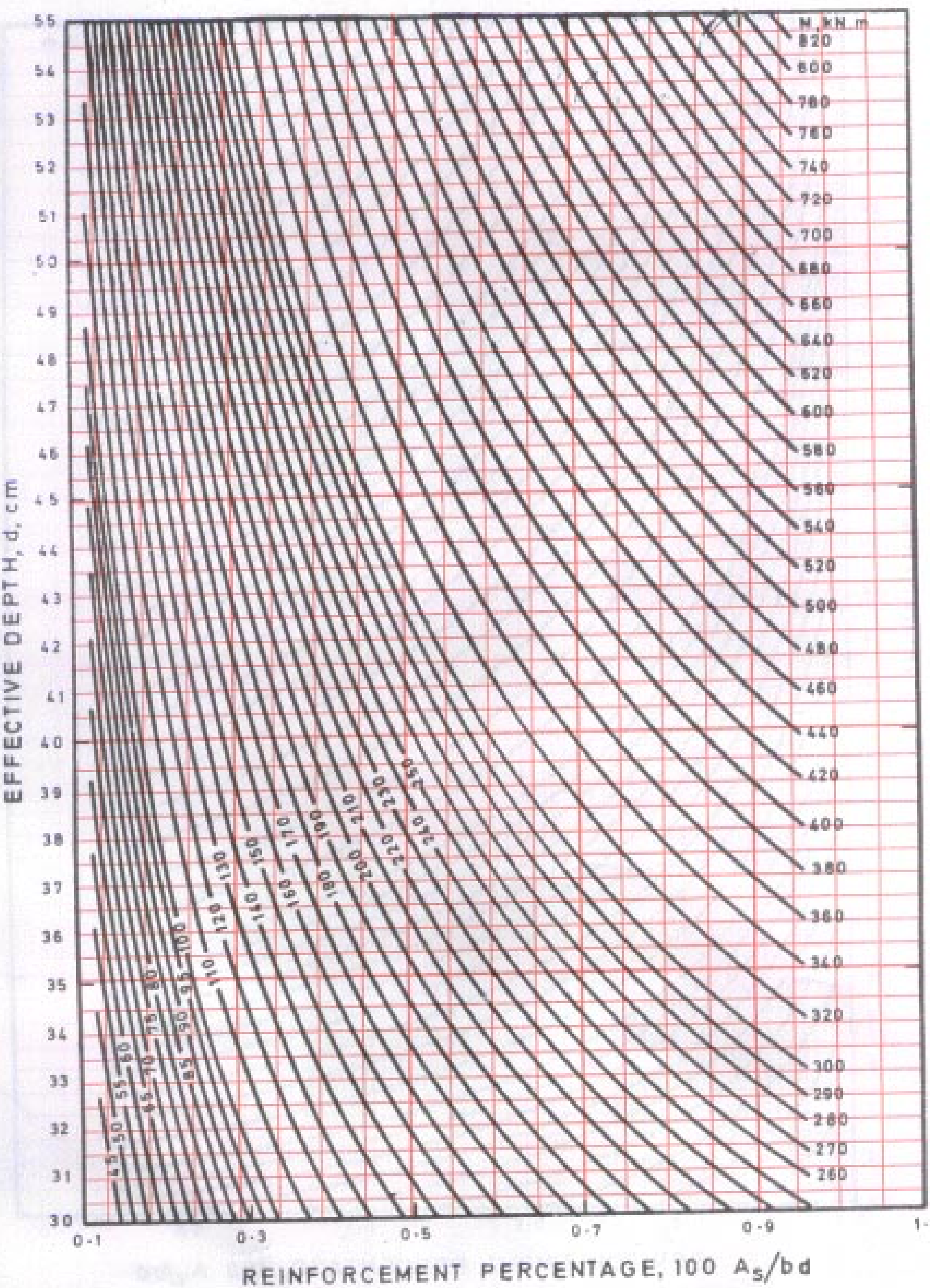

Chart 15 FLEXURE — Singly Reinforced Section

Moment of Resistance kN.m per Metre Width

f_y = 415 N/mm^2 f_{ck} = 20 N/mm^2

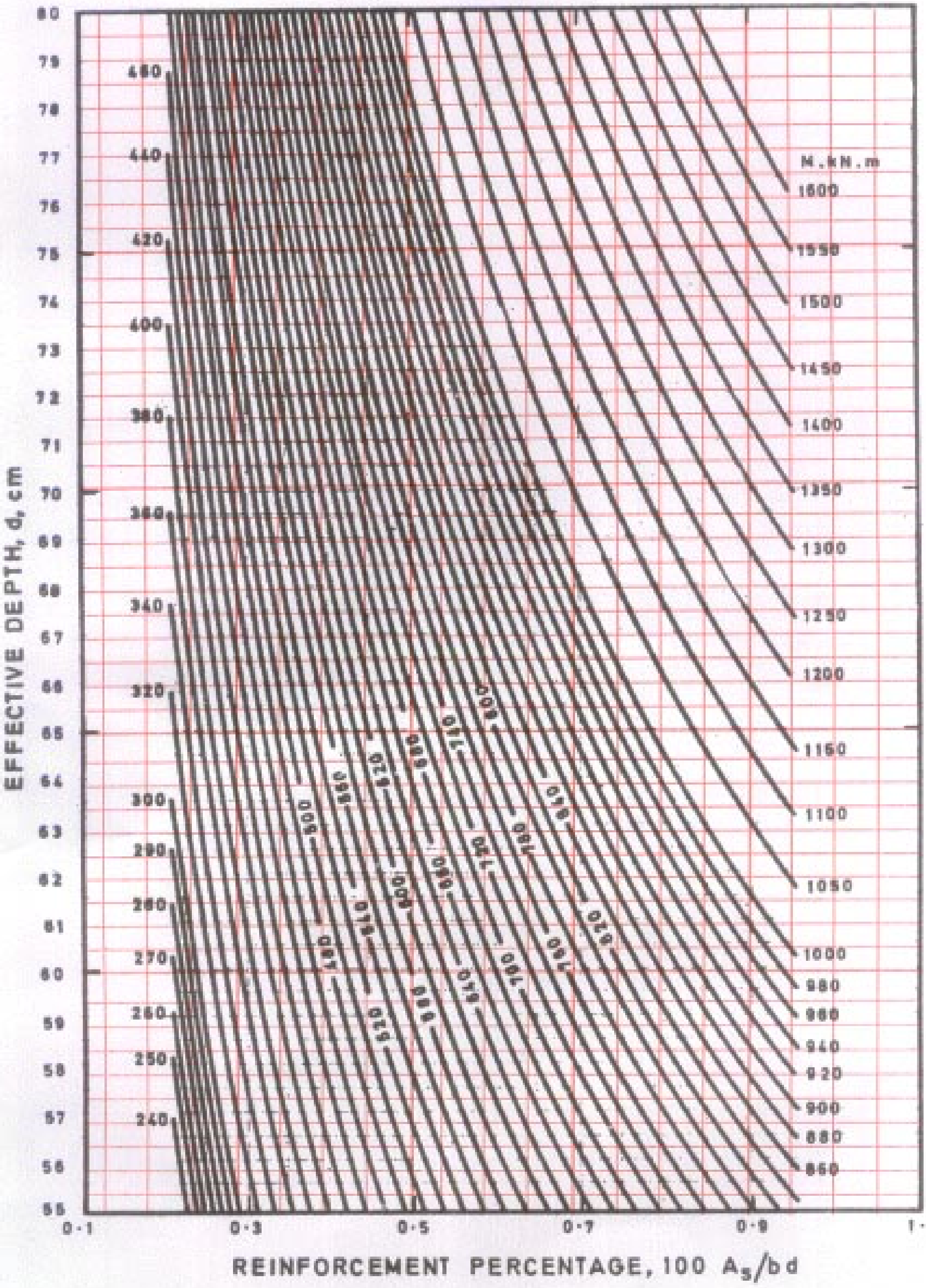

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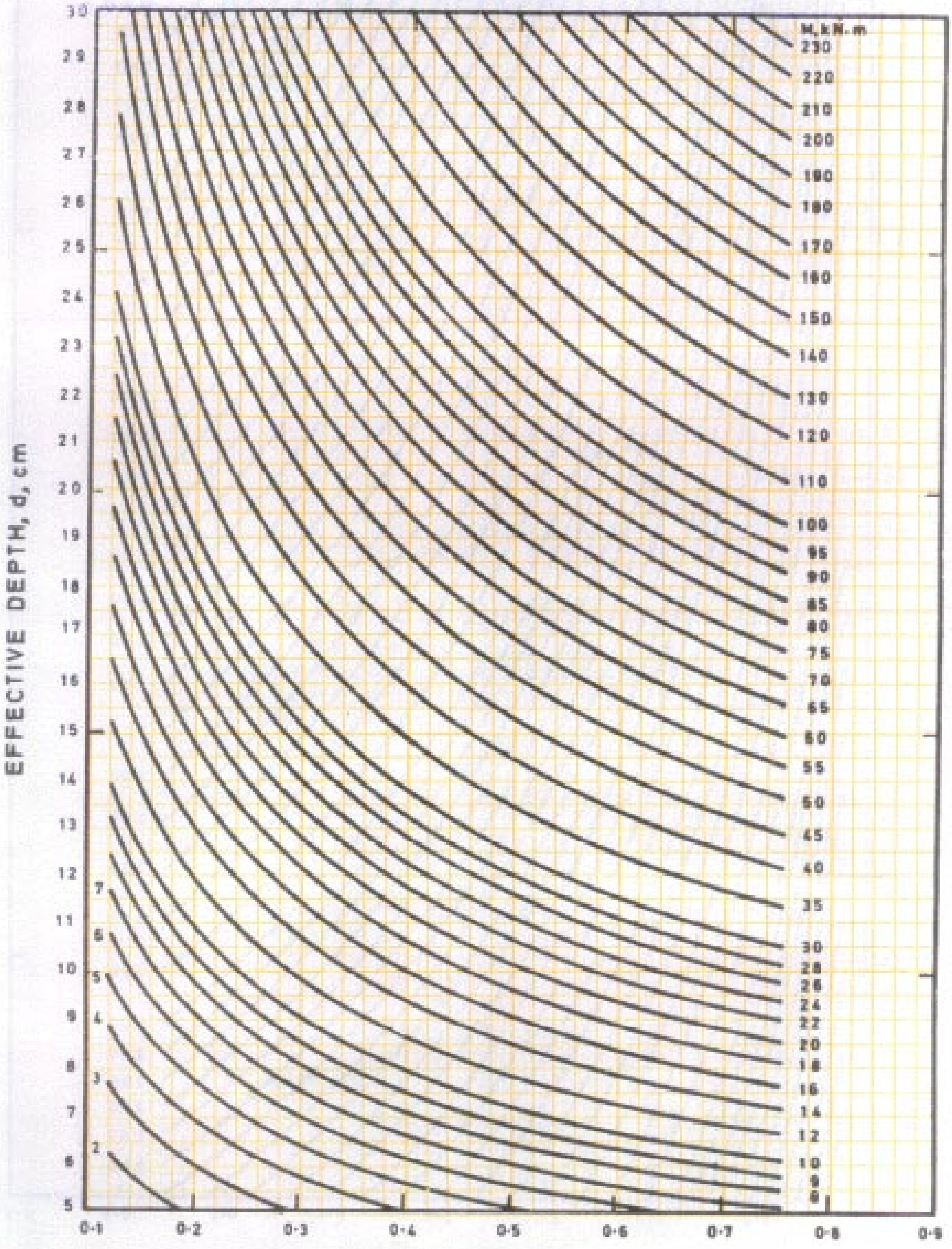
Chart 16 FLEXURE — Singly Reinforced Section

Moment of Resistance kN.m per Metre Width

$f_y = 500$ N/mm^2 $\qquad$ $f_{ck} = 20$ N/mm^2



f_y 500

f_{ck} 20

d 30–55

Chart 17 FLEXURE — Singly Reinforced Section

Moment of Resistance kN.m per Metre Width

$f_y = 500\ N/mm^2$ $f_{ck} = 20\ N/mm^2$

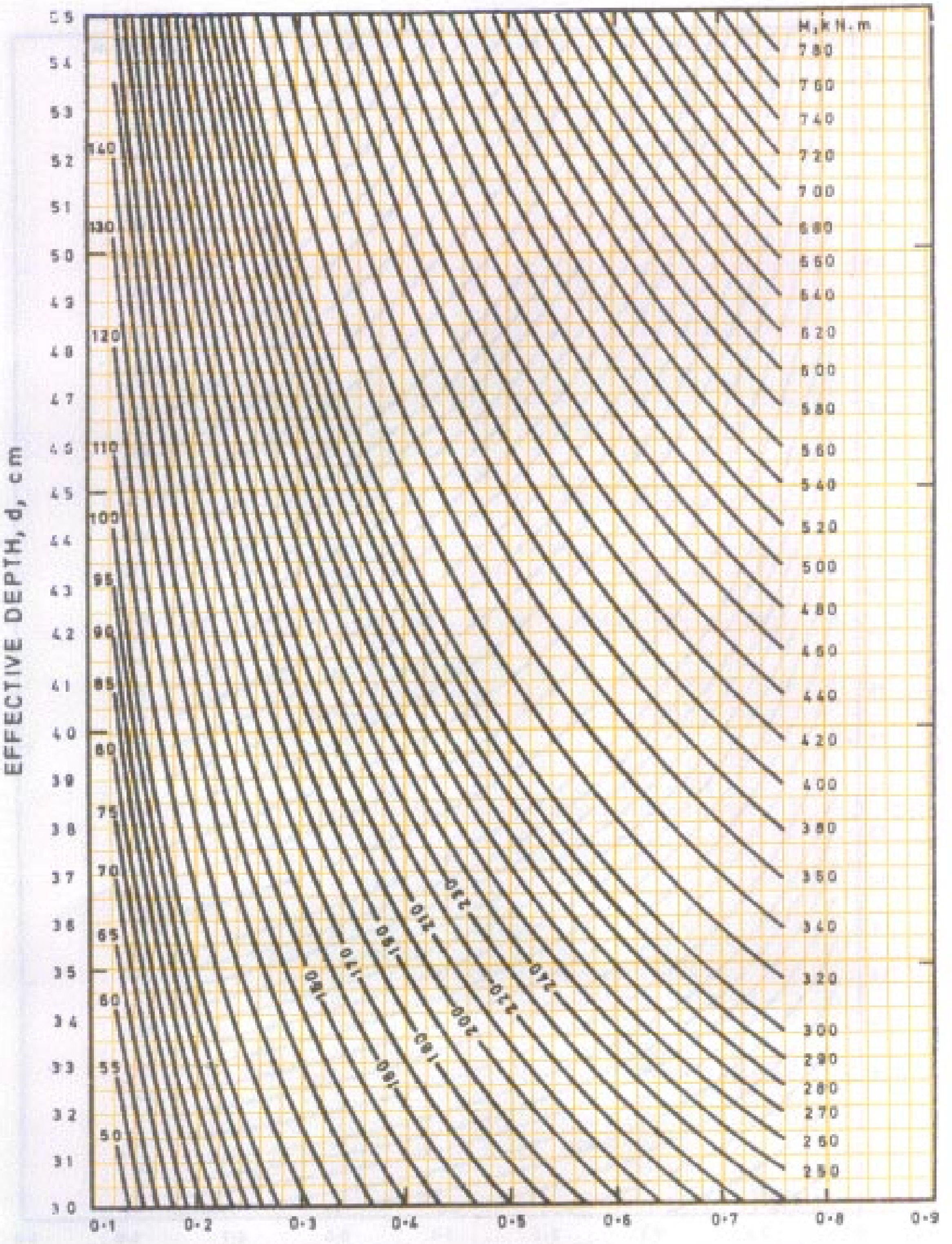


Chart 18 FLEXURE — Singly Reinforced Section

Moment of Resistance kN.m per Metre Width

$f_y = 500\ N/mm^2$ $f_{ck} = 20\ N/mm^2$

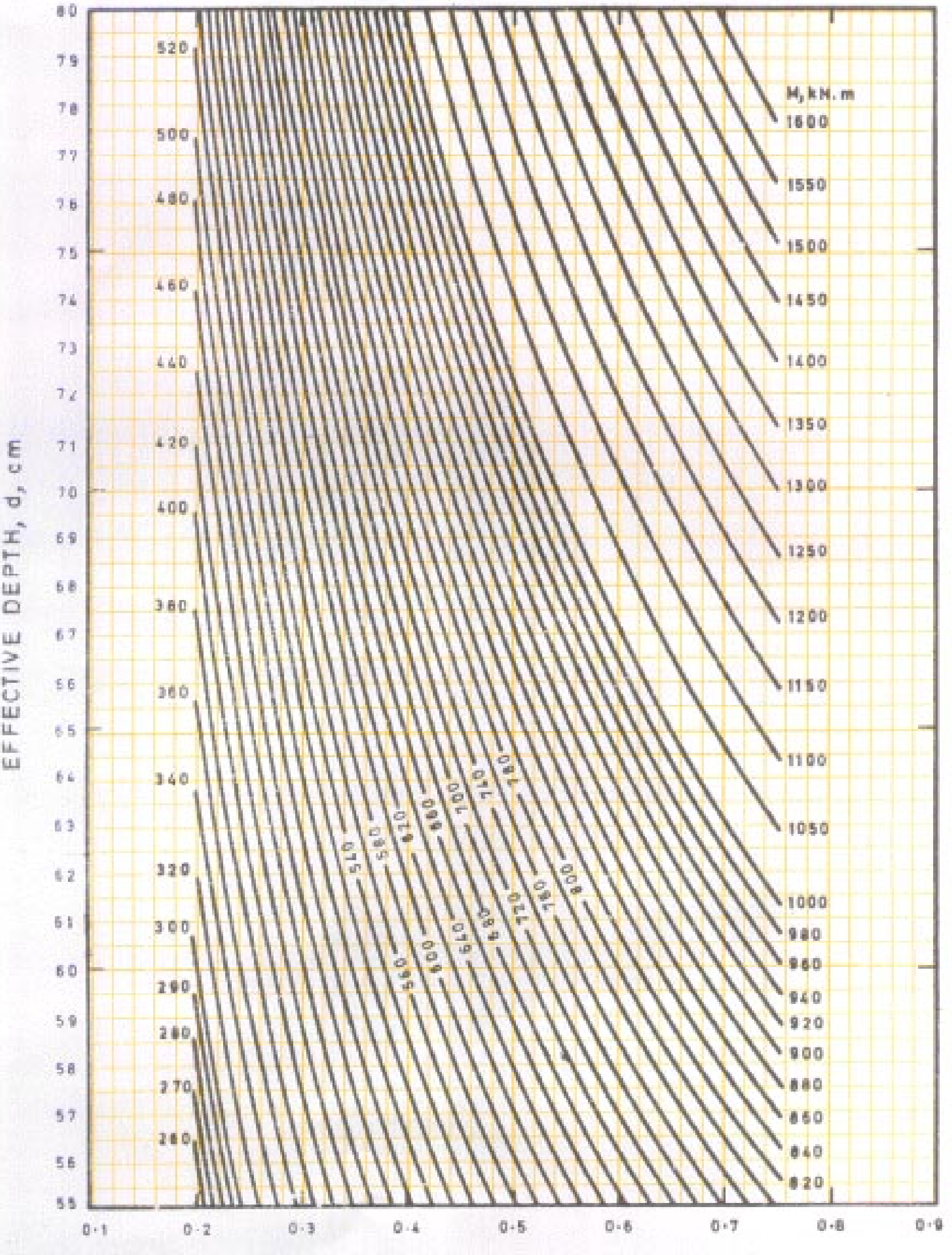


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Chart 19 FLEXURE — Doubly Reinforced Section

$f_y = 250$ N/mm²

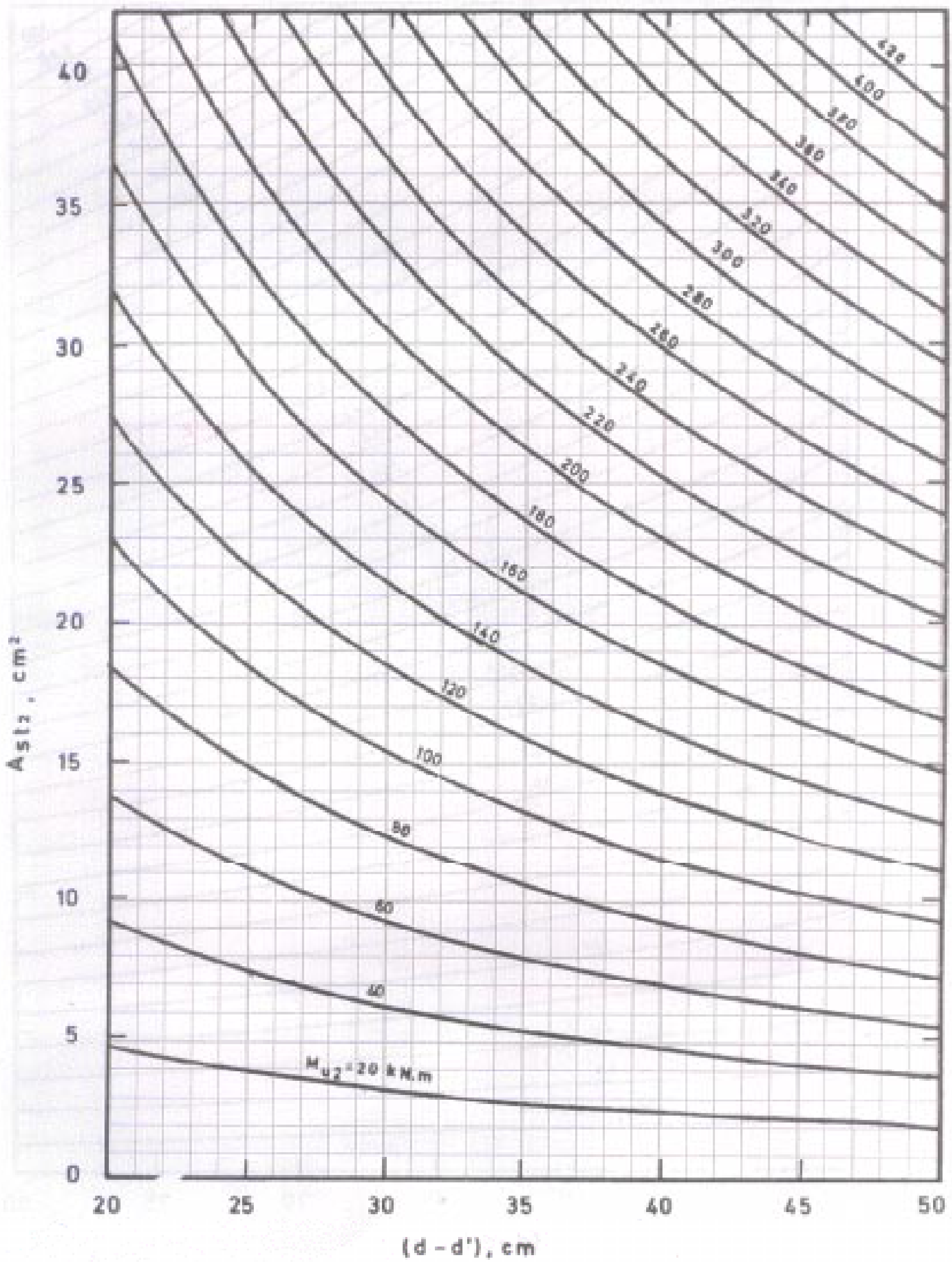


Chart 20 FLEXURE — Doubly Reinforced Section

$f_y = 250$ N/mm²

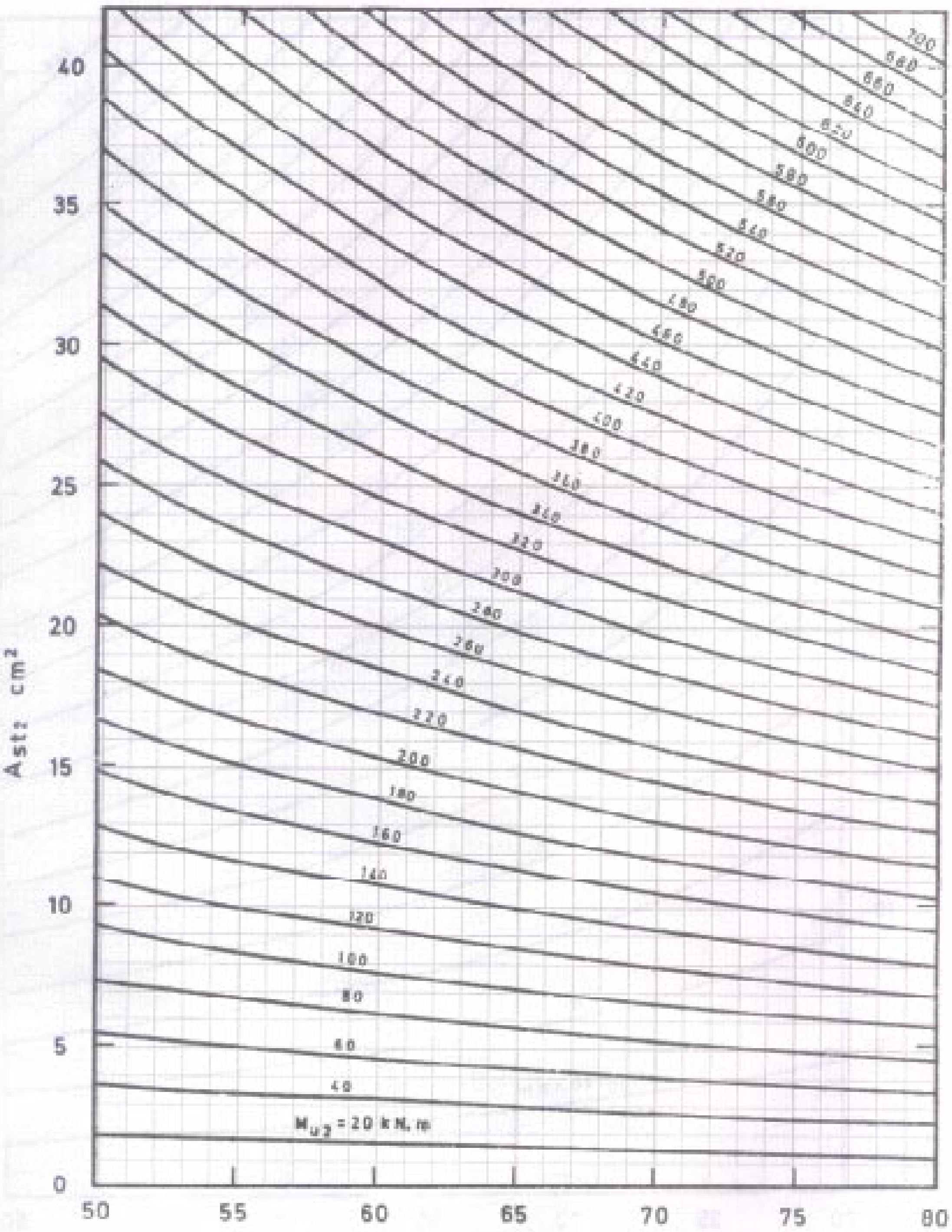
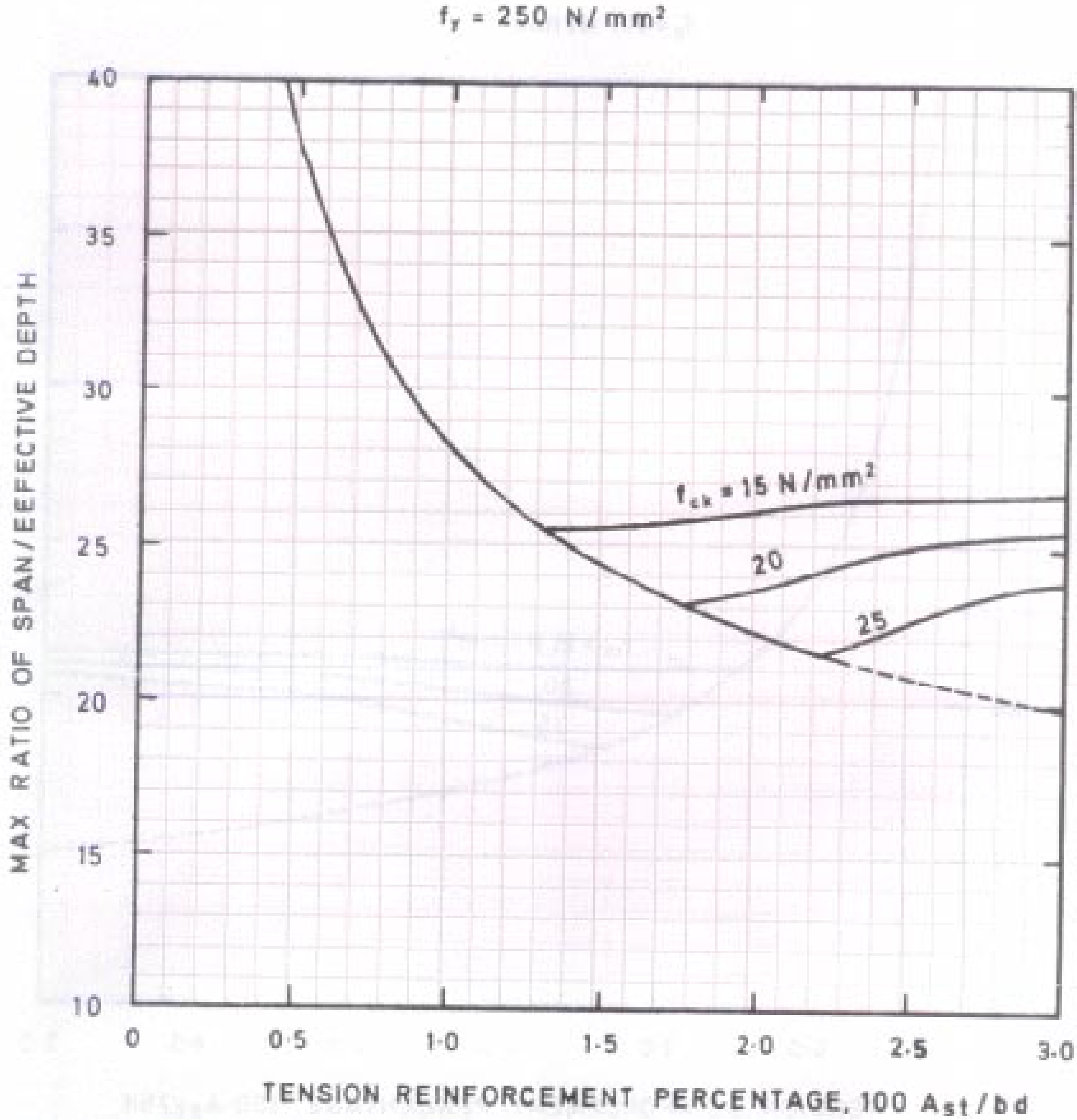


Chart 21 CONTROL OF DEFLECTION



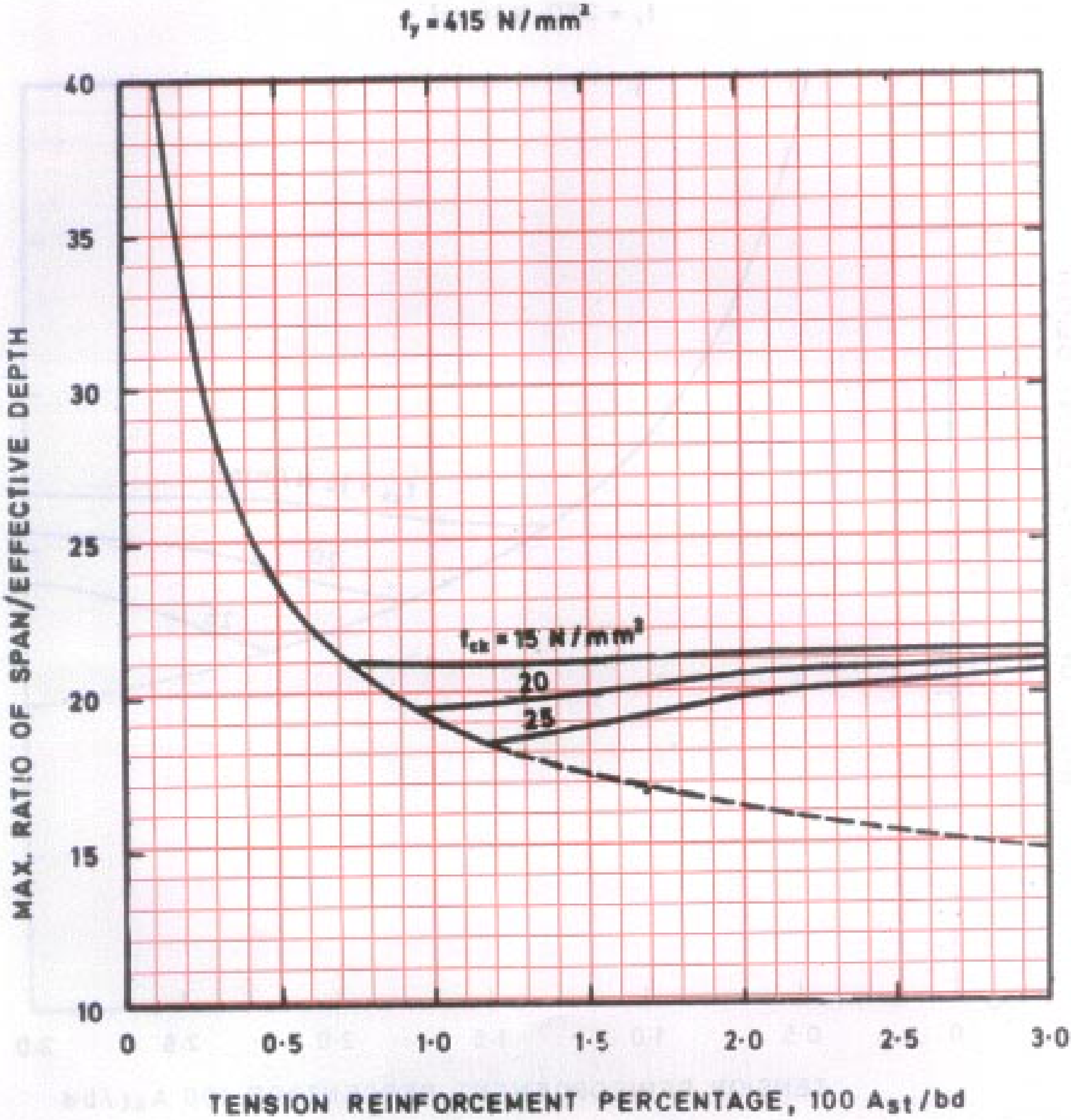
Values for span/effective depth ratio given in this chart are for simply supported spans up to 10 m. For spans over 10 m, multiply the values by 10/span in metres.

For continuous beam or slab, multiply the value for simply supported condition by 1.3.

For cantilevers up to 10 m, multiply the value from the chart by 0.35.

For cantilevers over 10 m, this chart is not valid.

Chart 22 CONTROL OF DEFLECTION



Values of Span/effective depth ratio given in this chart are for simply supported spans up to 10 m, For spans over 10 m, multiply the values by 10/span in metres.

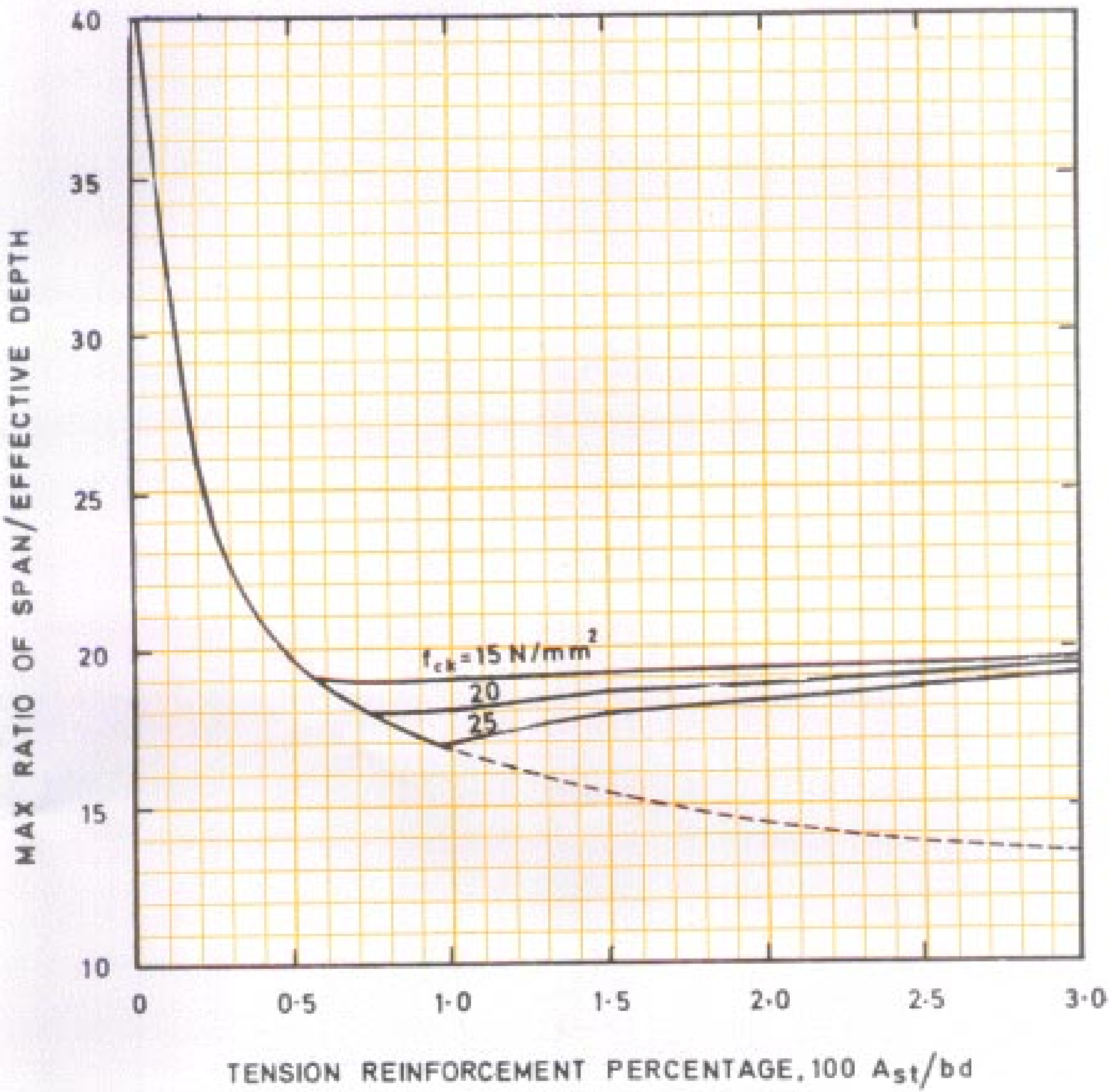
For continuous beam or slab, multiply the value for simply supported condition by 1.3.

For cantilevers up to 10 m, multiply the value from the chart by 0.35.

For cantilevers over 10 m, this chart is not valid.

Chart 23 CONTROL OF DEFLECTION

$f_y = 500$ N/mm^2



Values of span/effective depth ratio given in this chart are for simply supported spans up to 10 m. For spans over 10 m, multiply the values by 10/span in metres.

For continuous beam or slab, multiply the value for simply supported condition by 1.3.

For cantilevers up to 10 m, multiply the value from the chart by 0.35.

For cantilevers over 10 m, this chart is not valid.

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TABLE 1 FLEXURE — REINFORCEMENT PERCENTAGE, p_t FOR SINGLY REINFORCED SECTIONS

f_{ck} = 15 Nmm^2

M_u/bd^2, N/mm^2	f_y, N/mm^2 240	250	415	480	500	M_u/bd^2, N/mm	f_y, N/mm^2 240	250	415	480	500
0·30	0·147	0·141	0·085	0·074	0·071	1·50	0·829	0·796	0·480	0·415	0·398
0·35	0·172	0·166	0·100	0·086	0·083	1·52	0·842	0·809	0·487	0·421	0·404
0·40	0·198	0·190	0·114	0·099	0·095	1·54	0·856	0·821	0·495	0·428	0·411
0·45	0·224	0·215	0·129	0·112	0·107	1·56	0·869	0·834	0·503	0·434	0·417
0·50	0·250	0·240	0·144	0·125	0·120	1·58	0·882	0·847	0·510	0·441	0·423
0·55	0·276	0·265	0.159	0·138	0·132	1·60	0·896	0·860	0·518	0·448	0·430
0·60	0·302	0·290	0.175	0·151	0·145	1·62	0·909	0·873	0·526	0·455	0·436
0·65	0·329	0·316	0.190	0·164	0·158	1·64	0·923	0·886	0·534	0·461	0·443
0·70	0·356	0·342	0.206	0·178	0·171	1·66	0·936	0·899	0·542	0·468	0·449
0·75	0·383	0·368	0.221	0·191	0·184	1·68	0·950	0·912	0·550	0·475	0·456
0·80	0·410	0·394	0·237	0·205	0·197	1·70	0·964	0·925	0·558	0·482	0·463
0·82	0·421	0·405	0·244	0·211	0·202	1·72	0·978	0·939	0·566	0·489	0·469
0·84	0·433	0·415	0·250	0·216	0·208	1·74	0·992	0·952	0·574	0·496	0·476
0·86	0·444	0·426	0·257	0·222	0·213	1·76	1·006	0·966	0·582	0·503	0·483
0·88	0·455	0·437	0·263	0·227	0·218	1·78	1·020	0·980	0·590	0·510	0·490
0·90	0·466	0·448	0·270	0·233	0·224	1·80	1·035	0·993	0·598	0·517	0·497
0·92	0·477	0·458	0·276	0·239	0·229	1·82	1·049	1·007	0·607	0·525	0·504
0·94	0·489	0·469	0·283	0·244	0·235	1·84	1·064	1·021	0·615	0·532	0·511
0·96	0·500	0·480	0·289	0·250	0·240	1·86	1·078	1·035	0·624	0·539	0·518
0·98	0·512	0·491	0·296	0·256	0·246	1·88	1·093	1·049	0·632	0·546	0·525
1·00	0·523	0·502	0·303	0·262	0·251	1·90	1·108	1·063	0·641	0·554	0·532
1·02	0·535	0·513	0·309	0·267	0·257	1·92	1·123	1·078	0·649	0·561	0·539
1·04	0·546	0·524	0·316	0·273	0·262	1·94	1·138	1·092	0·658	0·569	0·546
1·06	0·558	0·536	0·323	0·279	0·268	1·96	1·153	1·107	0·667	0·576	0·553
1·08	0·570	0·547	0·329	0·285	0·273	1·98	1·168	1·121	0·676	0·584	0·561
1·10	0·581	0·558	0·336	0·291	0·279	2·00	1·184	1·136	0·685	0·592	
1·12	0·593	0·570	0·343	0·297	0·285	2·02	1·199	1·151	0·693		
1·14	0·605	0·581	0·350	0·303	0·290	2·04	1·215	1·166	0·703		
1·16	0·617	0·592	0·357	0·309	0·296	2·06	1·231	1·181	0·712		
1·18	0·629	0·604	0·364	0·315	0·302	2·08	1·247	1·197			
1·20	0·641	0·615	0·371	0·321	0·308	2·10	1·263	1·212			
1·22	0·653	0·627	0·378	0·327	0·314	2·12	1·279	1·228			
1·24	0·665	0·639	0·385	0·333	0·319	2·14	1·295	1·243			
1·26	0·678	0·650	0·392	0·339	0·325	2·16	1·312	1·259			
1·28	0·690	0·662	0·399	0·345	0·331	2·18	1·328	1·275			
1·30	0·702	0·674	0·406	0·351	0·337	2·20	1·345	1·291			
1·32	0·715	0·686	0·413	0·357	0·343	2·22	1·362	1·308			
1·34	0·727	0·698	0·420	0·364	0·349	2·24	1·379				
1·36	0·740	0·710	0·428	0·370	0·355						
1·38	0·752	0·722	0·435	0·376	0·361						
1·40	0·765	0·734	0·442	0·382	0·367						
1·42	0·778	0·747	0·450	0·389	0·373						
1·44	0·790	0·759	0·457	0·395	0·379						
1·46	0·803	0·771	0·465	0·402	0·386						
1·48	0·816	0·784	0·472	0·408	0·392						

NOTE — Blanks indicate inadmissible reinforcement percentage (*see* Table E).

f_y
240
250
415
480
500

f_{ck}
20

TABLE 2 FLEXURE — REINFORCEMENT PERCENTAGE, p_t FOR SINGLY REINFORCED SECTIONS

f_{ck} = 20 N/mm²

M_u/bd^2, N/mm²	f_y, N/mm² 240	250	415	480	500	M_u/bd^2, N/mm²	f_y, N/mm² 240	250	415	480	500
0·30	0·146	0·140	0·085	0·073	0·070	2·22	1·253	1·203	0·725	0·627	0·602
0·35	0·171	0·164	0·099	0·086	0·082	2·24	1·267	1·216	0·733	0·633	0·608
0·40	0·196	0·188	0·114	0·098	0·094	2·26	1·281	1·230	0·741	0·640	0·615
0·45	0·222	0·213	0·128	0·111	0·106	2·28	1·295	1·243	0·749	0·647	0·621
0·50	0·247	0·237	0·143	0·123	0·119	2·30	1·309	1·256	0·757	0·654	0·628
0·55	0·272	0·262	0·158	0·136	0·131	2·32	1·323	1·270	0·765	0·661	0·635
0·60	0·298	0·286	0·172	0·149	0·143	2·34	1·337	1·283	0·773	0·668	0·642
0·65	0·324	0·311	0·187	0·162	0·156	2·36	1·351	1·297	0·781	0·675	0·648
0·70	0·350	0·336	0·203	0·175	0·168	2·38	1·365	1·311	0·790	0·683	0·655
0·75	0·376	0·361	0·218	0·188	0·181	2·40	1·380	1·324	0·798	0·690	0·662
0·80	0·403	0·387	0·233	0·201	0·193	2·42	1·394	1·338	0·806	0·697	0·669
0·85	0·430	0·412	0·248	0·215	0·206	2·44	1·408	1·352	0·814	0·704	0·676
0·90	0·456	0·438	0·264	0·228	0·219	2·46	1·423	1·366	0·823	0·711	0·683
0·95	0·483	0·464	0·280	0·242	0·232	2·48	1·438	1·380	0·831	0·719	0·690
1·00	0·511	0·490	0·295	0·255	0·245	2·50	1·452	1·394	0·840	0·726	0·697
1·05	0·538	0·517	0·311	0·269	0·258	2·52	1·467	1·408	0·848	0·734	0·704
1·10	0·566	0·543	0·327	0·283	0·272	2·54	1·482	1·423	0·857	0·741	0·711
1·15	0·594	0·570	0·343	0·297	0·285	2·56	1·497	1·437	0·866	0·748	0·719
1·20	0·622	0·597	0·359	0·311	0·298	2·58	1·512	1·451	0·874	0·756	0·726
1·25	0·650	0·624	0·376	0·325	0·312	2·60	1·527	1·466	0·883	0·764	0·733
1·30	0·678	0·651	0·392	0·339	0·326	2·62	1·542	1·481	0·892	0·771	0·740
1·35	0·707	0·679	0·409	0·354	0·339	2·64	1·558	1·495	0·901	0·779	0·748
1·40	0·736	0·707	0·426	0·368	0·353	2·66	1·573	1·510	0·910	0·786	0·755
1·45	0·765	0·735	0·443	0·383	0·367	2·68	1·588	1·525	0·919	0·794	
1·50	0·795	0·763	0·460	0·397	0·382	2·70	1·604	1·540	0·928		
1·55	0·825	0·792	0·477	0·412	0·396	2·72	1·620	1·555	0·937		
1·60	0·855	0·821	0·494	0·427	0·410	2·74	1·636	1·570	0·946		
1·65	0·885	0·850	0·512	0·443	0·425	2·76	1·651	1·585	0·955		
1·70	0·916	0·879	0·530	0·458	0·440	2·78	1·667	1·601			
1·75	0·947	0·909	0·547	0·473	0·454	2·80	1·683	1·616			
1·80	0·978	0·939	0·565	0·489	0·469	2·82	1·700	1·632			
1·85	1·009	0·969	0·584	0·505	0·484	2·84	1·716	1·647			
1·90	1·041	1·000	0·602	0·521	0·500	2·86	1·732	1·663			
1·95	1·073	1·030	0·621	0·537	0·515	2·88	1·749	1·679			
2·00	1·106	1·062	0·640	0·553	0·531	2·90	1·766	1·695			
2·02	1·119	1·074	0·647	0·559	0·537	2·92	1·782	1·711			
2·04	1·132	1·087	0·655	0·566	0·543	2·94	1·799	1·727			
2·06	1·145	1·099	0·662	0·573	0·550	2·96	1·816	1·743			
2·08	1·159	1·112	0·670	0·579	0·556	2·98	1·833	1·760			
2·10	1·172	1·125	0·678	0·586	0·562						
2·12	1·185	1·138	0·685	0·593	0·569						
2·14	1·199	1·151	0·693	0·599	0·575						
2·16	1·212	1·164	0·701	0·606	0·582						
2·18	1·226	1·177	0·709	0·613	0·588						
2·20	1·239	1·190	0·717	0·620	0·595						

NOTE — Blanks indicate inadmissible reinforcement percentage (*see* Table E).

TABLE 3 FLEXURE — REINFORCEMENT PERCENTAGE, p_t FOR SINGLY REINFORCED SECTIONS

f_{ck} = 25 N/mm²

M_u/bd^2, N/mm²	f_y, N/mm² 240	250	415	480	500	M_u/bd^2, N/mm²	f_y, N/mm² 240	250	415	480	500
0·30	0·146	0·140	0·084	0·073	0·070	2·55	1·415	1·358	0·818	0·708	0·679
0·35	0·171	0·164	0·099	0·085	0·082	2·60	1·448	1·390	0·837	0·724	0·695
0·40	0·195	0·188	0·113	0·098	0·094	2·65	1·482	1·422	0·857	0·741	0·711
0·45	0·220	0·211	0·127	0·110	0·106	2·70	1·515	1·455	0·876	0·758	0·727
0·50	0·245	0·236	0·142	0·123	0·118	2·75	1·549	1·487	0·896	0·775	0·744
0·55	0·271	0·260	0·156	0·135	0·130	2·80	1·584	1·520	0·916	0·792	0·760
0·60	0·296	0·284	0·171	0·148	0·142	2·85	1·618	1·554	0·936	0·809	0·777
0·65	0·321	0·309	0·186	0·161	0·154	2·90	1·653	1·587	0·956	0·827	0·794
0·70	0·347	0·333	0·201	0·174	0·167	2·95	1·689	1·621	0·977	0·844	0·811
0·75	0·373	0·358	0·216	0·186	0·179	3·00	1·724	1·655	0·997	0·862	0·828
0·80	0·399	0·383	0·231	0·199	0·191	3·05	1·760	1·690	1·018	0·880	0·845
0·85	0·425	0·408	0·246	0·212	0·204	3·10	1·797	1·725	1·039	0·898	0·863
0·90	0·451	0·433	0·261	0·225	0·216	3·15	1·834	1·760	1·061	0·917	0·880
0·95	0·477	0·458	0·276	0·239	0·229	3·20	1·871	1·796	1·082	0·936	0·898
1·00	0·504	0·483	0·291	0·252	0·242	3·25	1·909	1·832	1·104	0·954	0·916
1·05	0·530	0·509	0·307	0·265	0·255	3·30	1·947	1·869	1·126	0·973	0·935
1·10	0·557	0·535	0·322	0·279	0·267	3·32	1·962	1·884	1·135	0·981	0·942
1·15	0·584	0·561	0·338	0·292	0·280	3·34	1·978	1·899	1·144	0·989	
1·20	0·611	0·587	0·353	0·306	0·293	3·36	1·993	1·914	1·153		
1·25	0·638	0·613	0·369	0·319	0·306	3·38	2·009	1·929	1·162		
1·30	0·666	0·639	0·385	0·333	0·320	3·40	2·025	1·944	1·171		
1·35	0·693	0·666	0·401	0·347	0·333	3·42	2·040	1·959	1·180		
1·40	0·721	0·692	0·417	0·360	0·346	3·44	2·056	1·974	1·189		
1·45	0·749	0·719	0·433	0·374	0·359	3·46	2·072	1·989			
1·50	0·777	0·746	0·449	0·388	0·373	3·48	2·088	2·005			
1·55	0·805	0·773	0·466	0·403	0·387	3·50	2·104	2·020			
1·60	0·834	0·800	0·482	0·417	0·400	3·52	2·120	2·036			
1·65	0·862	0·828	0·499	0·431	0·414	3·54	2·137	2·051			
1·70	0·891	0·856	0·515	0·446	0·428	3·56	2·153	2·067			
1·75	0·920	0·883	0·532	0·460	0·442	3·58	2·170	2·083			
1·80	0·949	0·911	0·549	0·475	0·456	3·60	2·186	2·099			
1·85	0·979	0·940	0·566	0·489	0·470	3·62	2·203	2·115			
1·90	1·009	0·968	0·583	0·504	0·484	3·64	2·219	2·131			
1·95	1·038	0·997	0·601	0·519	0·498	3·66	2·236	2·147			
2·00	1·068	1·026	0·618	0·534	0·513	3·68	2·253	2·163			
2·05	1·099	1·055	0·635	0·549	0·527	3·70	2·270	2·179			
2·10	1·129	1·084	0·653	0·565	0·542	3·72	2·287	2·196			
2·15	1·160	1·114	0·671	0·580	0·557	3·74	2·304				
2·20	1·191	1·143	0·689	0·596	0·572						
2·25	1·222	1·173	0·707	0·611	0·587						
2·30	1·254	1·204	0·725	0·627	0·602						
2·35	1·285	1·234	0·743	0·643	0.617						
2·40	1·317	1·265	0·762	0·659	0·632						
2·45	1·350	1·296	0·781	0·675	0·648						
2·50	1·382	1·327	0·799	0·691	0·663						

NOTE — Blanks indicate inadmissible reinforcement percentage (*see* Table E).

f_y 240 250 415 480 500

f_{ck} 30

TABLE 4 FLEXURE — REINFORCEMENT PERCENTAGE, p_t FOR SINGLY REINFORCED SECTIONS

f_{ck} = 30 N/mm²

M_u/bd^2, N/mm²	f_y, N/mm² 240	250	415	480	500
0·30	0·145	0·140	0·084	0·073	0·070
0·35	0·170	0·163	0·098	0·085	0·082
0·40	0·195	0·187	0·113	0·097	0·093
0·45	0·219	0·211	0·127	0·110	0·105
0·50	0·244	0·235	0·141	0·122	0·117
0·55	0·269	0·259	0·156	0·135	0·129
0·60	0·294	0·283	0·170	0·147	0·141
0·65	0·320	0·307	0·185	0·160	0·153
0·70	0·345	0·331	0·200	0·172	0·166
0·75	0·370	0·356	0·214	0·185	0·178
0·80	0·396	0·380	0·229	0·198	0·190
0·85	0·422	0·405	0·244	0·211	0·202
0·90	0·447	0·429	0·259	0·224	0·215
0·95	0·473	0·454	0·274	0·237	0·227
1·00	0·499	0·479	0·289	0·250	0·240
1·05	0·525	0·504	0·304	0·263	0·252
1·10	0·552	0·529	0·319	0·276	0·265
1·15	0·578	0·555	0·334	0·289	0·277
1·20	0·604	0·580	0·350	0·302	0·290
1·25	0·631	0·606	0·365	0·315	0·303
1·30	0·658	0·631	0·380	0·329	0·316
1·35	0·685	0·657	0·396	0·342	0·329
1·40	0·712	0·683	0·411	0·356	0·342
1·45	0·739	0·709	0·427	0·369	0·355
1·50	0·766	0·735	0·443	0·383	0·368
1·55	0·793	0·762	0·459	0·397	0·381
1·60	0·821	0·788	0·475	0·410	0·394
1·65	0·849	0·815	0·491	0·424	0·407
1·70	0·876	0·841	0·507	0·438	0·421
1·75	0·904	0·868	0·523	0·452	0·434
1·80	0·932	0·895	0·539	0·466	0·448
1·85	0·961	0·922	0·556	0·480	0·461
1·90	0·989	0·950	0·572	0·495	0·475
1·95	1·018	0·977	0·589	0·509	0·488
2·00	1·046	1·005	0·605	0·523	0·502
2·05	1·075	1·032	0·622	0·538	0·516
2·10	1·104	1·060	0·639	0·552	0·530
2·15	1·134	1·088	0·656	0·567	0·544
2·20	1·163	1·116	0·673	0·581	0·558
2·25	1·192	1·145	0·690	0·596	0·572
2·30	1·222	1·173	0·707	0·611	0·587
2·35	1·252	1·202	0·724	0·626	0·601
2·40	1·282	1·231	0·742	0·641	0·615
2·45	1·312	1·260	0·759	0·656	0·630
2·50	1·343	1·289	0·777	0·671	0·645

M_u/bd^2, N/mm²	f_y, N/mm² 240	250	415	480	500
2·55	1·374	1·319	0·794	0·687	0·659
2·60	1·404	1·348	0·812	0·702	0·674
2·65	1·435	1·378	0·830	0·718	0·689
2·70	1·467	1·408	0·848	0·733	0·704
2·75	1·498	1·438	0·866	0·749	0·719
2·80	1·530	1·469	0·885	0·765	0·734
2·85	1·562	1·499	0·903	0·781	0·750
2·90	1·594	1·530	0·922	0·797	0·765
2·95	1·626	1·561	0·940	0·813	0·781
3·00	1·659	1·592	0·959	0·829	0·796
3·05	1·691	1·624	0·978	0·846	0·812
3·10	1·725	1·656	0·997	0·862	0·828
3·15	1·758	1·687	1·017	0·879	0·844
3·20	1·791	1·720	1·036	0·896	0·860
3·25	1·825	1·752	1·055	0·913	0·876
3·30	1·859	1·785	1·075	0·930	0·892
3·35	1·893	1·818	1·095	0·947	0·909
3·40	1·928	1·851	1·115	0·964	0·925
3·45	1·963	1·884	1·135	0·981	0·942
3·50	1·998	1·918	1·156	0·999	0·959
3·55	2·034	1·952	1·176	1·017	0·976
3·60	2·069	1·986	1·197	1·035	0·993
3·65	2·105	2·021	1·218	1·053	1·011
3·70	2·142	2·056	1·239	1·071	1·028
3·75	2·178	2·091	1·260	1·089	1·046
3·80	2·215	2·127	1·281	1·108	1·063
3·85	2·253	2·163	1·303	1·126	1·081
3·90	2·291	2·199	1·325	1·145	1·099
3·95	2·329	2·236	1·347	1·164	1·118
4·00	2·367	2·273	1·369	1·184	
4·05	2·406	2·310	1·391		
4·10	2·445	2·348	1·414		
4·15	2·485	2·386			
4·20	2·525	2·424			
4·25	2·566	2·463			
4·30	2·607	2·502			
4·35	2·648	2·542			
4·40	2·690	2·583			
4·45	2·733	2·623			

NOTE — Blanks indicate inadmissible reinforcement percentage (*see* Table E).

TABLE 5 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 250 N/mm²
Thickness = 10·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	Bar Spacing, cm	Bar Diameter, mm 6	8	10	12
5	8·92	14·02	0·00	0·00	20	0·00	4·20	6·27	8·55
6	7·59	12·20	0·00	0·00	21	0·00	4·01	6·00	8·19
7	6·61	10·77	0·00	0·00	22	0·00	3·83	5·74	7·87
8	5·85	9·63	13·56	0·00	23	0·00	3·67	5·51	7·56
9	5·24	8·70	12·40	0·00	24	0·00	3·53	5·30	7·28
10	4·75	7·93	11·41	0·00	25	0·00	3·39	5·10	7·02
11	4·34	7·29	10·56	13·81	26	0·00	3·27	4·92	6·78
12	4·00	6·74	9·82	12·95	27	0·00	3·15	4·75	6·56
13	3·70	6·26	9·18	12·19	28	0·00	3·04	4·59	6·34
14	3·45	5·85	8·61	11·50	29	0·00	2·94	4·44	6·14
15	3·23	5·49	8·11	10·88	30	0·00	2·85	4·30	5·96
16	3·04	5·17	7·66	10·32	35	0·00	0·00	3·72	5·17
17	2·86	4·89	7·26	9·81	40	0·00	0·00	3·27	4·56
18	2·71	4·63	6·90	9·35					
19	0·00	4·40	6·57	8·93					

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.

NOTE 2 — Bar spacings below the dividing line exceed $3d$.

TABLE 6 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 250 N/mm²
Thickness = 11·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	Bar Spacing, cm	Bar Diameter, mm 6	8	10	12
5	10·15	16·21	0·00	0·00	20	0·00	4·74	7·12	9·78
6	8·62	14·02	0·00	0·00	21	0·00	4·53	6·81	9·36
7	7·48	12·33	17·37	0·00	22	0·00	4·33	6·52	8·98
8	6·61	10·99	15·70	0·00	23	0·00	4·15	6·26	8·63
9	5·92	9·91	14·30	0·00	24	0·00	3·98	6·01	8·31
10	5·36	9·02	13·12	17·23	25	0·00	3·83	5·79	8·01
11	4·90	8·28	12·11	16·04	26	0·00	3·69	5·58	7·73
12	4·51	7·65	11·25	15·00	27	0·00	3·56	5·38	7·47
13	4·18	7·10	10·49	14·08	28	0·00	3·43	5·20	7·22
14	3·89	6·63	9·83	13·25	29	0·00	3·32	5·03	6·99
15	3·64	6·22	9·25	12·52	30	0·00	3·21	4·87	6·78
16	3·42	5·86	8·73	11·86	35	0·00	0·00	4·21	5·87
17	3·23	5·53	8·26	11·26	40	0·00	0·00	3·70	5·18
18	0·00	5·24	7·84	10·72	45	0·00	0·00	3·30	4·63
19	0·00	4·98	7·47	10·23					

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.

NOTE 2 — Bar spacings below the dividing line exceed $3d$.

f_y 250

f_{ck} 15

t 12 13

TABLE 7 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 250 N/mm²
Thickness = 12·0 cm

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12
5	11·37	18·39	0·00	0·00
6	9·64	15·84	22·22	0·00
7	8·36	13·89	19·81	0·00
8	7·38	12·36	17·83	0·00
9	6·61	11·13	16·20	21·30
10	5·98	10·12	14·83	19·68
11	5·46	9·27	13·67	18·28
12	5·02	8·56	12·67	17·05
13	4·65	7·95	11·80	15·97
14	4·33	7·41	11·05	15·01
15	4·05	6·95	10·38	14·16
16	0·00	6·54	9·79	13·39
17	0·00	6·17	9·27	12·71
18	0·00	5·85	8·79	12·09
19	0·00	5·55	8·36	11·52
20	0·00	5·29	7·98	11·01
21	0·00	5·05	7·62	10·53
22	0·00	4·83	7·30	10·10
23	0·00	4·62	7·00	9·70
24	0·00	4·44	6·72	9·33
25	0·00	4·27	6·47	8·99
26	0·00	4·11	6·23	8·67
27	0·00	3·96	6·02	8·38
28	0·00	0·00	5·81	8·10
29	0·00	0·00	5·62	7·84
30	0·00	0·00	5·44	7·60
35	0·00	0·00	4·69	6·57
40	0·00	0·00	4·13	5·79
45	0·00	0·00	0·00	5·18

Note 1 — Zeros indicate inadmissible reinforcement percentage.

Note 2 — Bar spacings below the dividing line exceed 3d.

TABLE 8 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 250 N/mm²
Thickness = 13·0 cm

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12	16
5	12·60	22·58	0·00	0·00	0·00
6	10·67	17·66	25·06	0·00	0·00
7	9·24	15·45	22·25	0·00	0·00
8	8·15	13·72	19·97	26·22	0·00
9	7·29	12·34	18·10	24·03	0·00
10	6·59	11·21	16·54	22·14	0·00
11	6·02	10·27	15·22	20·51	0·00
12	5·53	9·47	14·09	19·10	0·00
13	5·12	8·79	13·12	17·86	0·00
14	4·77	8·19	12·27	16·77	0·00
15	0·00	7·68	11·52	15·80	24·35
16	0·00	7·22	10·86	14·93	23·21
17	0·00	6·82	10·27	14·15	22·16
18	0·00	6·45	9·74	13·45	21·20
19	0·00	6·13	9·26	12·81	20·31
20	0·00	5·83	8·83	12·24	19·49
21	0·00	5·57	8·43	11·71	18·73
22	0·00	5·32	8·07	11·22	18·02
23	0·00	5·10	7·74	10·77	17·36
24	0·00	4·89	7·44	10·36	16·75
25	0·00	4·70	7·15	9·97	16·18
26	0·00	0·00	6·89	9·62	15·64
27	0·00	0·00	6·65	9·29	15·14
28	0·00	0·00	6·42	8·98	14·67
29	0·00	0·00	6·21	8·69	14·23
30	0·00	0·00	6·01	8·43	13·81
35	0·00	0·00	5·18	7·28	12·04
40	0·00	0·00	4·55	6·41	10·66
45	0·00	0·00	0·00	5·73	9·57

Note 1— Zeros indicate inadmissible reinforcement percentage.

Note 2— Bar spacings below the dividing line exceed 3d.

TABLE 9 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 250 N/mm²
Thickness = 14·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	16
5	13·83	22·76	31·99	0·00	0·00
6	11·69	19·48	27·91	0·00	0·00
7	10·12	17·01	24·69	0·00	0·00
8	8·92	15·09	22·10	29·30	0·00
9	7·97	13·56	19·99	26·76	0·00
10	7·21	12·30	18·24	24·60	0·00
11	6·58	11·26	16·77	22·75	0·00
12	6·05	10·38	15·51	21·15	0·00
13	5·60	9·63	14·43	19·75	0·00
14	0·00	8·97	13·49	18·52	28·71
15	0·00	8·41	12·66	17·44	27·26
16	0·00	7·90	11·93	16·47	25·94
17	0·00	7·46	11·28	15·60	24·73
18	0·00	7·06	10·69	14·82	23·63
19	0·00	6·70	10·16	14·11	22·61
20	0·00	6·38	9·68	13·46	21·67
21	0·00	6·09	9·25	12·88	20·81
22	0·00	5·82	8·85	12·34	20·01
23	0·00	5·57	8·48	11·84	19·26
24	0·00	0·00	8·15	11·38	18·57
25	0·00	0·00	7·84	10·96	17·93
26	0·00	0·00	7·55	10·56	17·32
27	0·00	0·00	7·28	10·20	16·76
28	0·00	0·00	7·03	9·86	16·23
29	0·00	0·00	6·80	9·54	15·73
30	0·00	0·00	6·58	9·24	15·27
35	0·00	0·00	5·67	7·98	13·29
40	0·00	0·00	0·00	7·02	11·76
45	0·00	0·00	0·00	6·27	10·54

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.
NOTE 2 — Bar spacings below the dividing line exceed 3*d*.

TABLE 10 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 250 N/mm²
Thickness = 15·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	16	18
5	15·06	24·95	35·41	0·00	0·00	0·00
6	12·71	21·30	30·76	0·00	0·00	0·00
7	11·00	18·57	27·13	35·81	0·00	0·00
8	9·69	16·46	24·24	32·37	0·00	0·00
9	8·66	14·77	21·89	29·49	0·00	0·00
10	7·82	13·39	19·95	27·06	0·00	0·00
11	7·14	12·25	18·32	24·98	0·00	0·00
12	6·56	11·29	16·94	23·20	0·00	0·00
13	0·00	10·47	15·75	21·64	33·66	0·00
14	0·00	9·76	14·71	20·28	31·83	0·00
15	0·00	9·13	13·80	19·07	30·17	0·00
16	0·00	8·59	13·00	18·00	28·67	33·32
17	0·00	8·10	12·28	17·25	27·30	31·87
18	0·00	7·67	11·64	16·18	26·05	30·53
19	0·00	7·28	11·06	15·40	24·91	29·28
20	0·00	6·93	10·54	14·69	23·86	28·13
21	0·00	6·61	10·06	14·05	22·89	27·06
22	0·00	6·32	9·63	13·45	21·99	26·06
23	0·00	0·00	9·23	12·91	21·16	25·13
24	0·00	0·00	8·86	12·41	20·39	24·26
25	0·00	0·00	8·52	11·94	19·67	23·45
26	0·00	0·00	8·20	11·51	19·00	22·68
27	0·00	0·00	7·91	11·11	18·38	21·97
28	0·00	0·00	7·64	10·73	17·79	21·29
29	0·00	0·90	7·39	10·38	17·24	20·66
30	0·00	0·00	7·15	10·05	16·72	20·06
35	0·00	0·00	0·00	8·68	14·53	17·52
40	0·00	0·00	0·00	7·64	12·85	15·54
45	0·00	0·00	0·00	6·82	11·51	13·96

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.
NOTE 2 — Bar spacings below the dividing line exceed 3*d*.

TABLE 11 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 250 N/mm²
Thickness = 17·5 cm

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12	16	18	20
5	18·14	30·41	43·95	0·00	0·00	0·00	0·00
6	15·28	25·86	37·87	50.17	0·00	0·00	0·00
7	13·19	22·48	33·22	44.59	0·00	0·00	0·00
8	11·61	19·87	29·57	40.05	0·00	0·00	0·00
9	10·36	17·81	26·63	36.32	0·00	0·00	0·00
10	9·36	16·13	24·22	33·21	0·00	0·00	0·00
11	0·00	14·74	22·20	30·57	47·84	0·00	0·00
12	0·00	13·57	20·49	28·32	44·78	0·00	0·00
13	0·00	12·57	19·03	26·37	42·06	0·00	0·00
14	0·00	11·71	17·76	24·67	39·63	46·45	0·00
15	0·00	10·96	16·64	23·17	37·46	44·10	0·00
16	0·00	10·29	15·66	21·85	35·50	41·96	0·00
17	0·00	9·71	14·79	20·66	33·73	40·00	45·82
18	0·00	9·19	14·01	19·60	32·12	38·21	43·93
19	0·00	8·72	13·31	18·64	30·66	36·56	42·17
20	0·00	0·00	12·67	17·77	29·32	35·04	40·53
21	0·00	0·00	12·09	16·97	28·09	33·64	39·01
22	0·00	0·00	11·57	16·25	26·96	32·34	37·59
23	0·00	0·00	11·08	15·58	25·91	31·14	36·26
24	0·00	0·00	10·64	14·97	24·95	30·02	35·02
25	0·00	0·00	10·23	14·40	24·05	28·98	33·86
26	0·00	0·00	9·85	13·87	23·21	28·00	32·77
27	0·00	0·00	9·49	13·38	22·43	27·09	31·75
28	0·00	0·00	9·17	12·93	21·69	26·23	30·78
29	0·00	0·00	8·86	12·50	21·01	25·43	29·88
30	0·00	0·00	0·00	12·10	20·36	24·67	29·02
35	0·00	0·00	0·00	10·44	17·66	21·47	25·36
40	0·00	0·00	0·00	9·17	15·58	18·99	22·51
45	0·00	0·00	0·00	0·00	13·94	17·03	20·23

NOTE — Zeros indicate inadmissible reinforcement percentage.

TABLE 12 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 250 N/mm²
Thickness = 20·0 cm

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12	16	18	20	22	25
5	21·21	35·88	52·48	69·39	0·00	0·00	0·00	0·00	0·00
6	17·84	30·41	44·98	60·41	0·00	0·00	0·00	0·00	0·00
7	15·39	26·38	39·32	53·37	0·00	0·00	0·00	0·00	0·00
8	13·53	23·29	34·91	47·74	0·00	0·00	0·00	0·00	0·00
9	12·07	20·84	31·38	43·15	67·31	0·00	0·00	0·00	0·00
10	0·00	18·86	28·49	39·35	62·21	0·00	0·00	0·00	0·00
11	0·00	17·22	26·08	36·16	57·77	0·00	0·00	0·00	0·00
12	0·00	15·84	24·05	33·44	53·89	63·38	0·00	0·00	0·00
13	0·00	14·67	22·31	31·10	50·47	59·67	0·00	0·00	0·00
14	0·00	13·66	20·81	29·06	47·44	56·33	0·00	0·00	0·00
15	0·00	12·78	19·49	27·27	44·74	53·32	61·43	0·00	0·00
16	0·00	12·00	18·33	25·69	42·33	50·61	58·53	0·00	0·00
17	0·00	0·00	17·30	24·28	40·16	48·14	55·86	0·00	0·00
18	0·00	0·00	16·38	23·01	38·20	45·89	53·41	60·43	0·00
19	0·00	0·00	15·55	21·87	36·41	43·84	51·15	58·06	0·00
20	0·00	0·00	14·81	20·84	34·78	41·96	49·07	55·85	0·00
21	0·00	0·00	14·13	19·90	33·30	40·23	47·14	53·79	0·00
22	0·00	0·00	13·51	19·04	31·93	38·63	45·35	51·86	0·00
23	0·00	0·00	12·94	18·25	30·66	37·15	43·69	50·06	58·81
24	0·00	0·00	12·42	17·53	29·50	35·78	42·14	48·37	57·02
25	0·00	0·00	11·93	15·86	28·42	34·51	40·69	46·78	55·33
26	0·00	0·00	11·49	16·24	27·41	33·32	39·34	45·30	53·72
27	0·00	0·00	0·00	15·66	26·47	32·21	38·07	43·90	52·19
28	0·00	0·00	0·00	15·12	25·60	31·17	36·88	42·58	50·74
29	0·00	0·00	0·00	14·62	24·78	30·20	35·76	41·33	49·36
30	0·00	0·00	0·00	14·15	24·01	29·28	34·71	40·16	48·06
35	0·00	0·00	0·00	12·19	20·78	25·42	30·24	35·14	42·38
40	0·00	0·00	0·00	0·00	18·31	22·45	26·78	31·21	37·87
45	0·00	0·00	0·00	0·00	16·37	20·10	24·03	28·07	34·20

NOTE — Zeros indicate inadmissible reinforcement percentage.

f_y 250

f_{ck} 15

t 22.5

TABLE 13 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 250 N/mm²
Thickness = 22·5 cm

BAR SPACING, cm	BAR DIAMETER, mm								
	6	8	10	12	16	18	20	22	25
5	24·28	41·34	61·02	81·69	0·00	0·00	0·00	0·00	0·00
6	20·40	34·96	52·10	70·66	0·00	0·00	0·00	0·00	0·00
7	17·58	30·28	45·42	62·15	0·00	0·00	0·00	0·00	0·00
8	15·45	26·70	40·24	55·42	86·82	0·00	0·00	0·00	0·00
9	0·00	23·88	36·12	49·98	79·45	0·00	0·00	0·00	0·00
10	0·00	21·59	32·76	45·50	73·14	85·96	0·00	0·00	0·00
11	0·00	19·70	29·96	41·73	67·71	80·09	0·00	0·00	0·00
12	0·00	18·12	27·61	38·56	62·99	74·91	0·00	0·00	0·00
13	0·00	16·77	25·60	35·83	58·87	70·31	81·18	0·00	0·00
14	0·00	15·61	23·86	33·45	55·24	66·21	76·79	0·00	0·00
15	0·00	0·00	22·34	31·37	52·03	62·54	72·81	0·00	0·00
16	0·00	0·00	21·00	29·53	49·16	59·25	69·20	78·62	0·00
17	0·00	0·00	19·81	27·89	46·59	56·27	65·91	75·13	0·00
18	0·00	0·00	18·75	26·43	44·27	53·58	62·90	71·91	0·00
19	0·00	0·00	17·80	25·11	42·16	51·12	60·14	68·93	0·00
20	0·00	0·00	16·94	23·91	40·25	48·87	57·61	66·18	78·11
21	0·00	0·00	16·16	22·83	38·50	46·81	55·27	63·63	75·39
22	0·00	0·00	15·45	21·84	36·89	44·92	53·11	61·25	72·82
23	0·00	0·00	14·79	20·93	35·42	43·17	51·11	59·04	70·41
24	0·00	0·00	0·00	20·09	34·05	41·55	49·25	56·98	68·14
25	0·00	0·00	0·00	19·32	32·79	40·04	47·52	55·05	66·00
26	0·00	0·00	0·00	18·60	31·61	38·64	45·91	53·24	63·98
27	0·00	0·00	0·00	17·94	30·52	37·33	44·40	51·55	62·07
28	0·00	0·00	0·00	17·32	29·50	36·11	42·98	49·96	60·27
29	0·00	0·00	0·00	16·74	28·54	34·97	41·65	48·46	58·56
30	0·00	0·00	0·00	16·20	27·65	33·89	40·40	47·04	56·95
35	0·00	0·00	0·00	0·00	23·90	29·37	35·12	41·04	50·01
40	0·00	0·00	0·00	0·00	21·04	25·91	31·05	36·38	44·54
45	0·00	0·00	0·00	0·00	18·80	23·18	27·82	32·66	40·13

NOTE — Zeros indicate inadmissible reinforcement percentage.

TABLE 14 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm² .
f_y = 250 N/mm²
Thickness = 25·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	16	18	20	22	25
5	27·36	46·80	69·56	93·98	0·00	0·00	0·00	0·00	0·00
6	22·96	39·51	59·21	80·90	0·00	0·00	0·00	0·00	0·00
7	19·78	34·18	51·52	70·93	111·08	0·00	0·00	0·00	0·00
8	0·00	30·12	45·58	63·10	100·48	0·00	0·00	0·00	0·00
9	0·00	26·91	40·86	56·81	91·59	107·96	0·00	00	0·00
10	0·00	24·32	37·02	51·65	84·07	99·79	0·00	0·00	0·00
11	0·00	22·19	33·84	47·34	77·64	92·66	106·90	0·00	0·00
12	0·00	20·40	31·17	43·69	72·10	86·43	100·26	0·00	0·00
13	0·00	18·87	28·88	40·55	67·28	80·94	94·32	0·00	0·00
14	0·00	0·00	26·90	37·84	63·05	76·09	88·99	101·25	0·00
15	0·00	0·00	25·18	35·47	59·31	71·76	84·20	96·17	0·00
16	0·00	0·00	23·67	33·37	55·99	67·89	79·87	91·53	0·00
17	0·00	0·00	22·32	31·51	53·01	64·41	75·95	87·28	0·00
18	0·00	0·00	21·12	29·84	50·34	61·26	72·38	83·39	98·90
19	0·00	0·00	20·05	28·34	47·91	58·40	69·13	79·81	95·05
20	0·00	0·00	19·07	26·99	45·71	55·79	66·14	76·51	91·45
21	0·00	0·00	0·00	25·75	43·70	53·40	63·40	73·46	88·09
22	0·00	0·00	0·00	24·63	41·86	51·20	60·87	70·64	84·95
23	0·00	0·00	0·00	23·60	40·17	49·18	58·53	68·02	82·01
24	0·00	0·00	0·00	22·65	38·60	47·31	56·37	65·58	79·25
25	0·00	0·00	0·00	21·78	37·16	45·57	54·35	63·31	76·67
26	0·00	0·00	0·00	20·97	35·82	43·96	52·47	61·19	74·24
27	0·00	0·00	0·00	20·21	34·57	42·46	50·72	59·20	71·95
28	0·00	0·00	0·00	19·51	33·40	41·05	49·08	57·33	69·80
29	0·00	0·00	0·00	18·86	32·31	39·74	47·54	55·58	67·76
30	0·00	0·00	0·00	18·25	31·29	38·50	46·09	53·93	65·84
35	0·00	0·00	0·00	0·00	27·02	33·32	40·00	46·94	57·63
40	0·00	0·00	0·00	0·00	23·78	29·37	35·32	41·54	51·21
45	0·00	0·00	0·00	0·00	21·22	26·25	31·61	37·25	46·06

NOTE — Zeros indicate inadmissible reinforcement percentage.

TABLE 15 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 415 N/mm²
Thickness = 10·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	Bar Spacing, cm	Bar Diameter, mm 6	8	10	12
5	13·53	0·00	0·00	0·00	20	3·98	6·71	9·79	12·91
6	11·72	0·00	0·00	0·00	21	3·80	6·42	9·39	12·44
7	10·32	0·00	0·00	0·00	22	3·64	6·15	9·03	12·00
8	9·21	0·00	0·00	0·00	23	3·49	5·91	8·69	11·59
9	8·31	13·20	0·00	0·00	24	0·00	5·68	8·37	11·21
10	7·56	12·16	0·00	0·00	25	0·00	5·47	8·08	10·84
11	6·94	11·26	0·00	0·00	26	0·00	5·28	7·81	10·50
12	6·42	10·48	0·00	0·00	27	0·00	5·09	7·55	10·18
13	5·96	9·80	0·00	0·00	28	0·00	4·92	7·31	9·88
14	5·57	9·20	13·04	0·00	29	0·00	4·77	7·08	9·59
15	5·22	8·67	12·37	0·00	30	0·00	4·62	6·87	9·32
16	4·92	8·19	11·75	0·00	35	0·00	3·99	5·97	8·16
17	4·64	7·77	11·20	0·00	40	0·00	3·51	5·28	7·26
18	4·40	7·38	10·69	0·00					
19	4·18	7·03	10·22	0·00					

Note 1 — Zeros indicate inadmissible reinforcement percentage.
Note 2 — Bar spacings below the dividing line exceed 3d.

TABLE 16 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 415 N/mm²
Thickness = 11·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	Bar Spacing, cm	Bar Diameter, mm 6	8	10	12
5	15·57	0·00	0·00	0·00	20	4·49	7·62	11·21	14·95
6	13·42	0·00	0·00	0·00	21	4·29	7·29	10·74	14·39
7	11·77	0·00	0·00	0·00	22	0·00	6·98	10·32	13·86
8	10·48	16·67	0·00	0·00	23	0·00	6·70	9·92	13·37
9	9·44	15·21	0·00	0·00	24	0·00	6·44	9·55	12·91
10	8·59	13·97	0·00	0·00	25	0·00	6·20	9·21	12·48
11	7·87	12·91	0·00	0·00	26	0·00	5·97	8·90	12·07
12	7·27	12·00	0·00	0·00	27	0·00	5·77	8·60	11·69
13	6·75	11·20	15·96	0·00	28	0·00	5·57	8·32	11·34
14	6·30	10·50	15·06	0·00	29	0·00	5·39	8·06	11·00
15	5·90	9·88	14·26	0·00	30	0·00	5·22	7·82	10·68
16	5·55	9·33	13·53	0·00	35	0·00	4·51	6·78	9·33
17	5·24	8·83	12·86	0·00	40	0·00	0·00	5·99	8·28
18	4·97	8·39	12·26	16·22	45	0·00	0·00	5·36	7·44
19	4·72	7·99	11·71	15·56					

Note 1 — Zeros indicate inadmissible reinforcement percentage.
Note 2 — Bar spacings below the dividing line exceed 3d.

TABLE 17 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 415 N/mm²
Thickness = 12·0 cm

Bar Spacing, cm	Bar Diameter, mm			
	6	8	10	12
5	17·61	0·00	0·00	0·00
6	15·12	0·00	0·00	0·00
7	13·23	21·00	0·00	0·00
8	11·76	18·94	0·00	0·00
9	10·57	17·23	0·00	0·00
10	9·61	15·79	0·00	0·00
11	8·80	14·56	20·65	0·00
12	8·12	13·51	19·32	0·00
13	7·53	12·59	18·14	0·00
14	7·02	11·79	17·09	0·00
15	6·58	11·09	16·14	0·00
16	6·19	10·46	15·30	20·24
17	5·84	9·90	14·53	19·33
18	5·53	9·40	13·84	18·49
19	5·26	8·94	13·20	17·71
20	0·00	8·53	12·62	16·99
21	0·00	8·15	12·09	16·33
22	0·00	7·80	11·60	15·71
23	0·00	7·49	11·15	15·14
24	0·00	7·19	10·74	14·61
25	0·00	6·92	10·35	14·11
26	0·00	6·67	9·99	13·64
27	0·00	6·44	9·65	13·20
28	0·00	6·22	9·33	12·79
29	0·00	6·02	9·04	12·41
30	0·00	5·83	8·76	12·04
35	0·00	0·00	7·59	10·50
40	0·00	0·00	6·70	9·30
45	0·00	0·00	5·99	8·35

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.

NOTE 2 — Bar spacings below the dividing line exceed 3*d*.

TABLE 18 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 415 N/mm²
Thickness = 13·0 cm

Bar Spacing, cm	Bar Diameter, mm				
	6	8	10	12	16
5	19·65	0·00	0·00	0·00	0·00
6	16·82	0·00	0·00	0·00	0·00
7	14·69	23·59	0·00	0·00	0·00
8	13·03	21·21	0·00	0·00	0·00
9	11·71	19·24	0·00	0·00	0·00
10	10·63	17·60	24·99	0·00	0·00
11	9·73	16·21	23·23	0·00	0·00
12	8·97	15·02	21·68	0·00	0·00
13	8·32	13·99	20·32	0·00	0·00
14	7·75	13·09	19·11	0·00	0·00
15	7·26	12·30	18·03	23·95	0.00
16	6·83	11·59	17·07	22·79	0.00
17	6·44	10·97	16·20	21·73	0.00
18	6·10	10·40	15·41	20·75	0.00
19	0·00	9·90	14·69	19·86	0.00
20	0·00	9·43	14·04	19·04	0·00
21	0·00	9·01	13·44	18·27	0·00
22	0·00	8·63	12·89	17·57	0·00
23	0·00	8·28	12·39	16·92	0·00
24	0·00	7·95	11·92	16·31	0·00
25	0·00	7·65	11·48	15·74	0·00
26	0·00	7·37	11·08	15·21	0·00
27	0·00	7·11	10·70	14·72	22·92
28	0·00	6·87	10·35	14·25	22·30
29	0·00	6·64	10·01	13·81	21·70
30	0·00	6·43	9·70	13·40	21·13
35	0·00	0·00	8·40	11·66	18·66
40	0·00	0·00	7·41	10·32	16·69
45	0·00	0·00	6·62	9·25	15·09

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.

NOTE 2 — Bar spacings below the dividing line exceed 3*d*.

f_y 415

f_{ck} 15

t 14

TABLE 19 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 415 N/mm²
Thickness = 14·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	16
5	21·69	0·00	0·00	0·00	0·00
6	18·52	29·54	0·00	0·00	0·00
7	16·15	26·18	0·00	0·00	0·00
8	14·31	23·48	0·00	0·00	0·00
9	12·84	21·26	0·00	0·00	0·00
10	11·65	19·41	27·82	0·00	0·00
11	10·65	17·86	25·80	0·00	0·00
12	9·82	16·53	24·05	0·00	0·00
13	9·10	15·38	22·50	0·00	0·00
14	8·48	14·38	21·14	28·14	0·00
15	7·94	13·51	19·92	26·68	0·00
16	7·47	12·73	18·84	25·34	0·00
17	0·00	12·03	17·87	24·13	0·00
18	0·00	11·41	16·99	23·02	0·00
19	0·00	10·85	16·19	22·01	0·00

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	16
20	0·00	10·34	15·46	21·08	0·00
21	0·00	9·88	14·79	20·22	0·00
22	0·00	9·45	14·18	19·43	0·00
23	0·00	9·06	13·62	18·69	0·00
24	0·00	8·71	13·10	18·01	0·00
25	0·00	8·37	12·61	17·37	27·18
26	0·00	8·07	12·17	16·78	26·37
27	0·00	7·79	11·75	16·23	25·61
28	0·00	7·52	11·36	15·71	24·89
29	0·00	7·27	10·99	15·22	24·20
30	0·00	0·00	10·65	14·76	23·55
35	0·00	0·00	9·21	12·83	20·74
40	0·00	0·00	8·12	11·34	18·51
45	0·00	0·00	7·25	10·16	16·70

Note 1 — Zeros indicate inadmissible reinforcement percentage.
Note 2 — Bar spacings below the dividing line exceed 3*d*.

TABLE 20 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 415 N/mm²
Thickness = 15·0 cm

Bar Spacing, cm	Bar Diameter, mm					
	6	8	10	12	16	18
5	23·73	0·00	0·00	0·00	0·00	0·00
6	20·22	32·56	0·00	0·00	0·00	0·00
7	17·60	28·77	0·00	0·00	0·00	0·00
8	15·58	25·74	0·00	0·00	0·00	0·00
9	13·97	23·27	33·30	0·00	0·00	0·00
10	12·67	21·23	30·66	0·00	0·00	0·00
11	11·58	19·51	28·38	0·00	0·00	0·00
12	10·67	18·04	26·41	0·00	0·00	0·00
13	9·89	16·78	24·68	32·91	0·00	0·00
14	9·21	15·68	23·16	31·06	0·00	0·00
15	8·62	14·72	21·81	29·40	0·00	0·00
16	0·00	13·86	20·61	27·89	0·00	0·00
17	0·00	13·10	19·53	26·53	0·00	0·00
18	0·00	12·42	18·56	25·29	0·00	0·00
19	0·00	11·80	17·68	24·16	0·00	0·00
20	0·00	11·25	16·88	23·12	0·00	0·00
21	0·00	10·74	16·14	22·16	0·00	0·00
22	0·00	10·28	15·47	21·28	0·00	0·00
23	0·00	9·85	14·85	20·47	32·08	0·00
24	0·00	9·46	14·28	19·71	31·05	0·00
25	0·00	9·10	13·75	19·01	30·08	0·00
26	0·00	8·76	13·26	18·35	29·16	0·00
27	0·00	8·45	12·80	17·74	28·30	0·00
28	0·00	0·00	12·37	17·17	27·48	0·00
29	0·00	0·00	11·97	16·63	26·70	31·22
30	0·00	0·00	11·59	16·12	25·97	30·43
35	0·00	0·00	10·02	14·00	22·81	26·97
40	0·00	0·00	8·82	12·36	20·32	24·18
45	0·00	0·00	0·00	11·07	18·31	21·89

NOTE — Zeros indicate inadmissible reinforcement percentage.

f_y 415

f_{ck} 15

t 17.5

TABLE 21 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 415 N/mm²
Thickness = 17·5 cm

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12	16	18	20
5	28·83	46·46	0·00	0·00	0·00	0·00	0·00
6	24·47	40·12	0·00	0·00	0·00	0·00	0·00
7	21·25	35·25	0·00	0·00	0·00	0·00	0·00
8	18·77	31·41	45·24	0·00	0·00	0·00	0·00
9	16·81	28·31	41·17	0·00	0·00	0·00	0·00
10	15·22	25·76	37·74	0·00	0·00	0·00	0·00
11	13·90	23·63	34·82	46·53	0·00	0·00	0·00
12	12·79	21·82	32·31	43·47	0·00	0·00	0·00
13	11·85	20·27	30·13	40·76	0·00	0·00	0·00
14	0·00	18·92	28·22	38·35	0·00	0·00	0.00
15	0·00	17·74	26·54	36·20	0·00	0·00	0·00
16	0·00	16·70	25·04	34·27	0·00	0·00	0·00
17	0·00	15·77	23·70	32·53	0·00	0·00	0·00
18	0·00	14·94	22·50	30·96	0·00	0·00	0·00
19	0·00	14·19	21·41	29·53	46·43	0·00	0·00
20	0·00	13·51	20·42	28·22	44·64	0·00	0·00
21	0·00	12·90	19·52	27·02	42·98	0·00	0·00
22	0·00	12·34	18·69	25·92	41·42	0·00	0·00
23	0·00	11·82	17·93	24·90	39·97	0·00	0·00
24	0·00	0·00	17·23	23·96	38·61	45·35	0·00
25	0·00	0·00	16·58	23·09	37·34	43·97	0·00
26	0·00	0·00	15·98	22·28	36·14	42·66	0·00
27	0·00	0·00	15·42	21·52	35·02	41·43	0·00
28	0·00	0·00	14·90	20·81	33·96	40·25	0·00
29	0·00	0·00	14·41	20·15	32·96	39·14	0·00
30	0·00	0·00	13·96	19·53	32·01	38·08	0·00
35	0·00	0·00	12·05	16·91	27·99	33·53	38·89
40	0·00	0·00	0·00	14·91	24·86	29·92	34·91
45	0·00	0·00	0·00	13·33	22·34	26·99	31·64

Note — Zeros indicate inadmissible reinforcement percentage.

TABLE 22 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 415 N/mm²
Thickness = 20·0 cm

Bar Spacing, cm	Bar Diameter, mm								
	6	8	10	12	16	18	20	22	25
5	33·93	55·53	0·00	0·00	0·00	0·00	0·00	0·00	0·00
6	28·72	47·68	0·00	0·00	0·00	0·00	0·00	0·00	0·00
7	24·89	41·73	60·25	0·00	0·00	0·00	0·00	0·00	0·00
8	21·96	37·08	54·10	0·00	0·00	0·00	0·00	0·00	0·00
9	19·64	33·35	49·05	65·33	0·00	0·00	0·00	0·00	0·00
10	17·77	30·30	44·83	60·22	0·00	0·00	0·00	0·00	0·00
11	16·22	27·75	41·26	55·81	0·00	0·00	0·00	0·00	0·00
12	0·00	25·60	38·22	51·97	0·00	0·00	0·00	0·00	0·00
13	0·00	23·76	35·58	48·60	0·00	0·00	0·00	0·00	0·00
14	0·00	22·16	33·28	45·64	0·00	0·00	0·00	0·00	0·00
15	0·00	20·76	31·26	43·00	0·00	0·00	0·00	0·00	0·00
16	0·00	19·53	29·47	40·65	63·97	0·00	0·00	0·00	0·00
17	0·00	18·44	27·87	38·53	61·08	0·00	0·00	0·00	0·00
18	0·00	17·46	26·43	36·63	58·43	0·00	0·00	0·00	0·00
19	0·00	16·58	25·14	34·90	55·98	0·00	0·00	0·00	0·00
20	0·00	15·78	23·96	33·32	53·71	0·00	0·00	0·00	0·00
21	0·00	0·00	22·89	31·88	51·61	60·92	0·00	0·00	0·00
22	0·00	0·00	21·91	30·56	49·67	58·79	0·00	0·00	0·00
23	0·00	0·00	21·01	29·34	47·86	56·79	0·00	0·00	0·00
24	0·00	0·00	20·18	28·21	46·17	54·92	0·00	0·00	0·00
25	0·00	0·00	19·42	27·17	44·59	53·15	0·00	0·00	0·00
26	0·00	0·00	18·71	26·20	43·12	51·49	59·48	0·00	0·00
27	0·00	0·00	18·05	25·30	41·73	49·93	57·80	0·00	0·00
28	0·00	0·00	17·43	24·45	40·43	48·45	56·20	0·00	0·00
29	0·00	0·00	16·86	23·67	39·21	47·06	54·69	0·00	0·00
30	0·00	0·00	16·32	22·93	38·06	45·74	53·24	0·00	0·00
35	0·00	0·00	0·00	19·83	33·18	40·09	46·98	53·62	0·00
40	0·00	0·00	0·00	17·46	29·39	35·66	41·99	48·21	0·00
45	0·00	0·00	0·00	15·60	26·38	32·10	37·94	43·75	52·03

NOTE — Zeros indicate inadmissible reinforcement percentage.

TABLE 23 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm$_2$
f_y = 415 N/m^2
Thickness = 22·5cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	16	18	20	22	25
5	39·03	64·59	0·00	0·00	0·00	0·00	0·00	0·00	0·00
6	32·97	55·24	79·65	0·00	0·00	0·00	0·00	0·00	0·00
7	28·54	48·21	70·37	0·00	0·00	0·00	0·00	0·00	0·00
8	25·15	42·75	62·96	84·02	0·00	0·00	0·00	0·00	0·00
9	22·48	38·39	56·92	76·67	0·00	0·00	0·00	0·00	0·00
10	20·32	34·83	51·91	70·43	0·00	0·00	0·00	0·00	0·00
11	0·00	31·88	47·71	65·09	0·00	0·00	0·00	0·00	0·00
12	0·00	29·38	44·12	60·47	0·00	0·00	0·00	0·00	0·00
13	0·00	27·24	41·03	56·45	0·00	0·00	0·00	0·00	0·00
14	0·00	25·40	38·34	52·92	83·48	0·00	0·00	0·00	0·00
15	0·00	23·78	35·98	49·80	79·20	0·00	0·00	0·00	0·00
16	0·00	22·36	33·90	47·02	75·31	0·00	0·00	0·00	0·00
17	0·00	21·10	32·04	44·54	71·75	0·00	0·00	0·00	0·00
18	0·00	19·98	30·37	42·29	68·50	80·96	0·00	0·00	0·00
19	0·00	0·00	28·87	40·27	65·52	77·70	0·00	0·00	0·00
20	0·00	0·00	27·50	38·42	62·78	74·67	0·00	0·00	0·00
21	0·00	0·00	26·26	36·74	60·25	71·85	0·00	0·00	0·00
22	0·00	0·00	25·13	35·19	57·91	69·73	0·00	0·00	0·00
23	0·00	0·00	24·09	33·77	55·74	66·77	77·40	0·00	0·00
24	0·00	0·00	23·14	32·46	53·73	64·48	74·92	0·00	0·00
25	0·00	0·00	22·25	31·25	51·85	62·34	72·59	0·00	0·00
26	0·00	0·00	21·43	30·12	50·09	60·32	70·38	0·00	0·00
27	0·00	0·00	20·67	29·08	48·45	58·43	68·30	0·00	0·00
28	0·00	0·00	19·96	28·10	46·91	56·65	66·33	75·57	0·00
29	0·00	0·00	19·30	27·18	45·47	54·97	64·46	73·58	0·00
30	0·00	0·00	0·00	26·33	44·11	53·39	62·69	71·69	0·00
35	0·00	0·00	0·00	22·74	38·36	46·65	55·08	63·42	0·00
40	0·00	0·00	0·00	20·01	33·93	41·40	49·08	56·79	67·93
45	0·00	0·00	0·00	0·00	30·41	37·20	44·24	51·37	61·87

NOTE — Zeros indicate inadmissible reinforcement percentage.

f_y 415

f_{ck} 15

t 25

TABLE 24 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 415 N/mm²
Thickness = 25·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	16	18	20	22	25
5	44·14	73·66	105·62	0·00	0·00	0·00	0·00	0·00	0·00
6	37·23	62·80	91·46	0·00	0·00	0·00	0·00	0·00	0·00
7	32·18	54·69	80·50	107·33	0·00	0·00	0·00	0·00	0·00
8	28·34	48·42	71·82	96·78	0·00	0·00	0·00	0·00	0·00
9	25·31	43·43	64·79	88·01	0·00	0·00	0·00	0·00	0·00
10	0·00	39·37	59·00	80·63	0·00	0·00	0·00	0·00	0·00
11	0·00	36·00	54·15	74·36	0·00	0·00	0·00	0·00	0·00
12	0·00	33·16	50·03	68·97	0·00	0·00	0·00	0·00	0·00
13	0·00	30·73	46·48	64·30	102·14	0·00	0·00	0·00	0·00
14	0·00	28·64	43·41	60·21	96·44	0·00	0·00	0·00	0·00
15	0·00	26·81	40·71	56·61	91·29	0·00	0·00	0·00	0·00
16	0·00	25·20	38·33	53·40	86·64	102·61	0·00	0·00	0·00
17	0·00	0·00	36·20	50·54	82·43	97·98	0·00	0·00	0·00
18	0·00	0·00	34·31	47·96	78·58	93·72	0·00	0·00	0·00
19	0·00	0·00	32·60	45·64	75·07	89·78	0·00	0·00	0·00
20	0·00	0·00	31·05	43·52	71·85	86·15	99·95	0·00	0·00
21	0·00	0·00	29·64	41·60	68·89	82·78	96·32	0·00	0·00
22	0·00	0·00	28·35	39·83	66·16	79·66	92·91	0·00	0·00
23	0·00	d·00	27·17	38·21	63·63	76·76	89·72	0·00	0·0g
24	0·00	0·00	26·09	36·72	61·28	74·05	86·73	0·00	0·00
25	0·00	0·00	25·09	35·33	59·10	71·52	83·92	95·88	0·00
26	0·00	0·00	24·16	34·05	57·07	69·15	81·28	93·05	0·00
27	0·00	0·00	0·00	32·86	55·17	66·93	78·79	90·37	0·00
28	0·00	0·00	0·00	31·74	53·39	64·85	76·45	87·82	0·00
29	0·00	0·00	0·00	30·70	51·72	62·89	74·23	85·41	0·00
30	0·00	0·00	0·00	29·73	50·15	61·04	72·14	83·12	0·00
35	0·00	0·00	0·00	25·66	43·54	53·21	63·18	73·22	87·82
40	0·00	0·00	0·00	0·00	38·46	47·14	56·17	65·36	79·00
45	0·00	0·00	0·00	0·00	34·44	42·30	50·54	58·99.	71·71

Note — Zeros indicate inadmissible reinforcement percentage.

TABLE 25 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 250 N/mm²
Thickness = 10·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12
5	9·21	14·94	0·00	0·00
6	7·79	12·84	18·09	0·00
7	6·75	11·24	16·08	0·00
8	5·96	9·99	14·44	0·00
9	5·33	8·98	13·10	17·27
10	4·82	8·16	11·97	15·93
11	4·40	7·48	11·03	14·77
12	4·05	6·90	10·21	13·76
13	3·75	6·40	9·51	12·87
14	3·49	5·97	8·90	12·09
15	3·26	5·59	8·36	11·40
16	3·06	5·26	7·88	10·78
17	2·89	4·97	7·45	10·22
18	2·73	4·70	7·07	9·71
19	0·00	4·47	6·72	9·26
20	0·00	4·25	6·41	8·84
21	0·00	4·06	6·12	8·46
22	0·00	3·88	5·86	8·11
23	0·00	3·72	5·62	7·78
24	0·00	3·57	5·40	7·49
25	0·00	3·43	5·19	7·21
26	0·00	3·30	5·00	6·95
27	0·00	3·18	4·83	6·71
28	0·00	3·07	4·66	6·49
29	0·00	2·97	4·51	6·28
30	0·00	2·87	4·37	6·09
35	0·00	0·00	3·77	5·26
40	0·00	0·00	3·31	4·64

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.
NOTE 2 — Bar spacings below the dividing line exceed $3d$.

TABLE 26 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 250 N/mm²
Thickness = 11·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12
5	10·44	17·13	24·00	0·00
6	8·82	14·66	20·93	0·00
7	7·63	12·80	18·51	0·00
8	6·73	11·35	16·58	21·90
9	6·01	10·20	14·99	20·00
10	5·44	9·25	13·68	18·39
11	4·96	8·47	12·58	17·01
12	4·56	7·81	11·64	15·81
13	4·22	7·24	10·82	14·77
14	3·93	6·75	10·12	13·85
15	3·67	6·32	9·50	13·04
16	3·45	5·95	8·95	12·31
17	3·25	5·61	8·46	11·66
18	0·00	5·31	8·02	11·08
19	0·00	5·04	7·62	10·55
20	0·00	4·80	7·26	10·07
21	0·00	4·58	6·94	9·63
22	0·00	4·38	6·64	9·22
23	0·00	4·19	6·36	8·85
24	0·00	4·02	6·11	8·51
25	0·00	3·87	5·88	8·19
26	0·00	3·72	5·66	7·90
27	0·00	3·59	5·46	7·63
28	0·00	3·46	5·27	7·37
29	0·00	3·35	5·10	7·13
30	0·00	3·24	4·94	6·91
35	0·00	0·00	4·25	5·97
40	0·00	0·00	3·74	5·25
45	0·00	0·00	3·33	4·69

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.
NOTE 2 — Bar spacings below the dividing line exceed $3d$.

f_y 250 | f_{ck} 20 | t 12 13

TABLE 27 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 250 N/mm²
Thickness = 12·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12
5	11·67	19·31	27·41	0·00
6	9·84	16·48	23·78	0·00
7	8·51	14·36	20·95	27·64
8	7·50	12·72	18·71	24·97
9	6·70	11·41	16·89	22·73
10	6·05	10·35	15·39	20·85
11	5·52	9·46	14·13	19·24
12	5·07	8·72	13·06	17·86
13	4·69	8·08	12·14	16·66
14	4·37	7·53	11·34	15·60
15	4·08	7·05	10·63	14·67
16	0·00	6·63	10·01	13·85
17	0·00	6·25	9·46	13·11
18	0·00	5·92	8·97	12·44
19	0·00	5·62	8·52	11·84
20	0·00	5·35	8·12	11·30
21	0·00	5·10	7·75	10·80
22	0·00	4·87	7·41	10·34
23	0·00	4·67	7·11	9·92
24	9·00	4·48	6·82	9·54
25	0·00	4·30	6·56	9·18
26	0·00	4·14	6·32	8·85
27	0·00	3·99	6·09	8·54
28	0·00	0·00	5·88	8·25
29	0·00	0·00	5·69	7·98
30	0·00	0·00	5·50	7·73
35	0·00	0·00	4·74	6·67
40	0·00	0·00	4·16	5·87
45	0·00	0·00	0·00	5·24

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.
NOTE 2 — Bar spacings below the dividing line exceed 3*d*.

TABLE 28 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 250 N/mm²
Thickness = 13·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	16
5	12·90	21·50	30·83	0·00	0·00
6	10·87	18·30	26·62	34·96	0·00
7	9·39	15·92	23·39	31·16	0·00
8	8·26	14·08	20·84	28·04	0·00
9	7·38	12·63	18·79	25·47	0·00
10	6·67	11·44	17·10	23·31	0·00
11	6·08	10·46	15·68	21·48	33·09
12	5·57	9·63	14·48	19·91	30·94
13	5·17	8·92	13·45	18·55	29·11
14	4·81	8·31	12·56	17·36	27·46
15	0·00	7·78	11·77	16·31	25·98
16	0·00	7·31	11·08	15·39	24·65
17	0·00	6·90	10·47	14·56	23·43
18	0·00	6·53	9·91	13·81	22·33
19	0·00	6·19	9·42	13·14	21·33
20	0·00	5·89	8·97	12·53	20·41
21	0·00	5·62	8·56	11·97	19·56
22	0·00	5·37	8·19	11·46	18·78
23	0·00	5·14	7·85	10·99	18·06
24	0·00	4·93	7·53	10·56	17·39
25	0·00	4·74	7·24	10·16	16·77
26	0·00	0·00	6·97	9·79	16·19
27	0·00	0·00	6·72	9·45	15·65
28	0·00	0·00	6·49	9·13	15·14
29	0·00	0·00	6·28	8·83	14·66
30	0·00	0·00	6·07	8·55	14·22
35	0·00	0·00	5·23	7·37	12·34
40	0·00	0·00	4·59	6·48	10·89
45	0·00	0·00	0·00	5·78	9·75

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.
NOTE 2 — Bar spacings below the dividing line exceed 3*d*.

f_y 250

f_{ck} 20

t 14 15

TABLE 29 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 250 N/mm²
Thickness = 14·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	16
5	14·12	23·68	34·24	0·00	0·00
6	11·89	20·12	29·47	39·06	0·00
7	10·27	17·48	25·83	34·67	0·00
8	9·03	15·45	22·98	31·12	0·00
9	8·06	13·84	20·69	28·20	0·00
10	7·28	12·53	18·80	25·77	39·66
11	6·64	11·45	17·23	23·71	36·97
12	6·10	10·54	15·90	21·96	34·59
13	5·64	9·76	14·76	20·44	32·47
14	0·00	9·09	13·78	19·12	30·58
15	0·00	8·51	12·91	17·95	28·90
16	0·00	7·99	12·15	16·92	27·38
17	0·00	7·54	11·47	16·00	26·01
18	0·00	7·13	10·86	15·18	24·76
19	0·00	6·77	10·32	14·43	23·63
20	0·00	6·44	9·82	13·76	22·59
21	0·00	6·14	9·37	13·14	21·64
22	0·00	5·87	8·97	12·58	20·77
23	0·00	5·62	8·59	12·06	19·96
24	0·00	0·00	8·24	11·58	19·21
25	0·00	0·00	7·93	11·14	18·51
26	0·00	0·00	7·63	10·74	17·87
27	0·00	0·00	7·36	10·36	17·26
28	0·00	0·00	7·10	10·00	16·70
29	0·00	0·00	6·86	9·67	16·17
30	0·00	0·00	6·64	9·36	15·67
35	0·00	0·00	5·72	8·07	13·59
40	0·00	0·00	0·00	7·10	11·99
45	0·00	0·00	0·00	6·33	10·72

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.

NOTE 2 — Bar spacings below the dividing line exceed $3d$.

TABLE 30 FLEXURE—MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 250 N/mm²
Thickness = 15·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	16	18
5	15·35	25·87	37·65	49·46	0·00	0·00
6	12·92	21·94	32·31	43·16	0·00	0·00
7	11·15	19·04	28·27	38·18	3·00	0·00
8	9·80	16·82	25·11	34·19	3·00	0·00
9	8·75	15·05	22·58	30·93	0·00	0·00
10	7·90	13·62	20·51	28·22	44·04	0·00
11	7·20	12·44	18·79	25·95	40·94	0·00
12	6·61	11·45	17·33	24·00	38·23	44·42
13	0·00	10·60	16·08	22·33	35·83	41·88
14	0·00	9·87	14·99	20·87	33·71	39·58
15	0·00	9·24	14·05	19·59	31·81	37·50
16	0·00	8·68	13·21	18·46	30·11	35·62
17	0·00	8·18	12·47	17·45	28·58	33·91
18	0·00	7·74	11·81	16·54	27·19	32·34
19	0·00	7·34	11·22	15·73	25·93	30·91
20	0·00	6·98	10·68	14·99	24·78	29·60
21	0·00	6·66	10·19	14·31	23·72	28·39
22	0·00	6·36	9·74	13·69	22·75	27·28
23	0·00	0·00	9·33	13·13	21·86	26·24
24	0·00	0·00	8·96	12·61	21·03	25·28
25	0·00	0·00	8·61	12·13	20·26	24·39
26	0·00	0·00	8·29	11·68	19·55	23·55
27	0·00	0·00	7·99	11·27	18·88	22·78
28	0·00	0·00	7·71	10·88	18·26	22·05
29	0·00	0·00	7·45	10·52	17·68	21·36
30	0·00	0·00	7·21	10·18	17·13	20·72
35	0·00	0·00	0·00	8·78	14·83	18·00
40	0·00	0·00	0·00	7·71	13·08	15·91
45	0·00	0·00	0·00	6·88	11·69	14·25

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.

NOTE 2 — Bar spacings below the dividing line exceed $3d$.

TABLE 31 FLEXURE—MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 250 N/mm²
Thickness = 17·5 cm

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12	16	18	20
5	18·43	31·33	46·19	61·76	0·00	0·00	0·00
6	15·48	26·49	39·43	53·40	0·00	0·00	0·00
7	13·34	22·94	34·37	46·96	0·00	0·00	0·00
8	11·72	20·23	30·45	41·87	65·25	0·00	0·00
9	10·45	18·09	27·33	37·76	59·71	0·00	0·00
10	9·43	16·36	24·78	34·37	54·96	64·19	0·00
11	0·00	14·93	22·67	31·53	50·88	59·82	0·00
12	0·00	13·73	20·88	29·13	47·34	55·95	0·00
13	0·00	12·70	19·36	27·06	44·24	52·52	60·23
14	0·00	11·82	18·04	25·26	41·51	49·46	56·99
15	0·00	11·06	16·89	23·69	39·09	46·72	54·04
16	0·00	10·38	15·88	22·30	36·94	44·26	51·57
17	0·00	9·79	14·98	21·06	35·00	42·04	48·93
18	0·00	9·26	14·18	19·96	33·26	40·03	46·70
19	0·00	8·78	13·46	18·96	31·68	38·19	44·66
20	0·00	0·00	12·81	18·06	30·24	36·52	42·78
21	0·00	0·00	12·22	17·24	28·93	34·98	41·05
22	0·00	0·00	11·68	16·49	27·72	33·56	39·45
23	0·00	0·00	11·19	15·80	26·61	32·25	37·96
24	0·00	0·00	10·73	15·17	25·58	31·04	36·58
25	0·00	0·00	10·32	14·59	24·63	29·92	35·30
26	0·00	0·00	9·93	14·05	23·75	28·87	34·10
27	0·00	0·00	9·57	13·54	22·93	27·90	32·98
28	0·00	0·00	9·24	13·08	22·16	26·99	31·93
29	0·00	0·00	8·93	12·64	21·45	26·13	30·94
30	0·00	0·00	0·00	12·23	20·77	25·33	30·02
35	0·00	0·00	0·00	10·53	17·96	21·95	26·09
40	0·00	0·00	0·00	9·25	15·81	19·36	23·07
45	0·00	0·00	0·00	0·00	14·12	17·32	20·68

Note — Zeros indicate inadmissible reinforcement percentage.

f_y 250

f_{ck} 20

t 20

TABLE 32 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 250 N/mm²
Thickness = 20·0 cm

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12	16	18	20	22	25
5	21·50	36·80	54·73	74·05	0·00	0·00	0·00	0·00	0·00
6	18·04	31·05	46·54	63·65	0·00	0·00	0·00	0·00	0·00
7	15·34	26·85	40·47	55·74	87·37	0·00	0·00	0·00	0·00
8	13·64	23·65	35·78	49·55	78·91	0·00	0·00	0·00	0·00
9	12·16	21·12	32·07	44·59	71·85	84·51	0·00	0·00	0·00
10	0·00	19·09	29·05	40·52	65·89	78·02	0·00	0·00	0·00
11	0·00	17·41	26·55	37·12	60·81	72·39	83·28	0·00	0·00
12	0·00	16·00	24·44	34·25	56·44	67·47	78·04	0·00	0·00
13	0·00	14·81	22·64	31·79	52·64	63·16	73·36	82·81	0·00
14	0·00	13·78	21·09	29·63	49·32	59·34	69·18	78·44	0·00
15	0·00	12·88	19·74	27·79	46·38	55·94	65·42	74·47	0·00
16	0·00	12·09	18·55	26·14	43·77	52·91	62·04	70·84	0·00
17	0·00	0·00	17·50	24·68	41·43	50·18	58·97	67·53	0·00
18	0·00	0·00	16·55	23·37	39·33	47·71	56·18	64·49	76·03
19	0·00	0·00	15·71	22·20	37·43	45·47	53·64	61·70	73·04
20	0·00	0·00	14·95	21·13	35·70	43·43	51·31	59·14	70·25
21	0·00	0·00	14·25	20·17	34·13	41·56	49·18	56·77	67·66
22	0·00	0·00	13·62	19·28	32·69	39·85	47·21	54·58	65·23
23	0·00	0·00	13·04	18·47	31·36	38·27	45·39	52·54	62·96
24	0·00	0·00	12·51	17·73	30·14	36·81	43·70	50·65	60·83
25	0·00	0·00	12·02	17·05	29·01	35·45	42·13	48·89	58·84
26	0·00	0·00	11·57	16·41	27·95	34·19	40·67	47·24	56·96
27	0·00	0·00	0·00	15·82	26·98	33·02	39·30	45·70	55·20
28	0·00	0·00	0·00	15·27	26·07	31·92	38·03	44·25	53·54
29	0·00	0·00	0·00	14·76	25·21	30·90	36·83	42·90	51·97
30	0·00	0·00	0·00	14·28	24·42	29·94	35·71	41·62	50·49
35	0·00	0·00	0·00	12·29	21·08	25·90	30·97	36·21	44·17
40	0·00	0·00	0·00	0·00	18·54	22·82	27·34	32·04	39·24
45	0·00	0·00	0·00	0·00	16·53	20·39	24·47	28·72	35·29

Note — Zeros indicate inadmissible reinforcement percentage.

f_y 250

f_{ck} 20

t 22.5

TABLE 33 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 250 N/mm²
Thickness — 22·5 cm

Bar Spacing, cm	Bar Diameter, mm								
	6	8	10	12	16	18	20	22	25
5	24·57	42·26	63·27	86·34	0·00	0·00	0·00	0·00	0·00
6	20·60	35·60	53·66	73·89	115·76	0·00	0·00	0·00	0·00
7	17·73	30·75	46·56	64·52	102·98	0·00	0·00	0·00	0·00
8	15·56	27·06	41·12	57·24	92·57	109·29	0·00	0·00	0·00
9	0·00	24·16	36·81	51·42	83·99	99·87	0·00	0·00	0·00
10	0·00	21·82	33·32	46·66	76·82	91·85	106·23	0·00	0·00
11	0·00	19·89	30·43	42·71	70·75	84·96	98·80	0·00	0·00
12	0·00	18·28	28·00	39·37	65·55	79·00	92·27	104·82	0·00
13	0·00	16·91	25·93	36·52	61·05	73·79	86·50	98·70	0·00
14	0·00	15·73	24·14	34·04	57·12	69·22	81·38	93·20	0·00
15	0·00	0·00	22·59	31·89	53·66	65·16	76·81	88·24	104·15
16	0·00	0·00	21·22	29·98	50·60	61·55	72·71	83·75	99·35
17	0·00	0·00	20·01	28·30	47·86	58·31	69·01	79·68	94·93
18	0·00	0·00	18·93	26·79	45·40	55·39	65·67	75·97	90·85
19	0·00	0·00	17·96	25·43	43·18	52·75	62·63	72·58	87·08
20	0·00	0·00	17·08	24·21	41·17	50·35	59·85	69·47	83·59
21	0·00	0·00	16·29	23·09	39·33	48·15	57·31	66·61	80·36
22	0·00	0·00	15·56	22·08	37·65	46·13	54·97	63·97	77·35
23	0·00	0·00	14·90	21·15	36·11	44·28	52·81	61·53	74·56
24	0·00	0·00	0·00	20·29	34·69	42·57	50·81	59·26	71·95
25	0·00	0·00	0·00	19·50	33·38	40·98	48·96	57·15	69·51
26	0·00	0·00	0·00	18·77	32·16	39·51	47·24	55·19	67·22
27	0·00	0·00	0·00	18·10	31·02	38·14	45·63	53·35	65·08
28	0·00	0·00	0·00	17·47	29·97	36·86	44·13	51·63	63·07
29	0·00	0·00	0·00	16·88	28·98	35·67	42·72	50·02	61·17
30	0·00	0·00	0·00	16·33	28·06	34·55	41·40	48·50	59·38
35	0·00	0·00	0·00	0·00	24·20	29·85	35·85	42·11	51·80
40	0·00	0·00	0·00	0·00	21·27	26·28	31·61	37·20	45·91
45	0·00	0·00	0·00	0·00	18·98	23·47	28·26	33·31	41·21

Note — Zeros indicate inadmissible reinforcement percentage.

TABLE 34 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 250 N/mm²
Thickness = 25·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	16	18	20	22	25
5	27·65	47·72	71·80	98·64	0·00	0·00	0·00	0·00	0·00
6	23·16	40·15	60·77	84·14	133·98	0·00	0·00	0·00	0·00
7	19·93	34·65	52·66	73·30	118·59	140·15	0·00	0·00	0·00
8	0·00	30·47	46·46	64·92	106·23	126·58	0·00	0·00	0·00
9	0·00	27·20	41·55	58·25	96·13	115·24	133·67	0·00	0·00
10	0·00	24·55	37·59	52·81	87·74	105·68	123·40	139·87	0·00
11	0·00	22·38	34·31	48·30	80·68	97·53	114·32	130·41	0·00
12	0·00	20·56	31·56	44·49	74·65	90·52	106·49	122·04	0·00
13	0·00	19·01	29·21	41·24	69·46	84·43	99·63	114·59	0·00
14	0·00	0·00	27·19	38·43	64·93	79·09	93·57	107·96	128·41
15	0·00	0·00	25·43	35·98	60·95	74·38	88·19	102·01	121·93
16	0·00	0·00	23·89	33·83	57·43	70·19	83·38	96·66	116·03
17	0·00	0·00	22·52	31·91	54·29	66·45	79·06	91·83	110·62
18	0·00	0·00	21·30	30·20	51·47	63·08	75·15	87·45	105·67
19	0·00	0·00	20·20	28·67	48·93	60·03	71·61	83·45	101·12
20	0·00	0·00	19·21	27·28	46·63	57·26	68·39	79·80	96·93
21	0·00	0·00	0·00	26·02	44·54	54·73	65·44	76·45	93·06
22	0·00	0·00	0·00	24·87	42·62	52·42	62·73	73·36	89·48
23	0·00	0·00	0·00	23·82	40·86	50·29	60·23	70·51	86·16
24	0·00	0·00	0·00	22·83	39·24	48·33	57·93	67·87	83·06
25	0·00	0·00	0·00	21·96	37·75	46·52	55·79	65·42	80·18
26	0·00	0·00	0·00	21·14	36·36	44·83	53·80	63·13	77·48
27	0·00	0·00	0·00	20·37	35·07	43·26	51·95	61·00	74·96
28	0·00	0·00	0·00	19·66	33·87	41·80	50·22	59·01	72·59
29	0·00	0·00	0·00	19·00	32·75	40·44	48·61	57·14	70·37
30	0·00	0·00	0·00	18·38	31·70	39·16	47·09	55·39	68·28
35	0·00	0·00	0·00	0·00	27·32	33·80	40·73	48·01	59·42
40	0·00	0·00	0·00	0·00	24·01	29·74	35·88	42·36	52·58
45	0·00	0·00	0·00	0·00	21·41	26·54	32·06	37·90	47·14

NOTE — Zeros indicate inadmissible reinforcement percentage.

f_y 415 | f_{ck} 20 | t 10 11

TABLE 35 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 415 N/mm²
Thickness = 10·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12
5	14·33	0·00	0·00	0·00
6	12·27	0·00	0·00	0·00
7	10·72	17·11	0·00	0·00
8	9·52	15·40	0·00	0·00
9	8·55	13·98	0·00	0·00
10	7·77	12·79	0·00	0·00
11	7·11	11·79	16·78	0·00
12	6·55	10·92	15·67	0·00
13	6·08	10·18	14·70	0·00
14	5·67	9·52	13·83	0·00
15	5·31	8·95	13·05	17·22
16	4·99	8·44	12·36	16·39
17	4·71	7·99	11·73	15·64
18	4·46	7·58	11·16	14·94
19	4·24	7·21	10·65	14·30
20	4·03	6·87	10·18	13·72
21	3·85	6·57	9·74	13·17
22	3·68	6·29	9·35	12·67
23	3·52	6·03	8·98	12·20
24	0·00	5·79	8·64	11·76
25	0·00	5·57	8·33	11·36
26	0·00	5·37	8·03	10·98
27	0·00	5·18	7·76	10·62
28	0·00	5·00	7·51	10·29
29	0·00	4·84	7·27	9·97
30	0·00	4·69	7·04	9·68
35	0·00	4·04	6·10	8·43
40	0·00	3·55	5·38	7·46

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.
NOTE 2 — Bar spacings below the dividing line exceed 3*d*.

TABLE 36 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 415 N/mm²
Thickness = 11·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12
5	16·37	0·00	0·00	0·00
6	13·97	22·23	0·00	0·00
7	12·18	19·70	0·00	0·00
8	10·79	17·66	0·00	0·00
9	9·69	15·99	0·00	0·00
10	8·79	14·61	20·87	0·00
11	8·04	13·44	19·35	0·00
12	7·40	12·44	18·03	0·00
13	6·86	11·57	16·88	0·00
14	6·40	10·82	15·85	21·04
15	5·99	20·16	14·94	19·94
16	5·63	9·57	14·13	18·94
17	5·31	9·05	13·40	18·04
18	5·03	8·50	12·74	17·21
19	4·77	8·16	12·14	16·45
20	4·54	7·78	11·59	15·76
21	4·33	7·43	11·09	15·11
22	0·00	7·11	10·64	14·52
23	0·00	6·82	10·21	13·97
24	0·00	6·55	9·82	13·46
25	0·00	6·30	9·46	12·99
26	0·00	6·07	9·12	12·55
27	0·00	5·85	8·81	12·13
28	0·00	5·65	8·52	11·75
29	0·00	5·47	8·24	11·38
30	0·00	5·29	7·99	11·04
35	0·00	4·56	6·91	9·59
40	0·00	0·00	6·09	8·48
45	0·00	0·00	5·44	7·60

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.
NOTE 2 — Bar spacings below the dividing line exceed 3*d*.

TABLE 37 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 415 N/mm²
Thickness = 12·0 cm

Bar Spacing, cm	Bar Diameter, mm			
	6	8	10	12
5	18·41	0·00	0·00	0·00
6	15·67	25·25	0·00	0·00
7	13·64	22·29	0·00	0·00
8	12·07	19·93	0·00	0·00
9	10·32	18·01	25·76	0·00
10	9·81	16·42	23·70	0·00
11	8·96	15·08	21·93	0·00
12	8·26	13·95	20·40	26·99
13	7·65	12·97	19·06	25·39
14	7·13	12·12	17·88	23·95
15	6·67	11·37	16·83	22·66
16	6·27	10·71	15·90	21·49
17	5·91	10·12	15·07	20·44
18	5·60	9·59	14·31	19·48
19	5·31	9·12	13·63	18·60
20	0·00	8·69	13·01	17·80
21	0·00	8·29	12·44	17·06
22	0·00	7·93	11·92	16·38
23	0·00	7·61	11·45	15·75
24	0·00	7·30	11·00	15·16
25	0·00	7·02	10·59	14·62
26	0·00	6·77	10·21	14·12
27	0·00	6·52	9·86	13·64
28	0·00	6·30	9·53	13·20
29	0·00	6·09	9·22	12·79
30	0·00	5·90	8·93	12·40
35	0·00	0·00	7·72	10·76
40	0·00	0·00	6·80	9·50
45	0·00	0·00	6·07	8·50

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.
NOTE 2 — Bar spacings below the dividing line exceed 3*d*.

TABLE 38 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 415 N/mm²
Thickness = 13·0 cm

Bar Spacing, cm	Bar Diameter, mm				
	6	8	10	12	16
5	20·45	32·66	0·00	0·00	0·00
6	17·38	28·28	0·00	0·00	0·00
7	15·10	24·88	0·00	0·00	0·00
8	13·34	22·20	31·72	0·00	0·00
9	11·95	20·02	28·91	0·00	0·00
10	10·83	18·23	26·54	0·00	0·00
11	9·89	16·73	24·51	32·49	0·00
12	9·11	15·46	22·76	30·39	0·00
13	8·43	14·36	21·24	28·53	0·00
14	7·86	13·41	19·90	26·87	0·00
15	7·35	12·58	18·72	25·38	0·00
16	6·91	11·84	17·67	24·04	0·00
17	6·51	11·19	16·73	22·84	0·00
18	6·16	10·60	15·89	21·74	0·00
19	0·00	10·07	15·12	20·75	0·00
20	0·00	9·59	14·43	19·84	30·85
21	0·00	9·16	13·79	19·00	29·73
22	0·00	8·76	13·21	18·23	28·67
23	0·00	8·39	12·68	17·52	27·69
24	0·00	8·06	12·18	16·87	26·77
25	0·00	7·75	11·73	16·25	25·90
26	0·00	7·46	11·30	15·69	25·08
27	0·00	7·20	10·91	15·16	24·31
28	0·00	6·95	10·54	14·66	23·59
29	0·00	6·72	10·20	14·20	22·90
30	0·00	6·50	9·88	13·76	22·26
35	0·00	0·00	8·53	11·92	19·49
40	0·00	0·00	7·50	10·52	17·33
45	0·00	0·00	6·70	9·41	15·59

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.
NOTE 2 — Bar spacings below the dividing line exceed 3*d*.

f_y 415

f_{ck} 20

t 14

TABLE 39 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 415 N/mm²
Thickness = 14·0 cm

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12	16
5	22·49	36·29	0·00	0·00	0·00
6	19·08	31·30	0·00	0·00	0·00
7	16·36	27·48	39·12	0·00	0·00
8	14·62	24·47	35·26	0·00	0·00
9	13·09	22·04	32·06	0·00	0·00
10	11·85	20·05	29·37	38·94	0·00
11	10·82	18·38	27·08	36·20	0·00
12	9·96	16·97	25·12	33·79	0·00
13	9·22	15·76	23·42	31·67	0·00
14	8·58	14·71	21·93	29·78	0·00
15	8·03	13·79	20·61	28·10	0·00
16	7·55	12·98	19·44	26·60	0·00
17	0·00	12·25	18·40	25·24	0·00
18	0·00	11·61	17·46	24·01	0·00
19	0·00	11·03	16·61	22·90	35·87
20	0·00	10·50	15·85	21·88	34·48
21	0·00	10·02	15·14	20·95	33·18
22	0·00	9·58	14·50	20·09	31·97
23	0·00	9·18	13·91	19·30	30·84
24	0·00	8·82	13·37	18·57	29·79
25	0·00	8·48	12·86	17·89	28·80
26	0·00	8·16	12·39	17·26	27·87
27	0·00	7·87	11·96	16·67	27·00
28	0·00	7·60	11·55	16·12	26·18
29	0·00	7·34	11·18	15·60	25·41
30	0·00	0·00	10·82	15·12	24·68
35	0·00	0·00	9·34	13·09	21·56
40	0·00	0·00	8·21	11·54	19·14
45	0·00	0·00	7·33	10·32	17·20

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.

NOTE 2 — Bar spacings below the dividing line exceed 3*d*.

TABLE 40 FLEXURE—MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 415 N/mm²
Thickness = 15·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	16	18
5	24·53	39·92	0·00	0·00	0·00	0·00
6	20·78	34·32	0·00	0·00	0·00	0·00
7	18·01	30·07	43·16	0·00	0·00	0·00
8	15·90	26·73	38·80	0·00	0·00	0·00
9	14·22	24·06	35·21	0·00	0·00	0·00
10	12·87	21·86	32·20	43·03	0·00	0·00
11	11·75	20·03	29·66	39·91	0·00	0·00
12	10·81	18·48	27·48	37·19	0·00	0·00
13	10·00	17·15	25·60	34·80	0·00	0·00
14	9·31	16·00	23·95	32·70	0·00	0·00
15	8·71	15·00	22·50	30·82	0·00	0·00
16	0·00	14·11	21·22	29·15	0·00	0·00
17	0·00	13·32	30·07	27·64	43·25	0·00
18	0·00	12·61	19·04	26·28	41·40	0·00
19	0·00	11·98	18·11	25·04	39·69	0·00
20	0·00	11·41	17·26	23·92	38·11	0·00
21	0·00	10·88	16·49	22·89	36·64	0·00
22	0·00	10·41	15·79	21·94	35·27	41·27
23	0·00	9·97	15·14	21·07	34·00	39·90
24	0·00	9·57	14·55	20·27	32·81	38·60
25	0·00	9·20	14·00	19·52	31·70	37·38
26	0·00	8·86	13·48	18·83	30·66	36·24
27	0·00	8·54	13·01	18·18	29·69	35·15
28	0·00	0·00	12·57	17·58	28·77	34·13
29	0·00	0·00	12·15	17·01	27·91	33·16
30	0·00	0·00	11·77	16·48	27·09	32·24
35	0·00	0·00	10·15	14·26	23·64	28·29
40	0·00	0·00	8·92	12·56	20·95	25·19
45	0·00	0·00	0·00	11·22	18·81	22·69

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.

NOTE 2 — Bar spacings below the dividing line exceed 3*d*.

f_y 415 | f_{ck} 20 | t 17.5

TABLE 41 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 415 N/mm²
Thickness — 17·5 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	16	18	20
5	29·63	48·99	0·00	0·00	0·00	0·00	0·00
6	25·03	41·88	60·33	0·00	0·00	0·00	0·00
7	21·66	36·55	53·29	0·00	0·00	0·00	0·00
8	19·08	32·40	47·66	63·53	0·00	0·00	0·00
9	17·06	29·09	45·08	57·95	0·00	0·00	0·00
10	15·42	26·40	39·29	53·23	0·00	0·00	0·00
11	14·07	24·15	36·10	49·18	0·00	0·00	0·00
12	12·93	22·26	33·39	45·69	0·00	0·00	0·00
13	11·97	20·64	31·05	42·65	0·00	0·00	0·00
14	0·00	19·24	29·01	39·98	62·74	0·00	0·00
15	0·00	18·02	27·22	37·62	59·52	0·00	0·00
16	0·00	16·94	25·64	35·52	56·39	0·00	0·00
17	0·00	15·99	24·24	33·64	53·92	0·00	0·00
18	0·00	15·13	22·97	31·95	51·48	60·47	0·00
19	0·00	14·37	21·84	30·41	49·24	58·03	0·00
20	0·00	13·67	20·81	29·02	47·18	55·77	0·00
21	0·00	13·04	19·87	27·75	45·27	53·67	0·00
22	0·00	12·47	19·01	26·58	43·52	51·71	0·00
23	0·00	11·94	18·22	25·51	41·89	49·88	57·43
24	0·00	0·00	17·50	24·52	40·37	48·17	55·60
25	0·00	0·00	16·83	23·60	38·96	46·57	53·87
26	0·00	0·00	16·21	22·75	37·64	45·07	52·24
27	0·00	0·00	15·63	21·96	36·41	43·65	50·70
28	0·00	0·00	15·10	21·22	35·25	42·32	49·24
29	0·00	0·00	14·60	20·53	34·16	41·07	47·86
30	0·00	0·00	14·13	19·88	33·14	39·89	46·54
35	0·00	0·00	12·17	17·17	28·82	34·85	40·91
40	0·00	0·00	0·00	15·11	25·49	30·93	36·46
45	0·00	0·00	0·00	13·49	22·84	27·80	32·86

NOTE — Zeros indicate inadmissible reinforcement percentage.

f_y 415

f_{ck} 20

t 20

TABLE 42 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 415 N/mm²
Thickness = 20·0 cm

Bar Spacing, cm	Bar Diameter, mm								
	6	8	10	12	16	18	20	22	25
5	34·73	58·06	83·47	0·00	0·00	0·00	0·00	0·00	0·00
6	29·28	49·44	72·14	0·00	0·00	0·00	0·00	0·00	0·00
7	25·30	43·02	63·41	84·72	0·00	0·00	0·00	0·00	0·00
8	22·27	38·07	56·52	76·28	0·00	0·00	0·00	0·00	0·00
9	19·89	34·13	50·96	69·29	0·00	0·00	0·00	0.00	0·00
10	17·97	30·93	46·38	63·43	0·00	0·00	0·00	0·00	0·00
11	16·38	28·28	42·54	58·46	0·00	0·00	0·00	0·00	0·00
12	0·00	26·04	39·29	54·20	85·29	0·00	0·00	0·00	0·00
13	0·00	24·13	36·50	50·50	80·23	0·00	0·00	0·00	0·00
14	0·00	22·48	34·07	47·27	75·70	0·00	0·00	0·00	0·00
15	0·00	21·04	31·95	44·43	71·62	0·00	0·00	0·00	0·00
16	0·00	19·78	30·07	41·90	67·93	80·26	0·00	0·00	0·00
17	0·00	18·66	28·40	39·64	64·59	76·59	0·00	0·00	0·00
18	0·00	17·65	26·91	37·62	61·56	73·22	0·00	0·00	0·00
19	0·00	16·75	25·57	35·78	58·79	70·12	0·00	0·00	0·00
20	0·00	15·94	24·35	34·12	56·25	67·25	77·80	0·00	0·00
21	0·00	0·00	23·24	32·61	53·91	64·60	74·94	0·00	0·00
22	0·00	0·00	22·23	31·22	51·76	62·15	72·26	0·00	0·00
23	0·00	0·00	21·30	29·94	49·77	59·86	69·76	0·00	0·00
24	0·00	0·00	20·45	28·77	47·93	57·74	67·41	76·58	0·00
25	0·00	0·00	19·66	27·68	46·21	55·75	65·21	74·24	0·00
26	0·00	0·00	18·94	26·67	44·62	53·89	63·14	72·03	0·00
27	0·00	0·00	18·26	25·74	43·12	52·16	61·19	69·93	0·00
28	0·00	0·00	17·63	24·86	41·73	50·52	59·36	67·95	0·00
29	0·00	0·00	17·04	24·05	40·42	48·99	57·63	66·07	0·00
30	0·00	0·00	16·49	23·28	39·19	47·54	55·99	64·28	0·00
35	0·00	0·00	0·00	20·09	34·00	41·41	49·00	56·58	67·44
40	0·00	0·00	0·00	17·66	30·02	36·67	43·54	50·48	60·63
45	0·00	0·00	0·00	15·76	26·88	32·90	39·16	45·54	55·01

Note — Zeros indicate inadmissible reinforcement percentage.

f_y 415 | f_{ck} 20 | t 22.

TABLE 43 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 415 N/mm²
Thickness = 22·5 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	16	18	20	22	25
5	39·84	67·13	97·64	0·00	0·00	0·00	0·00	0·00	0·00
6	33·53	57·00	83·94	112·03	0·00	0·00	0·00	0·00	0·00
7	28·95	49·50	73·53	99·30	0·00	0·00	0·00	0·00	0·00
8	25·46	43·74	65·38	89·04	0·00	0·00	0·00	0·00	0·00
9	22·73	39·17	58·83	80·63	0·00	0·00	0·00	0·00	0·00
10	20·52	35·47	53·46	73·64	0·00	0·00	0·00	0·00	0·00
11	0·00	32·40	48·98	67·74	107·44	0·00	0·00	0·00	0·00
12	0·00	29·82	45·20	62·70	100·41	0·00	0·00	0·00	0·00
13	0·00	27·62	41·95	58·35	94·19	0·00	0·00	0·00	0·00
14	0·00	25·72	39·13	34·36	88·65	105·02	0·00	0·00	0·00
15	0·00	24·07	36·67	51·23	83·71	99·56	0·00	0·00	0·00
16	0·00	22·61	34·50	48·28	79·27	94·61	0·00	0·00	0·00
17	0·00	21·32	32·57	45·65	75·26	90·10	104·35	0·00	0·00
18	0·00	20·17	30·85	43·28	71·63	85·98	99·90	0·00	0·00
19	0·00	0·00	29·29	41·15	68·33	82·20	95·78	0·00	0·00
20	0·00	0·00	27·89	39·22	65·32	78·73	91·97	0·00	0·00
21	0·00	0·00	26·62	37·46	62·55	75·53	88·43	100·77	0·00
22	0·00	0·00	25·45	35·86	60·01	72·58	85·14	97·26	0·00
23	0·00	0·00	24·38	34·38	57·66	69·84	82·08	93·96	0·00
24	0·00	0·00	23·40	33·02	55·49	67·30	79·22	90·87	0·00
25	0·00	0·00	22·50	31·76	53·47	64·93	76·55	87·96	0·00
26	0·00	0·00	21·66	30·60	51·59	62·72	74·04	85·22	0·00
27	0·00	0·00	20·88	29·52	49·84	60·66	71·69	82·63	0·00
28	0·00	0·00	20·16	28·51	48·21	58·72	69·48	80·20	95·50
29	0·00	0·00	19·48	27·57	46·67	56·90	67·40	77·89	92·97
30	0·00	0·00	0·00	26·68	45·23	55·19	65·44	75·71	90·57
35	0·00	0·00	0·00	23·00	39·19	47·97	57·10	66·38	80·10
40	0·00	0·00	0·00	20·21	34·36	42·41	50·63	59·05	71·70
45	0·00	0·00	0·00	0·00	30·91	38·00	45·46	53·16	64·86

NOTE — Zeros indicate inadmissible reinforcement percentage.

f_y 415

f_{ck} 20

t 25

TABLE 44 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 415 N/mm²
Thickness = 25·0 cm

Bar Spacing, cm	Bar Diameter, mm								
	6	8	10	12	16	18	20	22	25
5	44·94	76·20	111·81	0·00	0·00	0·00	0·00	0·00	0·00
6	37·78	64·56	95·75	129·04	0·00	0·00	0·00	0·00	0·00
7	32·59	55·98	83·65	113·88	0·00	0·00	0·00	0·00	0·00
8	28·65	49·41	74·23	101·79	0·00	0·00	0·00	0·00	0·00
9	25·56	44·21	66·70	91·97	0·00	0·00	0·00	0·00	0·00
10	0·00	40·00	60·55	83·84	133·56	0·00	0·00	0·00	0·00
11	0·00	36·52	55·43	77·01	123·93	0·00	0·00	0·00	0·00
12	0·00	33·60	51·10	71·20	115·53	136·82	0·00	0·00	0·00
13	0·00	31·11	47·40	66·20	108·14	128·69	0·00	0·00	0·00
14	0·00	28·96	44·19	61·85	101·61	121·41	0·00	0·00	0·00
15	0·00	27·09	41·40	58·03	95·80	114·87	133·27	0·00	0·00
16	0·00	25·45	38·93	54·65	90·61	108·96	126·88	0·00	0·00
17	0·00	0·00	36·74	51·65	85·93	103·60	121·02	0·00	0·00
18	0·00	0·00	34·78	48·95	81·71	98·73	115·64	0·00	0·00
19	0·00	0·00	33·02	46·52	77·88	94·28	110·70	126·55	0·00
20	0·00	0·00	31·43	44·33	74·39	90·21	106·14	121·66	0·00
21	0·00	0·00	29·99	42·32	71·19	86·47	101·93	117·10	0·00
22	0·00	0·00	28·67	40·49	68·25	83·02	98·03	112·84	0·00
23	0·00	0·00	27·47	38·82	65·55	79·83	94·40	108·87	0·00
24	0·00	0·00	26·36	37·27	63·04	76·87	91·03	105·16	0·00
25	0·00	0·00	25·33	35·85	60·73	74·12	87·88	101·67	121·56
26	0·00	0·00	24·39	34·52	58·57	71·55	84·94	98·41	117·96
27	0·00	0·00	0·00	33·30	56·56	69·16	82·19	95·34	114·55
28	0·00	0·00	0·00	32·15	54·68	66·92	79·60	92·45	111·31
29	0·00	0·00	0·00	31·08	52·93	64·82	77·18	89·72	108·24
30	0·00	0·00	0·00	30·08	51·28	62·85	74·89	87·15	105·33
35	0·00	0·00	0·00	25·92	44·37	54·53	65·20	76·18	92·75
40	0·00	0·00	0·00	0·00	39·09	48·15	57·71	67·62	82·78
45	0·00	0·00	0·00	0·00	34·94	43·10	51·76	60·78	74·70

NOTE — Zeros indicate inadmissible reinforcement percentage.

TABLE 45 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

f_{ck} = 15 N/mm²
f_y = 250 N/mm²

M_u/bd^2, N/mm²	d'/d = 0·05		d'/d = 0·10		d'/d = 0·15		d'/d = 0·20	
	p_t	p_c	p_t	p_c	p_t	p_c	p_t	p_c
2·24	1·322	0·002	1·322	0·003	1·323	0·003	1·323	0·003
2·25	1·327	0·007	1·328	0·008	1·328	0·008	1·328	0·009
2·30	1·351	0·032	1·353	0·034	1·355	0·036	1·357	0·039
2·35	1·376	0·057	1·379	0·061	1·382	0·064	1·386	0·068
2·40	1·400	0·082	1·404	0·087	1·409	0·092	1·415	0·098
2·45	1·424	0·107	1·430	0·113	1·436	0·120	1·443	0·128
2·50	1·448	0·132	1·455	0·140	1·463	0·148	1·472	0·157
2·55	1·472	0·157	1·481	0·166	1·490	0·176	1·501	0·187
2·60	1·497	0·182	1·506	0·192	1·517	0·204	1·530	0·217
2·65	1·521	0·207	1·532	0·219	1·544	0·232	1·558	0·246
2·70	1·545	0·232	1·558	0·245	1·571	0·260	1·587	0·276
2·75	1·569	0·257	1·583	0·272	1·599	0·288	1·616	0·305
2·80	1·593	0·282	1·609	0·298	1·626	0·315	1·645	0·335
2·85	1·618	0·307	1·634	0·324	1·653	0·343	1·673	0·365
2·90	1·642	0·332	1·660	0·351	1·680	0·371	1.702	0·394
2·95	1·666	0·357	1·685	0·377	1·707	0·399	1·731	0·424
3·00	1·690	0·382	1·711	0·403	1·734	0·427	1·760	0·454
3·05	1·714	0·407	1·736	0·430	1·761	0·455	1·788	0·483
3·10	1·739	0·432	1·762	0·456	1·788	0·483	1·817	0·513
3·15	1·763	0·457	1·788	0·482	1·815	0·511	1·846	0·543
3·20	1·787	0·482	1·813	0·509	1·842	0·539	1·875	0·572
3·25	1·811	0·507	1·839	0·535	1·869	0·567	1·903	0·602
3·30	1·836	0·532	1·864	0·562	1·896	0·595	1·932	0·632
3·35	1·860	0·557	1·890	0·588	1·923	0·623	1·961	0·661
3·40	1·884	0·582	1·915	0·614	1·950	0·650	1·990	0·691
3·45	1·908	0·607	1·941	0·641	1·977	0·678	2·018	0·721
3·50	1·932	0·632	1·966	0·667	2·004	0·706	2·047	0·750
3·55	1·957	0·657	1·992	0·693	2·031	0·734	2·076	0·780
3·60	1·981	0·682	2·018	0·720	2·059	0·762	2·105	0·810
3·65	2·005	0·707	2·043	0·746	2·086	0·790	2·133	0·839
3·70	2·029	0·732	2·069	0·773	2·113	0·818	2·162	0·869
3·75	2·053	0·757	2·094	0·799	2·140	0·846	2·191	0·899
3·80	2·078	0·782	2·120	0·825	2·167	0·874	2·220	0·928
3·85	2·102	0·807	2·145	0·852	2·194	0·902	2·248	0·958
3·90	2·126	0·832	2·171	0·878	2·221	0·930	2·277	0·988
3·95	2·150	0·857	2·196	0·904	2·248	0·958	2·306	1·017
4·00	2·174	0·882	2·222	0·931	2·275	0·985	2·335	1·047
4·05	2·199	0·907	2·248	0·957	2·302	1·013	2·363	1·077
4·10	2·223	0·932	2·273	0·983	2·329	1·041	2·392	1·106
4·15	2·247	0·957	2·299	1·010	2·356	1·069	2·421	1·136
4·20	2·271	0·982	2·324	1·036	2·383	1·097	2·450	1·166
4·25	2·296	1·007	2·350	1·063	2·410	1·125	2·478	1·195
4·30	2·320	1·032	2·375	1·089	2·437	1·153	2·507	1·225
4·35	2·344	1·057	2·401	1·115	2·464	1·181	2·536	1·255
4·40	2·368	1·082	2·426	1·142	2·491	1·209	2·565	1·284

TABLE 46 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

f_{ck} = 20 N/mm²
f_y = 250 N/mm²

M_u/bd^2, N/mm²	d'/d = 0·05		d'/d = 0·10		d'/d = 0·15		d'/d = 0·20	
	p_t	p_c	p_t	p_c	p_t	p_c	p_t	p_c
2·99	1·765	0·005	1·765	0·005	1·765	0·006	1·766	0·006
3·00	1·769	0·010	1·770	0·011	1·771	0·011	1·771	0·012
3·05	1·794	0·035	1·796	0·037	1·798	0·039	1·800	0·042
3·10	1·818	0·061	1·821	0·064	1·825	0·068	1·829	0·072
3·15	1·842	0·086	1·847	0·091	1·852	0·096	1·858	0·102
3·20	1·866	0·111	1·872	0·117	1·879	0·124	1·886	0·132
3·25	1·891	0·136	1·898	0·144	1·906	0·152	1·915	0·162
3·30	1·915	0·162	1·923	0·171	1·933	0·181	1·944	0·192
3·35	1·939	0·187	1·949	0·197	1·960	0·209	1·973	0·222
3·40	1·963	0·212	1·974	0·224	1·987	0·237	2·001	0·252
3·45	1·987	0·237	2·000	0·250	2·014	0·265	2·030	0·282
3·50	2·012	0·263	2·026	0·277	2·041	0·293	2·059	0·312
3·55	2·036	0·288	2·051	0·304	2·068	0·322	2·088	0·342
3·60	2·060	0·313	2·077	0·330	2·095	0·350	2·116	0·372
3·65	2·084	0·338	2·102	0·357	2·122	0·378	2·145	0·402
3·70	2·108	0·364	2·128	0·384	2·149	0·406	2·174	0·432
3·75	2·133	0·389	2·153	0·410	2·177	0·434	2·203	0·462
3·80	2·157	0·414	2·179	0·437	2·204	0·463	2·231	0·492
3·85	2·181	0·439	2·204	0·464	2·231	0·491	2·260	0·522
3·90	2·205	0·464	2·230	0·490	2·258	0·519	2·289	0·552
3·95	2·229	0·490	2·256	0·517	2·285	0·547	2·318	0·582
4·00	2·254	0·515	2·281	0·544	2·312	0·576	2·346	0·612
4·05	2·278	0·540	2·307	0·570	2·339	0·604	2·375	0·642
4·10	2·302	0·565	2·332	0·597	2·366	0·632	2·404	0·672
4·15	2·326	0·591	2·358	0·624	2·393	0·660	2·433	0·701
4·20	2·351	0·616	2·383	0·650	2·420	0·688	2·461	0·731
4·25	2·375	0·641	2·409	0·677	2·447	0·717	2·490	0·761
4·30	2·399	0·666	2·434	0·703	2·474	0·745	2·519	0·791
4·35	2·423	0·692	2·460	0·730	2·501	0·773	2·548	0·821
4·40	2·447	0·717	2·486	0·757	2·528	0·801	2·576	0·851
4·45	2·472	0·742	2·511	0·782	2·555	0·830	2·605	0·881
4·50	2·496	0·767	2·537	0·810	2·582	0·858	2·634	0·911
4·55	2·520	0·793	2·562	0·837	2·609	0·886	2·663	0·941
4·60	2·544	0·818	2·588	0·863	2·637	0·914	2·691	0·971
4·65	2·568	0·843	2·613	0·890	2·664	0·942	2·720	1·001
4·70	2·593	0·868	2·639	0·917	2·691	0·971	2·749	1·031
4·75	2·617	0·894	2·664	0·943	2·718	0·999	2·778	1·061
4·80	2·641	0·919	2·690	0·970	2·745	1·027	2·806	1·091
4·85	2·665	0·944	2·716	0·997	2·772	1·055	2·835	1·121
4·90	2·689	0·969	2·741	1·023	2·799	1·083	2·864	1·151
4·95	2·714	0·995	2·767	1·050	2·826	1·112	2·893	1·181
5·00	2·738	1·020	2·792	1·077	2·853	1·140	2·921	1·211
5·05	2·762	1·045	2·818	1·103	2·880	1·168	2·950	1·241
5·10	2·786	1·070	2·843	1·130	2·907	1·196	2·979	1·271
5·15	2·811	1·096	2·869	1·157	2·934	1·225	3·008	1·301

f_y
250
f_{ck}
25

TABLE 47 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

f_{ck} = 25 N/mm²
f_y = 250 N/mm²

M_u/bd^2, N/mm²	d'/d = 0·05		d'/d = 0·10		d'/d = 0·15		d'/d = 0·20	
	p_t	p_c	p_t	p_c	p_t	p_c	p_t	p_c
3·73	2·202	0·002	2·202	0·003	2·202	0·003	2·203	0·003
3·75	2·212	0·013	2·213	0·013	2·213	0·014	2·214	0·015
3·80	2·236	0·038	2·238	0·040	2·240	0·043	2·243	0·045
3·85	2·260	0·064	2·264	0·067	2·267	0·071	2·272	0·076
3·90	2·284	0·089	2·289	0·094	2·294	0·100	2·300	0·106
3·95	2·309	0·115	2·315	0·121	2·322	0·128	2·329	0·136
4·00	2·333	0·140	2·340	0·148	2·349	0·157	2·358	0·167
4·05	2·357	0·166	2·366	0·175	2·376	0·185	2·387	0·197
4·10	2·381	0·191	2·391	0·202	2·403	0·214	2·415	0·227
4·15	2·406	0·217	2·417	0·229	2·430	0·242	2·444	0·258
4·20	2·430	0·242	2·443	0·256	2·457	0·271	2·473	0·288
4·25	2·454	0·268	2·468	0·283	2·484	0·299	2·502	0·318
4·30	2·478	0·293	2·494	0·310	2·511	0·328	2·530	0·348
4·35	2·502	0·319	2·519	0·337	2·538	0·356	2·559	0·379
4·40	2·527	0·344	2·545	0·364	2·565	0·385	2·588	0·409
4·45	2·551	0·370	2·570	0·391	2·592	0·414	2·617	0·439
4·50	2·575	0·395	2·596	0·417	2·619	0·442	2·645	0·470
4·55	2·599	0·421	2·621	0·444	2·646	0·471	2·674	0·500
4·60	2·623	0·447	2·647	0·471	2·673	0·499	2·703	0·530
4·65	2·648	0·472	2·673	0·498	2·700	0·528	2·732	0·561
4·70	2·672	0·498	2·698	0·525	2·727	0·556	2·760	0·591
4·75	2·696	0·523	2·724	0·552	2·754	0·585	2·789	0·621
4·80	2·720	0·549	2·749	0·579	2·782	0·613	2·818	0·651
4·85	2·744	0·574	2·775	0·606	2·809	0·642	2·847	0·682
4·90	2·769	0·600	2·800	0·633	2·836	0·670	2·875	0·712
4·95	2·793	0·625	2·826	0·660	2·863	0·699	2·904	0·742
5·00	2·817	0·651	2·851	0·687	2·890	0·727	2·933	0·773
5·05	2·841	0·676	2·877	0·714	2·917	0·756	2·962	0·803
5·10	2·866	0·702	2·903	0·741	2·944	0·784	2·990	0·833
5·15	2·890	0·727	2·928	0·768	2·971	0·813	3·019	0·864
5·20	2·914	0·753	2·954	0·795	2·998	0·841	3·048	0·894
5·25	2·938	0·778	2·979	0·822	3·025	0·870	3·077	0·924
5·30	2·962	0·804	3·005	0·848	3·052	0·898	3·105	0·955
5·35	2·987	0·829	3·030	0·875	3·079	0·927	3·134	0·985
5·40	3·011	0·855	3·056	0·902	3·106	0·955	3·163	1·015
5·45	3·035	0·880	3·081	0·929	3·133	0·984	3·192	1·045
5·50	3·059	0·906	3·107	0·956	3·160	1·012	3·220	1·076
5·55	3·083	0·931	3·133	0·983	3·187	1·041	3·249	1·106
5·60	3·108	0·957	3·158	1·010	3·214	1·070	3·278	1·136
5·65	3·132	0·982	3·184	1·037	3·242	1·098	3·307	1·167
5·70	3·156	1·008	3·209	1·064	3·269	1·127	3·335	1·197
5·75	3·180	1·033	3·235	1·091	3·296	1·155	3·364	1·227
5·80	3·204	1·059	3·260	1·118	3·323	1·184	3·393	1·258
5·85	3·229	1·085	3·286	1·145	3·350	1·212	3·422	1·288
5·90	3·253	1·110	3·311	1·172	3·377	1·241	3·450	1·318

TABLE 48 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

f_{ck} = 30 N/mm²
f_y = 250 N/mm²

M_u/bd^2, N/mm²	d'/d = 0·05		d'/d = 0·10		d'/d = 0·15		d'/d = 0·20	
	P_t	P_c	P_t	P_c	P_t	P_c	P_t	P_c
4·48	2·645	0·005	2·645	0·005	2·645	0·006	2·645	0·006
4·50	2·654	0·015	2·655	0·016	2·656	0·017	2·657	0·018
4·55	2·678	0·041	2·681	0·043	2·683	0·046	2·686	0·049
4·60	2·703	0·067	2·706	0·071	2·710	0·075	2·714	0·080
4·65	2·727	0·093	2·732	0·098	2·737	0·104	2·743	0·110
4·70	2·751	0·119	2·757	0·125	2·764	0·133	2·772	0·141
4·75	2·775	0·144	2·783	0·152	2·791	0·161	2·801	0·171
4·80	2·799	0·170	2·808	0·180	2·818	0·190	2·829	0·202
4·85	2·824	0·196	2·834	0·207	2·845	0·219	2·858	0·233
4·90	2·848	0·222	2·859	0·234	2·872	0·248	2·887	0·263
4·95	2·872	0·248	2·885	0·261	2·899	0·277	2·916	0·294
5·00	2·896	0·273	2·911	0·289	2·927	0·306	2·944	0·325
5·05	2·921	0·299	2·936	0·316	2·954	0·334	2·973	0·355
5·10	2·945	0·325	2·962	0·343	2·981	0·363	3·002	0·386
5·15	2·969	0·351	2·987	0·370	3·008	0·392	3·031	0·417
5·20	2·993	0·377	3·013	0·398	3·035	0·421	3·059	0·447
5·25	3·017	0·402	3·038	0·425	3·062	0·450	3·088	0·478
5·30	3·042	0·428	3·064	0·452	3·089	0·479	3·117	0·508
5·35	3·066	0·454	3·089	0·479	3·116	0·507	3·146	9·539
5·40	3·090	0·480	3·115	0·506	3·143	0·536	3·174	0·570
5·45	3·114	0·506	3·141	0·534	3·170	0·565	3·203	0·600
5·50	3·138	0·531	3·166	0·561	3·197	0·594	3·232	0·631
5·55	3·163	0·557	3·192	0·588	3·224	0·623	3·261	0·662
5·60	3·187	0·583	3·217	0·615	3·251	0·652	3·289	0·692
5·65	3·211	0·609	3·243	0·643	3·278	0·680	3·318	0·723
5·70	3·235	0·635	3·268	0·670	3·305	0·709	3·347	0·754
5·75	3·259	0·660	3·294	0·697	3·332	0·738	3·376	0·784
5·80	3·284	0·686	3·319	0·724	3·359	0·767	3·404	0·815
5·85	3·308	0·712	3·345	0·752	3·387	0·796	3·433	0·845
5·90	3·332	0·738	3·371	0·779	3·414	0·825	3·462	0·876
5·95	3·356	0·764	3·396	0·806	3·441	0·853	3·491	0·907
6·00	3·381	0·789	3·422	0·833	3·468	0·882	3·519	0·937
6·05	3·405	0·815	3·447	0·860	3·495	0·911	3·548	0·968
6·10	3·429	0·841	3·473	0·888	3·522	0·940	3·577	0·999
6·15	3·453	0·867	3·498	0·915	3·549	0·969	3·606	1·029
6·20	3·477	0·893	3·524	0·942	3·576	0·998	3·634	1·060
6·25	3·502	0·918	3·549	0·969	3·603	1·026	3·663	1·091
6·30	3·526	0·944	3·575	0·997	3·630	1·055	3·692	1·121
6·35	3·550	0·970	3·601	1·024	3·657	1·084	3·721	1·152
6·40	3·574	0·996	3·626	1·051	3·684	1·113	3·749	1·182
6·45	3·598	1·022	3·652	1·078	3·711	1·142	3·778	1·213
6·50	3·623	1·047	3·677	1·106	3·738	1·171	3·807	1·244
6·55	3·647	1·073	3·703	1·133	3·765	1·199	3·836	1·274
6·60	3·671	1·099	3·728	1·160	3·792	1·228	3·864	1·305
6·65	3·695	1·125	3·754	1·187	3·819	1·257	3·893	1·336

TABLE 49 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

f_{ck} = 15 N/mm²
f_y = 415 N/mm²

M_u/bd^2, N/mm²	d'/d = 0·05		d'/d = 0·10		d'/d = 0·15		d'/d = 0·20	
	p_t	p_c	p_t	p_c	p_t	p_c	p_t	p_c
2·08	0·719	0·003	0·720	0·003	0·720	0·003	0·720	0·003
2·10	0·725	0·009	0·726	0·009	0·726	0·010	0·727	0·011
2·20	0·754	0·039	0·757	0·041	0·759	0·045	0·761	0·050
2·30	0·784	0·069	0·787	0·073	0·791	0·080	0·796	0·089
2·40	0·813	0·099	0·818	0·106	0·824	0·115	0·831	0·127
2·50	0·842	0·129	0·849	0·138	0·857	0·150	0·865	0·166
2·60	0·871	0·160	0·880	0·170	0·889	0·185	0·900	0·205
2·70	0·900	0·190	0·910	0·202	0·922	0·220	0·935	0·244
2·80	0·929	0·220	0·941	0·234	0·954	0·255	0·969	0·282
2·90	0·959	0·250	0·972	0·267	0·987	0·290	1·004	0·321
3·00	0·988	0·280	1·003	0·299	1·020	0·325	1·039	0·360
3·10	1·017	0·311	1·034	0·331	1·052	0·360	1·073	0·399
3·20	1·046	0·341	1·064	0·363	1·085	0·395	1·108	0·438
3·30	1·075	0·371	1·095	0·395	1·117	0·430	1·142	0·476
3·40	1·104	0·401	1·126	0·427	1·150	0·465	1·177	0·515
3·50	1·134	0·432	1·157	0·460	1·183	0·500	1·212	0·554
3·60	1·163	0·462	1·188	0·492	1·215	0·535	1·246	0·593
3·70	1·192	0·492	1·218	0·524	1·248	0·571	1·281	0·631
3·80	1·221	0·522	1·249	0·556	1·280	0·606	1·316	0·670
3·90	1·250	0·552	1·280	0·588	1·313	0·641	1·350	0·709
4·00	1·279	0·583	1·311	0·621	1·346	0·676	1·385	0·748
4·10	1·309	0·613	1·342	0·653	1·378	0·711	1·420	0·787
4·20	1·338	0·643	1·372	0·685	1·411	0·746	1·454	0·825
4·30	1·367	0·673	1·403	0·717	1·443	0·781	1·489	0·864
4·40	1·396	0·703	1·434	0·749	1·476	0·816	1·524	0·903
4·50	1·425	0·734	1·465	0·781	1·509	0·851	1·558	0·942
4·60	1·455	0·764	1·495	0·814	1·541	0·886	1·593	0·980
4·70	1·484	0·794	1·526	0·846	1·574	0·921	1·627	1·019
4·80	1·513	0·824	1·557	0·878	1·606	0·956	1·662	1·058
4·90	1·542	0·855	1·588	0·910	1·639	0·991	1·697	1·097
5·00	1·571	0·885	1·619	0·942	1·672	1·026	1·731	1·136
5·10	1·600	0·915	1·649	0·975	1·704	1·061	1·766	1·174
5·20	1·630	0·945	1·680	1·007	1·737	1·096	1·801	1·213
5·30	1·659	0·975	1·711	1·039	1·769	1·131	1·835	1·252
5·40	1·688	1·006	1·742	1·071	1·802	1·166	1·870	1·291
5·50	1·717	1·036	1·773	1·103	1·835	1·201	1·905	1·329
5·60	1·746	1·066	1·803	1·136	1·867	1·236	1·939	1·368
5·70	1·775	1·096	1·834	1·168	1·900	1·271	1·974	1·407
5·80	1·805	1·126	1·865	1·200	1·932	1·306	2·008	1·446
5·90	1·834	1·157	1·896	1·232	1·965	1·341	2·043	1·485
6·00	1·863	1·187	1·927	1·264	1·998	1·376	2·078	1·523
6·10	1·892	1·217	1·957	1·296	2·030	1·411	2·112	1·562
6·20	1·921	1·247	1·988	1·329	2·063	1·446	2·147	1·601
6·30	1·950	1·278	2·019	1·361	2·095	1·481	2·182	1·640
6·40	1·980	1·308	2·050	1·393	2·128	1·517	2·216	1·678

TABLE 50 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

f_{ck} = 20 N/mm²
f_y = 415 N/mm²

M_u/bd^2, N/mm²	d'/d = 0·05		d'/d = 0·10		d'/d = 0·15		d'/d = 0·20	
	P_t	P_c	P_t	P_c	P_t	P_c	P_t	P_c
2·77	0·958	0·002	0·958	0·002	0·959	0·003	0·959	0·003
2·80	0·967	0·011	0·968	0·012	0·968	0·013	0·969	0·015
2·90	0·996	0·042	0·998	0·045	1·001	0·049	1·004	0·054
3·00	1·025	0·072	1·029	0·077	1·034	0·084	1·038	0·093
3·10	1·055	0·103	1·060	0·109	1·066	0·119	1·073	0·132
3·20	1·084	0·133	1·091	0·142	1·099	0·154	1·108	0·171
3·30	1·113	0·164	1·122	0·174	1·131	0·190	1·142	0·210
3·40	1·142	0·194	1·152	0·207	1·164	0·225	1·177	0·249
3·50	1·171	0·224	1·183	0·239	1·197	0·260	1·212	0·288
3·60	1·200	0·255	1·214	0·271	1·229	0·295	1·246	0·327
3·70	1·230	0·285	1·245	0·304	1·262	0·331	1·281	0·366
3·80	1·259	0·316	1·276	0·336	1·294	0·366	1·315	0·405
3·90	1·288	0·346	1·306	0·369	1·327	0·401	1·350	0·444
4·00	1·317	0·376	1·337	0·401	1·360	0·437	1·385	0·483
4·10	1·346	0·407	1·368	0·433	1·392	0·472	1·419	0·522
4·20	1·375	0·437	1·399	0·466	1·425	0·507	1·454	0·561
4·30	1·405	0·468	1·429	0·498	1·457	0·542	1·489	0·600
4·40	1·434	0·498	1·460	0·530	1·490	0·578	1·523	0·640
4·50	1·463	0·528	1·491	0·563	1·523	0·613	1·558	0·679
4·60	1·492	0·559	1·522	0·595	1·555	0·648	1·593	0·718
4·70	1·521	0·589	1·553	0·628	1·588	0·683	1·627	0·757
4·80	1·550	0·620	1·583	0·660	1·620	0·719	1·662	0·796
4·90	1·580	0·650	1·614	0·692	1·653	0·754	1·696	0·835
5·00	1·609	0·680	1·645	0·725	1·686	0·789	1·731	0·874
5·10	1·638	0·711	1·676	0·757	1·718	0·825	1·766	0·913
5·20	1·667	0·741	1·707	0·790	1·751	0·860	1·800	0·952
5·30	1·696	0·772	1·737	0·822	1·783	0·895	1·835	0·991
5·40	1·725	0·802	1·768	0·854	1·816	0·930	1·870	1·030
5·50	1·755	0·832	1·799	0·887	1·849	0·966	1·904	1·069
5·60	1·784	0·863	1·830	0·919	1·881	1·001	1·939	1·108
5·70	1·813	0·893	1·861	0·952	1·914	1·036	1·974	1·147
5·80	1·842	0·924	1·891	0·984	1·946	1·071	2·008	1·186
5·90	1·871	0·954	1·922	1·016	1·979	1·107	2·043	1·225
6·00	1·900	0·985	1·953	1·049	2·012	1·142	2·078	1·264
6·10	1·930	1·015	1·984	1·081	2·044	1·177	2·112	1·303
6·20	1·959	1·045	2·014	1·114	2·077	1·213	2·147	1·342
6·30	1·988	1·076	2·045	1·146	2·109	1·248	2·181	1·381
6·40	2·017	1·106	2·076	1·178	2·142	1·283	2·216	1·421
6·50	2·046	1·137	2·107	1·211	2·175	1·318	2·251	1·460
6·60	2·075	1·167	2·138	1·243	2·207	1·354	2·285	1·499
6·70	2·105	1·197	2·168	1·276	2·240	1·389	2·320	1·538
6·80	2·134	1·228	2·199	1·308	2·272	1·424	2·355	1·577
6·90	2·163	1·258	2·230	1·340	2·305	1·459	2·389	1·616
7·00	2·192	1·289	2·261	1·373	2·338	1·495	2·424	1·655
7·10	2·221	1·319	2·292	1·405	2·370	1·530	2·459	1·694

TABLE 51 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

f_{ck} = 25 N/mm²
f_y = 415 N/mm²

M_u/bd^2, N/mm²	d'/d = 0·05		d'/d = 0·10		d'/d = 0·15		d'/d = 0·20	
	p_t	p_c	p_t	p_c	p_t	p_c	p_t	p_c
3·46	1·197	0·002	1·197	0·002	1·197	0·003	1·197	0·003
3·50	1·209	0·014	1·210	0·015	1·210	0·017	1·211	0·019
3·60	1·238	0·045	1·240	0·048	1·243	0·052	1·246	0·058
3·70	1·267	0·076	1·271	0·081	1·276	0·088	1·281	0·097
3·80	1·296	0·106	1·302	0·113	1·308	0·123	1·315	0·137
3·90	1·325	0·137	1·333	0·146	1·341	0·159	1·350	0·176
4·00	1·355	0·167	1·363	0·178	1·373	0·194	1·385	0·215
4·10	1·384	0·198	1·394	0·211	1·406	0·230	1·419	0·254
4·20	1·413	0·229	1·425	0·244	1·439	0·265	1·454	0·294
4·30	1·442	0·259	1·456	0·276	1·471	0·301	1·488	0·333
4·40	1·471	0·290	1·487	0·309	1·504	0·336	1·523	0·372
4·50	1·500	0·320	1·517	0·341	1·536	0·372	1·558	0·412
4·60	1·530	0·351	1·548	0·374	1·569	0·407	1·592	0·451
4·70	1·559	0·382	1·579	0·407	1·602	0·443	1·627	0·490
4·80	1·588	0·412	1·610	0·439	1·634	0·478	1·662	0·530
4·90	1·617	0·443	1·641	0·472	1·667	0·514	1·696	0·569
5·00	1·646	0·474	1·671	0·504	1·699	0·549	1·731	0·608
5·10	1·675	0·504	1·702	0·537	1·732	0·585	1·766	0·648
5·20	1·705	0·535	1·733	0·570	1·765	0·620	1·800	0·687
5·30	1·734	0·565	1·764	0·602	1·797	0·656	1·835	0·726
5·40	1·763	0·596	1·795	0·635	1·830	0·691	1·869	0·766
5·50	1·792	0·627	1·825	0·667	1·862	0·727	1·904	0·805
5·60	1·821	0·657	1·856	0·700	1·895	0·762	1·939	0·844
5·70	1·851	0·688	1·887	0·733	1·928	0·798	1·973	0·884
5·80	1·880	0·718	1·918	0·765	1·960	0·833	2·008	0·923
5·90	1·909	0·749	1·948	0·798	1·993	0·869	2·043	0·962
6·00	1·938	0·780	1·979	0·830	2·025	0·904	2·077	1·002
6·10	1·967	0·810	2·010	0·863	2·058	0·940	2·112	1·041
6·20	1·996	0·841	2·041	0·896	2·091	0·975	2·147	1·080
6·30	2·026	0·871	2·072	0·928	2·123	1·011	2·181	1·120
6·40	2·055	0·902	2·102	0·961	2·156	1·046	2·216	1·159
6·50	2·084	0·933	2·133	0·993	2·188	1·082	2·251	1·198
6·60	2·113	0·963	2·164	1·026	2·221	1·118	2·285	1·238
6·70	2·142	0·994	2·195	1·059	2·254	1·153	2·320	1·277
6·80	2·171	1·024	2·226	1·091	2·286	1·189	2·354	1·316
6·90	2·201	1·055	2·256	1·124	2·319	1·224	2·389	1·356
7·00	2·230	1·086	2·287	1·157	2·351	1·260	2·424	1·395
7·10	2·259	1·116	2·318	1·189	2·384	1·295	2·458	1·434
7·20	2·288	1·147	2·349	1·222	2·417	1·331	2·493	1·474
7·30	2·317	1·177	2·380	1·254	2·449	1·366	2·528	1·513
7·40	2·346	1·208	2·410	1·287	2·482	1·402	2·562	1·552
7·50	2·376	1·239	2·441	1·320	2·514	1·437	2·597	1·591
7·60	2·405	1·269	2·472	1·352	2·547	1·473	2·632	1·631
7·70	2·434	1·300	2·503	1·385	2·580	1·508	2·666	1·670
7·80	2·463	1·330	2·534	1·417	2·612	1·544	2·701	1·709

TABLE 52 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

f_{ck} = 30 N/mm²
f_y = 415 N/mm²

M_u/bd^2, N/mm²	d'/d = 0·05		d'/d = 0·10		d'/d = 0·15		d'/d = 0·20	
	p_t	p_c	p_t	p_c	p_t	p_c	p_t	p_c
4·15	1·436	0·002	1·436	0·002	1·436	0·002	1·436	0·003
4·20	1·451	0·017	1·451	0·019	1·452	0·020	1·454	0·022
4·30	1·480	0·048	1·482	0·051	1·485	0·056	1·488	0·062
4·40	1·509	0·079	1·513	0·084	1·518	0·092	1·523	0·102
4·50	1·538	0·110	1·544	0·117	1·550	0·127	1·558	0·141
4·60	1·567	0·141	1·575	0·150	1·583	0·163	1·592	0·181
4·70	1·596	0·171	1·605	0·183	1·615	0·199	1·627	0·220
4·80	1·626	0·202	1·636	0·215	1·648	0·235	1·661	0·260
4·90	1·655	0·233	1·667	0·248	1·681	0·270	1·696	0·300
5·00	1·684	0·264	1·698	0·281	1·713	0·306	1·731	0·339
5·10	1·713	0·295	1·729	0·314	1·746	0·342	1·765	0·379
5·20	1·742	0·325	1·759	0·347	1·778	0·378	1·800	0·418
5·30	1·771	0·356	1·790	0·380	1·811	0·413	1·835	0·458
5·40	1·801	0·387	1·821	0·412	1·844	0·449	1·869	0·498
5·50	1·830	0·418	1·852	0·445	1·876	0·485	1·904	0·537
5·60	1·859	0·449	1·883	0·478	1·909	0·521	1·939	0·577
5·70	1·888	0·479	1·913	0·511	1·941	0·556	1·973	0·616
5·80	1·917	0·510	1·944	0·544	1·974	0·592	2·008	0·656
5·90	1·946	0·541	1·975	0·576	2·007	0·628	2·042	0·696
6·00	1·976	0·572	2·006	0·609	2·039	0·664	2·077	0·735
6·10	2·005	0·603	2·036	0·642	2·072	0·699	2·112	0·775
6·20	2·034	0·634	2·067	0·675	2·104	0·735	2·146	0·814
6·30	2·063	0·664	2·098	0·708	2·137	0·771	2·181	0·854
6·40	2·092	0·695	2·129	0·741	2·170	0·807	2·216	0·894
6·50	2·121	0·726	2·160	0·773	2·202	0·842	2·250	0·933
6·60	2·151	0·757	2·190	0·806	2·235	0·878	2·285	0·973
6·70	2·180	0·788	2·221	0·839	2·267	0·914	2·320	1·012
6·80	2·209	0·818	2·252	0·872	2·300	0·950	2·354	1·052
6·90	2·238	0·849	2·283	0·905	2·333	0·985	2·389	1·092
7·00	2·267	0·880	2·314	0·937	2·365	1·021	2·424	1·131
7·10	2·296	0·911	2·344	0·970	2·398	1·057	2·458	1·171
7·20	2·326	0·942	2·375	1·003	2·431	1·093	2·493	1·210
7·30	2·355	0·972	2·406	1·036	2·463	1·128	2·527	1·250
7·40	2·384	1·003	2·437	1·069	2·496	1·164	2·562	1·290
7·50	2·413	1·034	2·468	1·102	2·528	1·200	2·597	1·329
7·60	2·442	1·065	2·498	1·134	2·561	1·236	2·631	1·369
7·70	2·471	1·096	2·529	1·167	2·594	1·271	2·666	1·408
7·80	2·501	1·126	2·560	1·200	2·626	1·307	2·701	1·448
7·90	2·530	1·157	2·591	1·233	2·659	1·343	2·735	1·488
8·00	2·559	1·188	2·621	1·266	2·691	1·379	2·770	1·527
8·10	2·588	1·219	2·652	1·299	2·724	1·414	2·805	1·567
8·20	2·617	1·250	2·683	1·331	2·757	1·450	2·839	1·606
8·30	2·646	1·280	2·714	1·364	2·789	1·486	2·874	1·646
8·40	2·676	1·311	2·745	1·397	2·822	1·522	2·908	1·686
8·50	2·705	1·342	2·775	1·430	2·854	1·557	2·943	1·725

TABLE 53 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

f_{ck} = 15 N/mm²
f_y = 500 N/mm²

M_u/bd^2, N/mm²	d'/d = 0·05		d'/d = 0·10		d'/d = 0·15		d'/d = 0·20	
	p_t	p_c	p_t	p_c	p_t	p_c	p_t	p_c
2·00	0·568	0·001	0·568	0·001	0·568	0·001	0·568	0·002
2·10	0·592	0·026	0·593	0·029	0·595	0·032	0·596	0·036
2·20	0·616	0·052	0·619	0·056	0·622	0·062	0·625	0·070
2·30	0·640	0·077	0·644	0·084	0·649	0·092	0·654	0·105
2·40	0·664	0·102	0·670	0·111	0·676	0·122	0·683	0·139
2·50	0·689	0·127	0·695	0·138	0·703	0·153	0·711	0·173
2·60	0·713	0·153	0·721	0·166	0·730	0·183	0·740	0·208
2·70	0·737	0·178	0·746	0·193	0·757	0·213	0·769	0·242
2·80	0·761	0·203	0·772	0·221	0·784	0·244	0·798	0·276
2·90	0·785	0·228	0·798	0·248	0·811	0·274	0·826	0·310
3·00	0·810	0·253	0·823	0·276	0·838	0·304	0·855	0·345
3·10	0·834	0·279	0·849	0·303	0·865	0·334	0·884	0·379
3·20	0·858	0·304	0·874	0·330	0·892	0·365	0·913	0·413
3·30	0·882	0·329	0·900	0·358	0·919	0·395	0·941	0·448
3·40	0·906	0·354	0·925	0·385	0·946	0·425	0·970	0·482
3·50	0·931	0·380	0·951	0·413	0·974	0·455	0·999	0·516
3·60	0·955	0·405	0·976	0·440	1·001	0·486	1·028	0·551
3·70	0·979	0·430	1·002	0·468	1·028	0·516	1·056	0·585
3·80	1·003	0·455	1·028	0·495	1·055	0·546	1·085	0·619
3·90	1·028	0·481	1·053	0·523	1·082	0·577	1·114	0·654
4·00	1·052	0·506	1·079	0·550	1·109	0·607	1·143	0·688
4·10	1·076	0·531	1·104	0·577	1·136	0·637	1·171	0·722
4·20	1·100	0·556	1·130	0·605	1·163	0·667	1·200	0·757
4·30	1·124	0·582	1·155	0·632	1·190	0·698	1·229	0·791
4·40	1·149	0·607	1·181	0·660	1·217	0·728	1·258	0·825
4·50	1·173	0·632	1·206	0·687	1·244	0·758	1·286	0·860
4·60	1·197	0·657	1·232	0·715	1·271	0·789	1·315	0·894
4·70	1·221	0·682	1·258	0·742	1·298	0·819	1·344	0·928
4·80	1·245	0·708	1·283	0·769	1·325	0·849	1·373	0·963
4·90	1·270	0·733	1·309	0·797	1·352	0·879	1·401	0·997
5·00	1·294	0·758	1·334	0·824	1·379	0·910	1·430	1·031
5·10	1·318	0·783	1·360	0·852	1·406	0·940	1·459	1·066
5·20	1·342	0·809	1·385	0·879	1·4[illegible]4	0·970	1·488	1·100
5·30	1·366	0·834	1·411	0·907	1·461	1·000	1·516	1·134
5·40	1·391	0·859	1·436	0·934	1·488	1·031	1·545	1·169
5·50	1·415	0·884	1·462	0·962	1·515	1·061	1·574	1·203
5·60	1·439	0·910	1·488	0·989	1·542	1·091	1·603	1·237
5·70	1·463	0·935	1·513	1·016	1·569	1·122	1·631	1·272
5·80	1·488	0·960	1·539	1·044	1·596	1·152	1·660	1·306
5·90	1·512	0·985	1·564	1·071	1·623	1·182	1·689	1·340
6·00	1·536	1·011	1·590	1·099	1·650	1·212	1·718	1·375
6·10	1·560	1·036	1·615	1·126	1·677	1·243	1·746	1·409
6·20	1·584	1·061	1·641	1·154	1·704	1·273	1·775	1·443
6·30	1·609	1·086	1·666	1·181	1·731	1·303	1·804	1·478
6·40	1·633	1·111	1·692	1·208	1·758	1·334	1·833	1·512

TABLE 54 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

f_{ck} = 20 N/mm²
f_y = 500 N/mm²

M_u/bd^2, N/mm²	d'/d = 0·05 p_t	d'/d = 0·05 p_c	d'/d = 0·10 p_t	d'/d = 0·10 p_c	d'/d = 0·15 p_t	d'/d = 0·15 p_c	d'/d = 0·20 p_t	d'/d = 0·20 p_c
2·67	0·758	0·002	0·758	0·003	0·758	0·003	0·758	0·003
2·70	0·765	0·010	0·765	0·011	0·766	0·012	0·767	0·014
2·80	0·789	0·035	0·791	0·038	0·793	0·042	0·795	0·048
2·90	0·813	0·061	0·816	0·066	0·820	0·073	0·824	0·083
3·00	0·837	0·086	0·842	0·094	0·847	0·103	0·853	0·117
3·10	0·862	0·111	0·868	0·121	0·874	0·134	0·882	0·152
3·20	0·886	0·137	0·893	0·149	0·901	0·164	0·910	0·186
3·30	0·910	0·162	0·919	0·176	0·928	0·195	0·939	0·221
3·40	0·934	0·188	0·944	0·204	0·955	0·225	0·968	0·255
3·50	0·958	0·213	0·970	0·232	0·982	0·256	0·997	0·290
3·60	0·983	0·238	0·995	0·259	1·009	0·286	1·025	0·324
3·70	1·007	0·264	1·021	0·287	1·036	0·316	1·054	0·359
3·80	1·031	0·289	1·046	0·314	1·064	0·347	1·083	0·394
3·90	1·055	0·314	1·072	0·342	1·091	0·377	1·112	0·428
4·00	1·080	0·340	1·098	0·369	1·118	0·408	1·140	0·463
4·10	1·104	0·365	1·123	0·397	1·145	0·438	1·169	0·497
4·20	1·128	0·391	1·149	0·425	1·172	0·469	1·198	0·532
4·30	1·152	0·416	1·174	0·452	1·199	0·499	1·227	0·566
4·40	1·176	0·441	1·200	0·480	1·226	0·530	1·255	0·601
4·50	1·201	0·467	1·225	0·507	1·253	0·560	1·284	0·635
4·60	1·225	0·492	1·251	0·535	1·280	0·591	1·313	0·670
4·70	1·249	0·517	1·276	0·563	1·307	0·621	1·342	0·704
4·80	1·273	0·543	1·302	0·590	1·334	0·651	1·370	0·739
4·90	1·297	0·568	1·328	0·618	1·361	0·682	1·399	0·773
5·00	1·322	0·593	1·353	0·645	1·388	0·712	1·428	[illegible]
5·10	1·346	0·619	1·379	0·673	1·415	0·743	1·457	0·843
5·20	1·370	0·644	1·404	0·701	1·442	0·773	1·485	0·877
5·30	1·394	0·670	1·430	0·728	1·469	0·804	1·514	0·912
5·40	1·418	0·695	1·455	0·756	1·496	0·834	1·543	0·946
5·50	1·443	0·720	1·481	0·783	1·524	0·865	1·572	0·981
5·60	1·467	0·746	1·506	0·811	1·551	0·895	1·600	1·015
5·70	1·491	0·771	1·532	0·839	1·578	0·925	1·629	1·050
5·80	1·515	0·796	1·558	0·866	1·605	0·956	1·658	1·084
5·90	1·540	0·822	1·583	0·894	1·632	0·986	1·687	1·119
6·00	1·564	0·847	1·609	0·921	1·659	1·017	1·715	1·153
6·10	1·588	0·873	1·634	0·949	1·686	1·047	1·744	1·188
6·20	1·612	0·898	1·660	0·976	1·713	1·078	1·773	1·223
6·30	1·636	0·923	1·685	1·004	1·740	1·108	1·802	1·257
6·40	1·661	0·949	1·711	1·032	1·767	1·139	1·830	1·292
6·50	1·685	0·974	1·736	1·059	1·794	1·169	1·859	1·326
6·60	1·709	0·999	1·762	1·087	1·821	1·200	1·888	1·361
6·70	1·733	1·025	1·788	1·114	1·848	1·230	1·917	1·395
6·80	1·757	1·050	1·813	1·142	1·875	1·260	1·945	1·430
6·90	1·782	1·076	1·839	1·170	1·902	1·291	1·974	1·464
7·00	1·806	1·101	1·864	1·197	1·929	1·321	2·003	1·499

TABLE 55 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

f_y 500

f_{ck} 25

f_{ck} = 25 N/mm²
f_y = 500 N/mm²

M_u/bd^2, N/mm²	d'/d = 0·05		d'/d = 0·10		d'/d = 0·15		d'/d = 0·20	
	p_t	p_c	p_t	p_c	p_t	p_c	p_t	p_c
3·33	0·945	0·001	0·945	0·001	0·945	0·001	0·945	0·001
3·40	0·962	0·019	0·963	0·021	0·964	0·023	0·965	0·026
3·50	0·986	0·044	0·989	0·048	0·991	0·053	0·994	0·060
3·60	1·010	0·070	1·014	0·076	1·018	0·084	1·023	0·095
3·70	1·035	0·095	1·040	0·104	1·045	0·115	1·052	0·130
3·80	1·059	0·121	1·065	0·132	1·072	0·145	1·080	0·165
3·90	1·083	0·146	1·091	0·159	1·099	0·176	1·109	0·200
4·00	1·107	0·172	1·116	0·187	1·126	0·206	1·138	0·234
4·10	1·131	0·197	1·142	0·215	1·154	0·237	1·167	0·269
4·20	1·156	0·223	1·167	0·242	1·181	0·268	1·195	0·304
4·30	1·180	0·248	1·193	0·270	1·208	0·298	1·224	0·339
4·40	1·204	0·274	1·219	0·298	1·235	0·329	1·253	0·373
4·50	1·228	0·299	1·244	0·326	1·262	0·360	1·282	0·408
4·60	1·253	0·325	1·270	0·353	1·289	0·390	1·310	0·443
4·70	1·277	0·350	1·295	0·381	1·316	0·421	1·339	0·478
4·80	1·301	0·376	1·321	0·409	1·343	0·451	1·368	0·512
4·90	1·325	0·402	1·346	0·437	1·370	0·482	1·397	0·547
5·00	1·349	0·427	1·372	0·464	1·397	0·513	1·425	0·582
5·10	1·374	0·453	1·397	0·492	1·424	0·543	1·454	0·617
5·20	1·398	0·478	1·423	0·520	1·451	0·574	1·483	0·651
5·30	1·422	0·504	1·449	0·548	1·478	0·605	1·512	0·686
5·40	1·446	0·529	1·474	0·575	1·505	0·635	1·540	0·721
5·50	1·470	0·555	1·500	0·603	1·532	0·666	1·569	0·756
5·60	1·495	0·380	1·525	0·631	1·559	0·697	1·598	0·790
5·70	1·519	0·606	1·551	0·659	1·586	0·727	1·627	0·825
5·80	1·543	0·631	1·576	0·686	1·614	0·758	1·655	0·860
5·90	1·567	0·657	1·602	0·714	1·641	0·788	1·684	0·895
6·00	1·592	0·682	1·627	0·742	1·668	0·819	1·713	0·929
6·10	1·616	0·708	1·653	0·770	1·695	0·850	1·742	0·964
6·20	1·640	0·733	1·679	0·797	1·722	0·880	1·770	0·999
6·30	1·664	0·759	1·704	0·825	1·749	0·911	1·799	1·034
6·40	1·688	0·784	1·730	0·853	1·776	0·942	1·828	1·068
6·50	1·713	0·810	1·755	0·881	1·803	0·972	1·857	1·103
6·60	1·737	0·835	1·781	0·908	1·830	1·003	1·885	1·138
6·70	1·761	0·861	1·806	0·936	1·857	1·033	1·914	1·173
6·80	1·785	0·886	1·832	0·964	1·884	1·064	1·943	1·207
6·90	1·809	0·912	1·857	0·992	1·911	1·095	1·972	1·242
7·00	1·834	0·937	1·883	1·019	1·938	1·125	2·000	1·277
7·10	1·858	0·963	1·909	1·047	1·965	1·156	2·029	1·312
7·20	1·882	0·988	1·934	1·075	1·992	1·187	2·058	1·346
7·30	1·906	1·014	1·960	1·103	2·019	1·217	2·087	1·381
7·40	1·930	1·039	1·985	1·130	2·046	1·248	2·115	1·416
7·50	1·955	1·065	2·011	1·138	2·074	1·278	2·144	1·431
7·60	1·979	1·090	2·036	1·186	2·101	1·309	2·173	1·486
7·70	2·003	1·116	2·062	1·213	2·128	1·340	2·202	1·520

TABLE 56 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

f_{ck} = 30 N/mm²
f_y = 500 N/mm²

M_u/bd^2, N/mm²	d'/d = 0·05		d'/d = 0·10		d'/d = 0·15		d'/d = 0·20	
	p_t	p_c	p_t	p_c	p_t	p_c	p_t	p_c
4·00	1·135	0·002	1·135	0·002	1·135	0·003	1·135	0·003
4·10	1·159	0·028	1·161	0·030	1·162	0·034	1·164	0·038
4·20	1·183	0·054	1·186	0·058	1·189	0·064	1·193	0·073
4·30	1·208	0·079	1·212	0·086	1·216	0·095	1·222	0·108
4·40	1·232	0·105	1·237	0·114	1·244	0·126	1·250	0·143
4·50	1·256	0·130	1·263	0·142	1·271	0·157	1·279	0·178
4·60	1·280	0·156	1·289	0·170	1·298	0·188	1·308	0·213
4·70	1·305	0·182	1·314	0·198	1·325	0·218	1·337	0·248
4·80	1·329	0·207	1·340	0·226	1·352	0·249	1·365	0·283
4·90	1·353	0·233	1·365	0·254	1·379	0·280	1·394	0·318
5·00	1·377	0·259	1·391	0·281	1·406	0·311	1·423	0·353
5·10	1·401	0·284	1·416	0·309	1·433	0·342	1·452	0·388
5·20	1·426	0·310	1·442	0·337	1·460	0·372	1·480	0·423
5·30	1·450	0·336	1·467	0·365	1·487	0·403	1·509	0·458
5·40	1·474	0·361	1·493	0·393	1·514	0·434	1·538	0·493
5·50	1·498	0·387	1·519	0·421	1·541	0·465	1·567	0·528
5·60	1·522	0·413	1·544	0·449	1·568	0·496	1·595	0·563
5·70	1·547	0·438	1·570	0·477	1·595	0·526	1·624	0·598
5·80	1·571	0·464	1·595	0·505	1·622	0·557	1·653	0·633
5·90	1·595	0·490	1·621	0·533	1·649	0·588	1·682	0·668
6·00	1·619	0·515	1·646	0·560	1·676	0·619	1·710	0·703
6·10	1·643	0·541	1·672	0·588	1·704	0·650	1·739	0·738
6·20	1·668	0·566	1·697	0·616	1·731	0·680	1·768	0·773
6·30	1·692	0·592	1·723	0·644	1·758	0·711	1·797	0·807
6·40	1·716	0·618	1·749	0·672	1·785	0·742	1·825	0·842
6·50	1·740	0·643	1·774	0·700	1·812	0·773	1·854	0·877
6·60	1·765	0·669	1·800	0·728	1·839	0·804	1·883	0·912
6·70	1·789	0·695	1·825	0·756	1·866	0·835	1·912	0·947
6·80	1·813	0·720	1·851	0·784	1·893	0·865	1·940	0·982
6·90	1·837	0·746	1·876	0·812	1·920	0·896	1·969	1·017
7·00	1·861	0·772	1·902	0·839	1·947	0·927	1·998	1·052
7·10	1·886	0·797	1·927	0·867	1·974	0·958	2·027	1·087
7·20	1·910	0·823	1·953	0·895	2·001	0·989	2·055	1·122
7·30	1·934	0·849	1·979	0·923	2·028	1·019	2·084	1·157
7·40	1·958	0·874	2·004	0·951	2·055	1·050	2·113	1·192
7·50	1·982	0·900	2·030	0·979	2·082	1·081	2·142	1·227
7·60	2·007	0·926	2·055	1·007	2·109	1·112	2·170	1·262
7·70	2·031	0·951	2·081	1·035	2·136	1·143	2·199	1·297
7·80	2·055	0·977	2·106	1·063	2·164	1·173	2·228	1·332
7·90	2·079	1·002	2·132	1·091	2·191	1·204	2·257	1·367
8·00	2·103	1·028	2·157	1·118	2·218	1·235	2·285	1·402
8·10	2·128	1·054	2·183	1·146	2·245	1·266	2·314	1·437
8·20	2·152	1·079	2·209	1·174	2·272	1·297	2·343	1·472
8·30	2·176	1·105	2·234	1·202	2·299	1·327	2·372	1·507

TABLE 57 FLEXURE — LIMITING MOMENT OF RESISTANCE FACTOR, $M_{u,lim}/b_w d^2 f_{ck}$, FOR SINGLY REINFORCED T-BEAMS, N/mm²

f_y = 250 N/mm²

D_f/d	b_f/b_w									
	1·0	2·0	3·0	4·0	5·0	6·0	7·0	8·0	9·0	10·0
0·06	0·149	0·175	0·201	0·227	0·253	0·279	0·305	0·331	0·357	0·383
0·07	0·149	0·179	0·209	0·239	0·270	0·300	0·330	0·360	0·390	0·420
0·08	0·149	0·183	0·218	0·252	0·286	0·320	0·355	0·389	0·423	0·457
0·09	0·149	0·187	0·226	0·264	0·302	0·341	0·379	0·417	0·456	0·494
0·10	0·149	0·191	0·234	0·276	0·318	0·361	0·403	0·446	0·488	0·530
0·11	0·149	0·195	0·242	0·288	0·334	0·381	0·427	0·474	0·520	0·566
0·12	0·149	0·199	0·250	0·300	0·350	0·401	0·451	0·501	0·551	0·602
0·13	0·149	0·203	0·257	0·312	0·366	0·420	0·474	0·528	0·583	0·637
0·14	0·149	0·207	0·265	0·323	0·381	0·439	0·497	0·555	0·614	0·672
0·15	0·149	0·211	0·273	0·335	0·397	0·458	0·520	0·582	0·644	0·706
0·16	0·149	0·215	0·280	0·346	0·412	0·477	0·543	0·609	0·674	0·740
0·17	0·149	0·218	0·288	0·357	0·427	0·496	0·565	0·635	0·704	0·773
0·18	0·149	0·222	0·295	0·368	0·441	0·514	0·587	0·660	0·733	0·806
0·19	0·149	0·226	0·302	0·379	0·456	0·532	0·609	0·686	0·763	0·839
0·20	0·149	0·229	0·310	0·390	0·470	0·550	0·631	0·711	0·791	0·872
0·21	0·149	0·233	0·317	0·400	0·484	0·568	0·652	0·736	0·820	0·903
0·22	0·149	0·236	0·324	0·411	0·498	0·586	0·673	0·760	0·848	0·935
0·23	0·149	0·240	0·330	0·421	0·511	0·602	0·692	0·783	0·873	0·964
0·24	0·149	0·242	0·334	0·427	0·520	0·613	0·705	0·798	0·891	0·984
0·25	0·149	0·244	0·339	0·434	0·529	0·624	0·719	0·814	0·909	1·003
0·26	0·149	0·246	0·343	0·440	0·538	0·635	0·732	0·829	0·926	1·023
0·27	0·149	0·248	0·348	0·447	0·546	0·645	0·745	0·844	0·943	1·043
0·28	0·149	0·250	0·352	0·453	0·555	0·656	0·758	0·859	0·961	1·062
0·29	0·149	0·253	0·356	0·460	0·563	0·667	0·770	0·874	0·978	1·081
0·30	0·149	0·255	0·360	0·466	0·572	0·677	0·783	0·889	0·995	1·100
0·31	0·149	0·257	0·365	0·472	0·580	0·688	0·796	0·903	1·011	1·119
0·32	0·149	0·259	0·369	0·479	0·588	0·698	0·808	0·918	1·028	1·138
0·33	0·149	0·261	0·373	0·485	0·597	0·709	0·820	0·932	1·044	1·156
0·34	0·149	0·263	0·377	0·491	0·605	0·719	0·833	0·947	1·061	1·175
0·35	0·149	0·265	0·381	0·497	0·613	0·729	0·845	0·961	1·077	1·193
0·36	0·149	0·267	0·385	0·503	0·621	0·739	0·857	0·975	1·093	1·211
0·37	0·149	0·269	0·389	0·509	0·629	0·749	0·869	0·989	1·109	1·229
0·38	0·149	0·271	0·393	0·515	0·637	0·759	0·880	1·002	1·124	1·246
0·39	0·149	0·273	0·397	0·521	0·644	0·768	0·892	1·016	1·140	1·264
0·40	0·149	0·275	0·401	0·526	0·652	0·778	0·904	1·029	1·155	1·281
0·41	0·149	0·277	0·404	0·532	0·660	0·787	0·915	1·043	1·170	1·298
0·42	0·149	0·279	0·408	0·538	0·667	0·797	0·926	1·056	1·186	1·315
0·43	0·149	0·280	0·412	0·543	0·675	0·806	0·938	1·069	1·200	1·332
0·44	0·149	0·282	0·416	0·549	0·682	0·815	0·949	1·082	1·215	1·349
0·45	0·149	0·284	0·419	0·554	0·689	0·825	0·960	1·095	1·230	1·365

TABLE 58 FLEXURE — LIMITING MOMENT OF RESISTANCE FACTOR, $M_{u,lim}/b_w d^2 f_{ck}$, FOR SINGLY REINFORCED T-BEAMS, N/mm²

f_y = 415 N/mm²

D_f/d	b_f/b_w									
	1·0	2·0	3·0	4·0	5·0	6·0	7·0	8·0	9·0	10·0
0·06	0·138	0·164	0·190	0·216	0·242	0·268	0·294	0·320	0·346	0·372
0·07	0·138	0·168	0·198	0·228	0·259	0·289	0·319	0·349	0·379	0·409
0·08	0·138	0·172	0·207	0·241	0·275	0·309	0·344	0·378	0·412	0·446
0·09	0·138	0·176	0·215	0·253	0·291	0·330	0·368	0·406	0·445	0·483
0·10	0·138	0·180	0·223	0·265	0·308	0·350	0·392	0·435	0·477	0·519
0·11	0·138	0·184	0·231	0·277	0·324	0·370	0·416	0·463	0·509	0·555
0·12	0·138	0·188	0·239	0·289	0·339	0·390	0·440	0·490	0·541	0·591
0·13	0·138	0·192	0·247	0·301	0·355	0·409	0·463	0·518	0·572	0·626
0·14	0·138	0·196	0·254	0·312	0·370	0·428	0·487	0·545	0·603	0·661
0·15	0·138	0·200	0·262	0·324	0·386	0·448	0·509	0·571	0·633	0·695
0·16	0·138	0·204	0·269	0·335	0·401	0·466	0·532	0·598	0·663	0·729
0·17	0·138	0·207	0·277	0·346	0·416	0·485	0·554	0·624	0·693	0·762
0·18	0·138	0·211	0·284	0·357	0·430	0·503	0·576	0·649	0·723	0·796
0·19	0·138	0·215	0·291	0·368	0·445	0·522	0·598	0·675	0·752	0·828
0·20	0·138	0·218	0·299	0·379	0·459	0·540	0·620	0·700	0·780	0·861
0·21	0·138	0·221	0·305	0·388	0·471	0·554	0·638	0·721	0·804	0·887
0·22	0·138	0·224	0·309	0·395	0·480	0·566	0·651	0·737	0·822	0·908
0·23	0·138	0·226	0·314	0·402	0·489	0·577	0·665	0·753	0·841	0·928
0·24	0·138	0·228	0·318	0·408	0·498	0·588	0·678	0·768	0·859	0·949
0·25	0·138	0·230	0·323	0·415	0·507	0·600	0·692	0·784	0·876	0·969
0·26	0·138	0·233	0·327	0·422	0·516	0·611	0·705	0·800	0·894	0·989
0·27	0·138	0·235	0·331	0·428	0·525	0·622	0·718	0·815	0·912	1·008
0·28	0·138	0·237	0·336	0·435	0·534	0·632	0·731	0·830	0·929	1·028
0·29	0·138	0·239	0·340	0·441	0·542	0·643	0·744	0·845	0·946	1·047
0·30	0·138	0·241	0·344	0·448	0·551	0·654	0·757	0·860	0·963	1·066
0·31	0·138	0·243	0·349	0·454	0·559	0·664	0·770	0·875	0·980	1·085
0·32	0·138	0·245	0·353	0·460	0·568	0·675	0·782	0·890	0·997	1·104
0·33	0·138	0·248	0·357	0·466	0·576	0·685	0·795	0·904	1·014	1·123
0·34	0·138	0·250	0·361	0·473	0·584	0·696	0·807	0·919	1·030	1·142
0·35	0·138	0·252	0·365	0·479	0·592	0·706	0·819	0·933	1·046	1·160
0·36	0·138	0·254	0·369	0·485	0·600	0·716	0·831	0·947	1·063	1·178
0·37	0·138	0·256	0·373	0·491	0·608	0·726	0·843	0·961	1·079	1·196
0·38	0·138	0·258	0·377	0·497	0·616	0·736	0·855	0·975	1·094	1·214
0·39	0·138	0·260	0·381	0·503	0·624	0·746	0·867	0·989	1·110	1·232
0·40	0·138	0·262	0·385	0·508	0·632	0·755	0·879	1·002	1·126	1·249
0·41	0·138	0·263	0·389	0·514	0·640	0·765	0·890	1·016	1·141	1·267
0·42	0·138	0·265	0·393	0·520	0·647	0·775	0·902	1·029	1·156	1·284
0·43	0·138	0·267	0·396	0·526	0·655	0·784	0·913	1·042	1·172	1·301
0·44	0·138	0·269	0·400	0·531	0·662	0·793	0·924	1·055	1·187	1·318
0·45	0·138	0·271	0·404	0·537	0·670	0·803	0·936	1·068	1·201	1·334

TABLE 59 FLEXURE — LIMITING MOMENT OF RESISTANCE FACTOR, $M_{u,lim}/b_w d^2 f_{ck}$, FOR SINGLY REINFORCED T-BEAMS, N/mm²

f_y = 500 N/mm²

D_f/d	b_f/b_w									
	1·0	2·0	3·0	4·0	5·0	6·0	7·0	8·0	9·0	10·0
0·06	0·133	0·159	0·185	0·211	0·237	0·263	0·289	0·315	0·341	0·367
0·07	0·133	0·163	0·193	0·223	0·254	0·284	0·314	0·344	0·374	0·404
0·08	0·133	0·167	0·202	0·236	0·270	0·304	0·339	0·373	0·407	0·441
0·09	0·133	0·171	0·210	0·248	0·286	0·325	0·363	0·401	0·440	0·478
0·10	0·133	0·175	0·218	0·260	0·303	0·345	0·387	0·430	0·472	0·514
0·11	0·133	0·179	0·226	0·272	0·318	0·365	0·411	0·458	0·504	0·550
0·12	0·133	0·183	0·234	0·284	0·334	0·385	0·435	0·485	0·536	0·586
0·13	0·133	0·187	0·241	0·296	0·350	0·404	0·458	0·513	0·567	0·621
0·14	0·133	0·191	0·249	0·307	0·365	0·423	0·481	0·540	0·598	0·656
0·15	0·133	0·195	0·257	0·319	0·381	0·442	0·504	0·566	0·628	0·690
0·16	0·133	0·199	0·264	0·330	0·396	0·461	0·527	0·593	0·658	0·724
0·17	0·133	0·202	0·272	0·341	0·411	0·480	0·549	0·619	0·688	0·757
0·18	0·133	0·206	0·279	0·352	0·425	0·498	0·571	0·644	0·717	0·791
0·19	0·133	0·210	0·286	0·363	0·440	0·516	0·593	0·670	0·747	0·823
0·20	0·133	0·213	0·292	0·372	0·452	0·532	0·611	0·691	0·771	0·850
0·21	0·133	0·215	0·297	0·379	0·461	0·543	0·625	0·707	0·789	0·871
0·22	0·133	0·217	0·302	0·386	0·470	0·555	0·639	0·723	0·808	0·892
0·23	0·133	0·220	0·306	0·393	0·479	0·566	0·653	0·739	0·826	0·912
0·24	0·133	0·222	0·311	0·400	0·488	0·577	0·666	0·755	0·844	0·933
0·25	0·133	0·224	0·315	0·406	0·497	0·589	0·680	0·771	0·862	0·953
0·26	0·133	0·226	0·320	0·413	0·506	0·600	0·693	0·786	0·880	0·973
0·27	0·133	0·229	0·324	0·420	0·515	0·611	0·706	0·802	0·897	0·993
0·28	0·133	0·231	0·328	0·426	0·524	0·622	0·719	0·817	0·915	1·012
0·29	0·133	0·233	0·333	0·433	0·532	0·632	0·732	0·832	0·932	1·032
0·30	0·133	0·235	0·337	0·439	0·541	0·643	0·745	0·847	0·949	1·051
0·31	0·133	0·237	0·341	0·445	0·550	0·654	0·758	0·862	0·966	1·070
0·32	0·133	0·239	0·346	0·452	0·558	0·664	0·770	0·877	0·983	1·089
0·33	0·133	0·241	0·350	0·458	0·566	0·675	0·783	0·891	1·000	1·108
0·34	0·133	0·243	0·354	0·464	0·575	0·685	0·795	0·906	1·016	1·127
0·35	0·133	0·245	0·358	0·470	0·583	0·695	0·808	0·920	1·033	1·145
0·36	0·133	0·248	0·362	0·476	0·591	0·705	0·820	0·934	1·049	1·163
0·37	0·133	0·250	0·366	0·483	0·599	0·716	0·832	0·949	1·065	1·181
0·38	0·133	0·252	0·370	0·489	0·607	0·725	0·844	0·962	1·081	1·199
0·39	0·133	0·254	0·374	0·494	0·615	0·735	0·856	0·976	1·097	1·217
0·40	0·133	0·255	0·378	0·500	0·623	0·745	0·868	0·990	1·112	1·235
0·41	0·133	0·257	0·382	0·506	0·630	0·755	0·879	1·004	1·128	1·252
0·42	0·133	0·259	0·386	0·512	0·638	0·764	0·891	1·017	1·143	1·270
0·43	0·133	0·261	0·389	0·518	0·646	0·774	0·902	1·030	1·158	1·287
0·44	0·133	0·263	0·393	0·523	0·653	0·783	0·913	1·043	1·174	1·304
0·45	0·133	0·265	0·397	0·529	0·661	0·793	0·925	1·056	1·188	1·320

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COMPRESSION MEMBERS

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3. COMPRESSION MEMBERS

3.1 AXIALLY LOADED COMPRESSION MEMBERS

All compression members are to be designed for a minimum eccentricity of load in two principal directions. Clause *24.4* of the Code specifies the following minimum eccentricity, e_{min} for the design of columns:

$e_{min} = \frac{l}{500} + \frac{D}{30}$, subject to a minimum of 2 cm.

where

l is the unsupported length of the column (*see 24.1.3* of the Code for definition of unsupported length), and

D is the lateral dimension of the column in the direction under consideration.

After determining the eccentricity, the section should be designed for combined axial load and bending (*see 3.2*). However, as a simplification, when the value of the minimum eccentricity calculated as above is less than or equal to $0{\cdot}05D$, *38.3* of the Code permits the design of short axially loaded compression members by the following equation:

$$P_u = 0{\cdot}4 f_{ck} A_c + 0{\cdot}67 f_y A_{sc}$$

where

P_u is the axial load (ultimate),
A_c is the area of concrete, and
A_{sc} is the area of reinforcement.

The above equation can be written as

$$P_u = 0{\cdot}4 f_{ck} \left(A_g - \frac{pA_g}{100} \right) + 0{\cdot}67 f_y \frac{pA_g}{100}$$

where

A_g is the gross area of cross section, and
p is the percentage of reinforcement.

Dividing both sides by A_g,

$$\frac{P_u}{A_g} = 0{\cdot}4 f_{ck} \left(1 - \frac{p}{100} \right) + 0{\cdot}67 f_y \frac{p}{100}$$

$$= 0{\cdot}4 f_{ck} + \frac{p}{100} (0{\cdot}67 f_y - 0{\cdot}4 f_{ck})$$

Charts 24 to *26* can be used for designing short columns in accordance with the above equations. In the lower section of these charts, P_u/A_g has been plotted against reinforcement percentage p for different grades of concrete. If the cross section of the column is known, P_u/A_g can be calculated and the reinforcement percentage read from the chart. In the upper section of the charts, P_u/A_g is plotted against P_u for various values of A_g. The combined use of the upper and lower sections would eliminate the need for any calculation. *This is particularly useful as an aid for deciding the sizes of columns at the preliminary design stage of multistoreyed buildings.*

Example 5 Axially Loaded Column

Determine the cross section and the reinforcement required for an axially loaded column with the following data:

Factored load	3 000 kN
Concrete grade	M20
Characteristic strength of reinforcement	415 N/mm²
Unsupported length of column	3·0 m

The cross-sectional dimensions required will depend on the percentage of reinforcement. Assuming 1·0 percent reinforcement and referring to *Chart 25*,

Required cross-sectional area of column, $A_g = 2\,700$ cm²

Provide a section of 60 × 45 cm.

$$\text{Area of reinforcement, } A_s = 1{\cdot}0 \times \frac{60 \times 45}{100} = 27 \text{ cm}^2$$

We have to check whether the minimum eccentricity to be considered is within 0·05 times the lateral dimensions of the column. In the direction of longer dimension,

$$e_{min} = \frac{l}{500} + \frac{D}{30}$$

$$= \frac{3{\cdot}0 \times 10^2}{500} + \frac{60}{30} = 0{\cdot}6 + 2{\cdot}0 = 2{\cdot}6 \text{ cm}$$

or, $e_{min}/D = 2{\cdot}6/60 = 0{\cdot}043$

In the direction of the shorter dimension,

$$e_{min} = \frac{3{\cdot}0 \times 10^2}{500} + \frac{45}{30} = 0{\cdot}6 + 1{\cdot}5$$

$$= 2{\cdot}1 \text{ cm}$$

or, $e_{min}/b = 2{\cdot}1/45 = 0{\cdot}047$

The minimum eccentricity ratio is less than 0·05 in both directions. Hence the design of the section by the simplified method of *38.3* of the Code is valid.

3.2 COMBINED AXIAL LOAD AND UNIAXIAL BENDING

As already mentioned in *3.1*, all compression members should be designed for minimum eccentricity of load. It should always be ensured that the section is designed for a moment which is not less than that due to the prescribed minimum eccentricity.

3.2.1 Assumptions—Assumptions (a), (c), (d) and (e) for flexural members (*see 2.1*) are also applicable to members subjected to combined axial load and bending. The assumption (b) that the maximum strain in concrete at the outermost compression fibre is 0·003 5 is also applicable when the neutral axis lies within the section and in the limiting case when the neutral axis lies along one edge of the section; in the latter case the strain varies from 0·003 5 at the highly compressed edge to zero at the opposite edge. For purely axial compression, the strain is assumed to be uniformly equal to 0·002 across the section [*see 38.1*(a) of the Code]. The strain distribution lines for these two cases intersect each other at a depth of $\frac{3D}{7}$ from the highly compressed edge. This point is assumed to act as a fulcrum for the strain distribution line when the neutral axis lies outside the section (*see* Fig. 7). This leads to the assumption that the strain at the highly compressed edge is 0·003 5 minus 0·75 times the strain at the least compressed edge [*see 38.1*(b) of the Code].

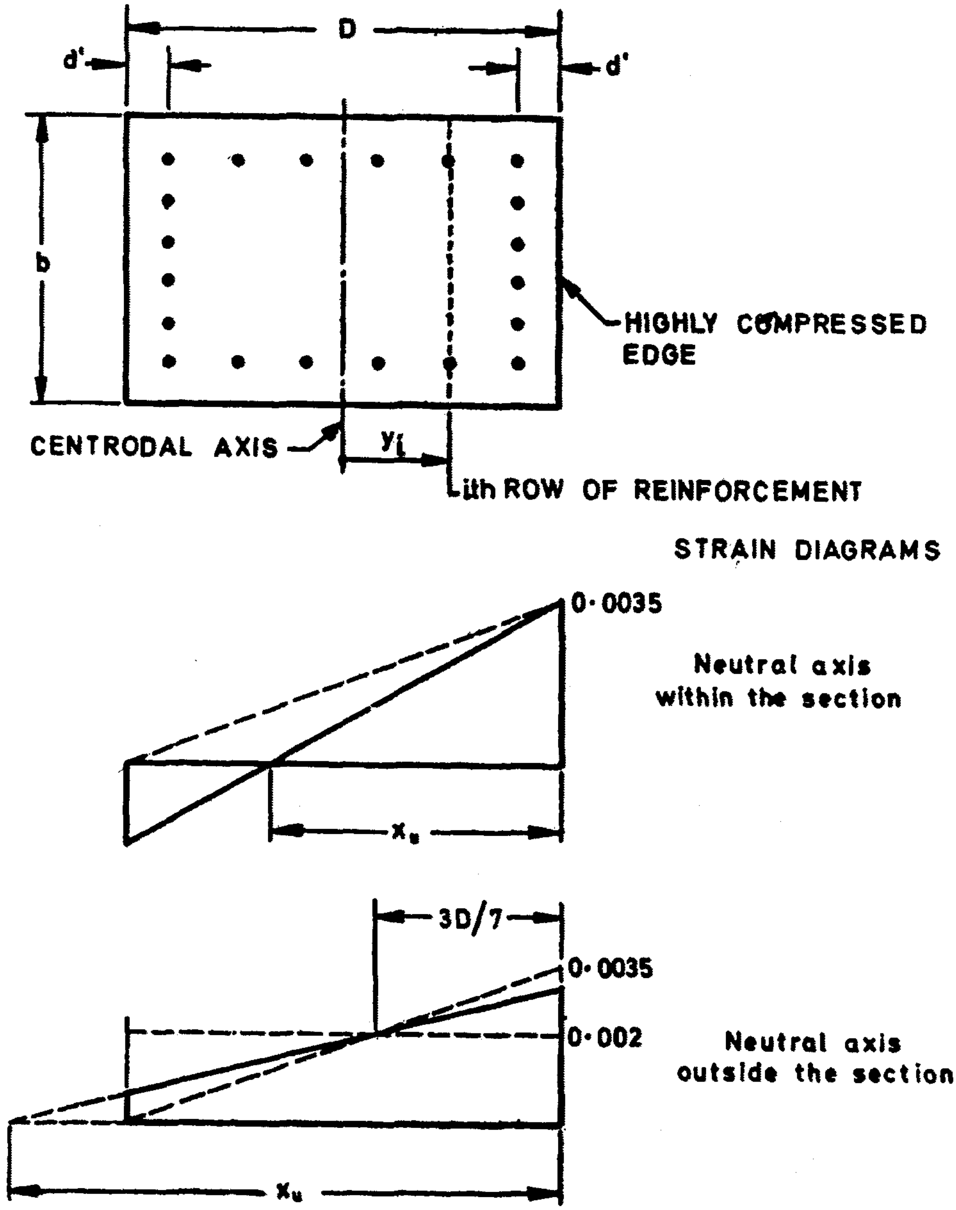


FIG. 7 COMBINED AXIAL LOAD AND UNIAXIAL BENDING

3.2.2 Stress Block Parameters When the Neutral Axis Lies Outside the Section — When the neutral axis lies outside the section, the shape of the stress block will be as indicated in Fig. 8. The stress is uniformly $0{\cdot}446 f_{ck}$ for a distance of $\frac{3D}{7}$ from the highly compressed edge because the strain is more than 0·002 and thereafter the stress diagram is parabolic.

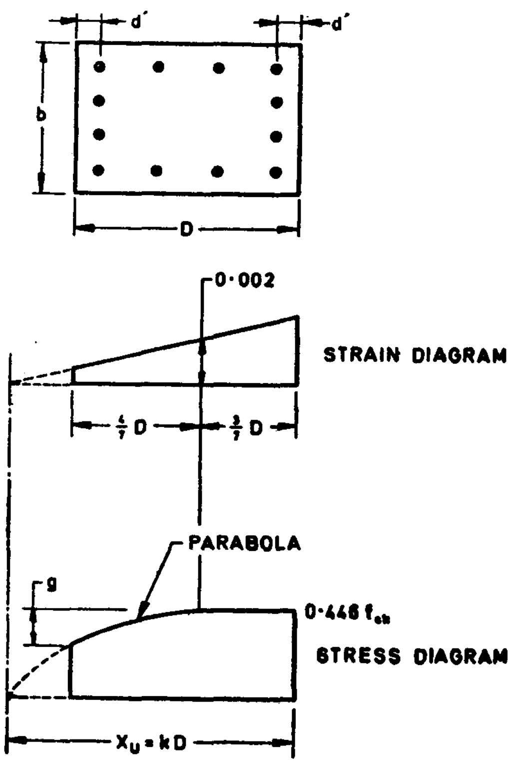


FIG. 8 STRESS BLOCK WHEN THE NEUTRAL AXIS LIES OUTSIDE THE SECTION

Let $x_u = kD$ and let g be the difference between the stress at the highly compressed edge and the stress at the least compressed edge. Considering the geometric properties of a parabola,

$$g = 0{\cdot}446 f_{ck} \left[\frac{\frac{4}{7}D}{kD - \frac{3}{7}D} \right]^2$$

$$= 0{\cdot}446 f_{ck} \left(\frac{4}{7k-3} \right)^2$$

Area of stress block

$$= 0{\cdot}446 f_{ck}\, D - \frac{g}{3}\left(\frac{4}{7}D\right)$$

$$= 0{\cdot}446 f_{ck}\, D - \frac{4}{21} gD$$

$$= 0{\cdot}446 f_{ck}\, D \left[1 - \frac{4}{21}\left(\frac{4}{7k-3}\right)^2\right]$$

The centroid of the stress block will be found by taking moments about the highly compressed edge.

Moment about the highly compressed edge

$$= 0{\cdot}446 f_{ck}\, D \left(\frac{D}{2}\right) - \frac{4}{21}\, gD \left[\frac{3}{7}D + \frac{3}{4}\left(\frac{4}{7}D\right)\right]$$

$$= 0{\cdot}446 f_{ck} \frac{D^2}{2} - \frac{8}{49} gD^2$$

The position of the centroid is obtained by dividing the moment by the area. For different values of k, the area of stress block and the position of its centroid are given in Table H.

TABLE H STRESS BLOCK PARAMETERS WHEN THE NEUTRAL AXIS LIES OUTSIDE THE SECTION

(*Clause* 3.2.2)

$k = \frac{x_u}{D}$	AREA OF STRESS BLOCK	DISTANCE OF CENTROID FROM HIGHLY COMPRESSED EDGE
(1)	(2)	(3)
1·00	$0{\cdot}361 f_{ck}\, D$	$0{\cdot}416\, D$
1·05	$0{\cdot}374 f_{ck}\, D$	$0{\cdot}432\, D$
1·10	$0{\cdot}384 f_{ck}\, D$	$0{\cdot}443\, D$
1·20	$0{\cdot}399 f_{ck}\, D$	$0{\cdot}458\, D$
1·30	$0{\cdot}409 f_{ck}\, D$	$0{\cdot}468\, D$
1·40	$0{\cdot}417 f_{ck}\, D$	$0{\cdot}475\, D$
1·50	$0{\cdot}422 f_{ck}\, D$	$0{\cdot}480\, D$
2·00	$0{\cdot}435 f_{ck}\, D$	$0{\cdot}491\, D$
2·50	$0{\cdot}440 f_{ck}\, D$	$0.495\, D$
3·00	$0{\cdot}442 f_{ck}\, D$	$0{\cdot}497\, D$
4·00	$0{\cdot}444 f_{ck}\, D$	$0{\cdot}499\, D$

NOTE — Values of stress block parameters have been tabulated for values of k up to 4·00 for information only. For construction of interaction diagrams it is generally adequate to consider values of k up to about 1·2.

3.2.3 Construction of Interaction Diagram — Design charts for combined axial compression and bending are given in the form of interaction diagrams in which curves for P_u/bDf_{ck} versus $M_u/bD^2 f_{ck}$ are plotted for different values of p/f_{ck}, where p is the reinforcement percentage.

3.2.3.1 For the case of purely axial compression, the points plotted on the y-axis of the charts are obtained as follows:

$$P_u = 0{\cdot}446\, f_{ck} bd + \frac{pbD}{100}\,(f_{sc} - 0{\cdot}446\, f_{ck})$$

$$\frac{P_u}{f_{ck} bD} = 0{\cdot}446 + \frac{p}{100\, f_{ck}}\,(f_{sc} - 0{\cdot}446\, f_{ck})$$

where

f_{sc} is the compressive stress in steel corresponding to a strain of 0·002.

The second term within parenthesis represents the deduction for the concrete replaced by the reinforcement bars. This term is usually neglected for convenience. However, as a better approximation, a constant value corresponding to concrete grade M20 has been used in the present work, so that the error is negligibly small over the range of concrete mixes normally used. An accurate consideration of this term will necessitate the preparation of separate Charts for each grade of concrete, which is not considered worthwhile.

3.2.3.2 When bending moments are also acting in addition to axial load, the points for plotting the Charts are obtained by assuming different positions of neutral axis. For each position of neutral axis, the strain distribution across the section and the stress block parameters are determined as explained earlier. The stresses in the reinforcement are also calculated from the known strains. Thereafter the resultant axial force and the moment about the centroid of the section are calculated as follows:

a) ***When the neutral axis lies outside the section***

$$P_u = C_1 f_{ck} bD + \sum_{i=1}^{n} \frac{p_i bD}{100}\,(f_{si} - f_{ci})$$

where

C_1 = coefficient for the area of stress block to be taken from Table H (*see 3.2.2*);

p_i = $\frac{A_{si}}{bD}$ where A_{si} is the area of reinforcement in the ith row;

f_{si} = stress in the ith row of reinforcement, compression being positive and tension being negative;

f_{ci} = stress in concrete at the level of the ith row of reinforcement; and

n = number of rows of reinforcement.

The above expression can be written as

$$\frac{P_u}{f_{ck}\, bD} = C_1 + \sum_{i=1}^{n} \frac{p_i}{100\, f_{ck}}\,(f_{si} - f_{ci})$$

Taking moment of the forces about the centroid of the section,

$$M_u = C_1 f_{ck}\, bD \left(\frac{D}{2} - C_2 D\,.\right) + \sum_{i=1}^{n} \frac{p_i\, bD}{100}\,(f_{si} - f_{ci}) y_i$$

where

$C_2 D$ is the distance of the centroid of the concrete stress block, measured from the highly compressed edge; and

y_i is the distance from the centroid of the section to the ith row of reinforcement; positive towards the highly compressed edge and negative towards the least compressed edge.

Dividing both sides of the equation by $f_{ck}\, bD^2$,

$$\frac{M_u}{f_{ck} bD^2} = C_1\,(0{\cdot}5 - C_2) + \sum_{i=1}^{n} \frac{p_i}{f_{ck}\, 100}\,(f_{si} - f_{ci}) \left(\frac{y_i}{D}\right)$$

b) ***When the neutral axis lies within the section***

In this case, the stress block parameters are simpler and they can be directly incorporated into the expressions which are otherwise same as for the earlier case. Thus we get the following expressions:

$$\frac{P_u}{f_{ck}\, bD} = 0{\cdot}36\, k + \sum_{i=1}^{n} \frac{p_i}{100\, f_{ck}}\,(f_{si} - f_{ci})$$

$$\frac{M_u}{f_{ck} bD^2} = 0{\cdot}36\, k\,(0{\cdot}5 - 0{\cdot}416\, k) + \sum_{i=1}^{n} \frac{p_i}{f_{ck}\, 100}\,(f_{si} - f_{ci}) \left(\frac{y_i}{D}\right)$$

where

$$k = \frac{\text{Depth of neutral axis}}{D}$$

An approximation is made for the value of f_{ci} for M20, as in the case of *3.2.3.1*. For circular sections the procedure is same as above, except that the stress block parameters given earlier are not applicable; hence the section is divided into strips and summation is done for determining the forces and moments due to the stresses in concrete.

3.2.3.3 Charts for compression with bending — Charts for rectangular sections have been given for reinforcement on two sides (*Charts 27* to *38*) and for reinforcement on four sides (*Charts 39* to *50*). The Charts for the latter case have been prepared for a section with 20 bars equally distributed on all sides, but they can be used without significant error for any other number of bars (greater than 8) provided the bars are distributed equally on the four sides. The Charts for circular section (*Charts 51* to *62*) have been prepared for a section with 8 bars, but they can generally be used for sections with any number of bars but not less than 6. Charts have been given for three grades of steel and four values of d'/D for each case mentioned above.

The dotted lines in these charts indicate the stress in the bars nearest to the tension face of the member. The line for $f_{st} = 0$ indicates that the neutral axis lies along the outermost row of reinforcement. For points lying above this line on the Chart, all the bars in the section will be in compression. The line for $f_{st} = f_{yd}$ indicates that the outermost tension reinforcement reaches the design yield strength. For points below this line, the outermost tension reinforcement undergoes inelastic deformation while successive inner rows may reach a stress of f_{yd}. It should be noted that all these stress values are at the failure condition corresponding to the limit state of collapse and not at working loads.

3.2.3.4 Charts for tension with bending — These Charts are extensions of the Charts for compression with bending. Points for plotting these Charts are obtained by assuming low values of k in the expressions given earlier. For the case of purely axial tension,

$$P_u = \frac{pbD}{100}\ (0.87 f_y)$$

$$\frac{P_u}{f_{ck}\ bD} = \frac{p}{100 f_{ck}}\ (0.87 f_y)$$

Charts 66 to *75* are given for rectangular sections with reinforcement on two sides and *Charts 76* to *85* are for reinforcement on four sides. It should be noted that these charts are meant for strength calculations only; they do not take into account crack control which may be important for tension members.

Example 6 Square Column with Uniaxial Bending

Determine the reinforcement to be provided in a square column subjected to uniaxial bending, with the following data:

Size of column	45×45 cm
Concrete mix	M 25
Characteristic strength of reinforcement	415 N/mm²
Factored load (characteristic load multiplied by γ_f)	2 500 kN
Factored moment	200 kN.m
Arrangement of reinforcement:	(a) On two sides (b) On four sides

(Assume moment due to minimum eccentricity to be less than the actual moment).

Assuming 25 mm bars with 40 mm cover,

$d' = 40 + 12.5 = 52.5$ mm $= 5.25$ cm

$d'/D = 5.25/45 = 0.12$

Charts for $d'/D = 0.15$ will be used

$$\frac{P_u}{f_{ck}\ bD} = \frac{2\,500 \times 10^3}{25 \times 45 \times 45 \times 10^2} = 0.494$$

$$\frac{M_u}{f_{ck}\ bD^2} = \frac{200 \times 10^6}{25 \times 45 \times 45 \times 45 \times 10^3} = 0.088$$

a) Reinforcement on two sides,
Referring to *Chart 33*,
$p/f_{ck} = 0.09$
Percentage of reinforcement,
$p = 0.09 \times 25 = 2.25$
$A_s = p\ bD/100 = 2.25 \times 45 \times 45/100$
$= 45.56$ cm²

b) Reinforcement on four sides
from *Chart 45*,
$p/f_{ck} = 0.10$
$p = 0.10 \times 25 = 2.5$
$A_s = 2.5 \times 45 \times 45/100 = 50.63$ cm²

Example 7 Circular Column with Uniaxial Bending

Determine the reinforcement to be provided in a circular column with the following data:

Diameter of column	50 cm
Grade of concrete	M 20
Characteristic strength of reinforcement	250 N/mm² for bars up to 20 mmϕ 240 N/mm² for bars over 20 mm ϕ

Factored load 1 600 kN
Factored moment 125 kN.m
Lateral reinforcement:

(a) Hoop reinforcement
(b) Helical reinforcement

(Assume moment due to minimum eccentricity to be less than the actual moment).

Assuming 25 mm bars with 40 mm cover,
$d' = 40 \times 12{\cdot}5 = 52{\cdot}5$ mm $= 5{\cdot}25$ cm
$d'/D = 5{\cdot}25/50 = 0{\cdot}105$

Charts for $d'/D = 0{\cdot}10$ will be used.

(a) *Column with hoop reinforcement*

$$\frac{P_u}{f_{ck}\,D^2} = \frac{1\,600 \times 10^3}{20 \times 50 \times 50 \times 10^2} = 0{\cdot}32$$
$$\frac{M_u}{f_{ck}\,D^3} = \frac{125 \times 10^6}{20 \times 50 \times 50 \times 50 \times 10^3} = 0{\cdot}05$$

Referring to *Chart 52*, for $f_y = 250$ N/mm²
$p/f_{ck} = 0{\cdot}87$
$p = 0{\cdot}87 \times 20 = 1{\cdot}74$
$A_s = p\pi D^2/400$
$= 1{\cdot}74 \times \pi \times 50 \times 50/400 = 34{\cdot}16$ cm²

For $f_y = 240$ N/mm²,
$A_s = 34{\cdot}16 \times 250/240 = 35{\cdot}58$ cm²

(b) *Column with Helical Reinforcement*

According to *38.4* of the Code, the strength of a compression member with helical reinforcement is 1·05 times the strength of a similar member with lateral ties. Therefore, the given load and moment should be divided by 1·05 before referring to the chart.

Hence,

$$\frac{P_u}{f_{ck}\,D^2} = \frac{0{\cdot}32}{1{\cdot}05} = 0{\cdot}305$$
$$\frac{M_u}{f_{ck}\,D^3} = \frac{0{\cdot}05}{1{\cdot}05} = 0{\cdot}048$$

From *Chart 52*, for $f_y = 250$ N/mm²,
$p/f_{ck} = 0{\cdot}078$
$p = 0{\cdot}078 \times 20 = 1{\cdot}56$
$A_s = 1{\cdot}56 \times \pi \times 50 \times 50/400$
$= 30{\cdot}63$ cm²
For $f_y = 240$ N/mm², $A_s = 30{\cdot}63 \times 250/240$
$= 31{\cdot}91$ cm²

According to *38.4.1* of the Code the ratio of the volume of helical reinforcement to the volume of the core shall not be less than $0{\cdot}36\,(A_g/A_c - 1)\,f_{ck}/f_y$ where A_g is the gross area of the section and A_c is the area of the core measured to the outside diameter of the helix. Assuming 8 mm dia bars for the helix,

Core diameter $= 50 - 2\,(4{\cdot}0 - 0{\cdot}8)$
$= 43{\cdot}6$ cm
$A_g/A_c = 50^2/43{\cdot}6^2 = 1{\cdot}315$
$0{\cdot}36\,(A_g/A_c - 1)\,f_{ck}/f_y$
$= 0{\cdot}36 \times 0{\cdot}315 \times 20/250$
$= 0{\cdot}0091$

$$\frac{\text{Volume of helical reinforcement}}{\text{Volume of core}} = \frac{A_{sh}\pi\,.(42{\cdot}8)}{\frac{\pi}{4}(43{\cdot}6^2)\,s_h} = \frac{0{\cdot}09\,A_{sh}}{s_h}$$

where, A_{sh} is the area of the bar forming the helix and s_h is the pitch of the helix. In order to satisfy the codal requirement,

$0{\cdot}09\,A_{sh}/s_h \geqslant 0{\cdot}0091$

For 8 mm dia bar, $A_{sh} = 0{\cdot}503$ cm²

$$s_h \leqslant \frac{0{\cdot}09 \times 0{\cdot}503}{0{\cdot}0091}$$
$$\leqslant 4{\cdot}97 \text{ cm}$$

3.3 COMPRESSION MEMBERS SUBJECT TO BIAXIAL BENDING

Exact design of members subject to axial load and biaxial bending is extremely laborious. Therefore, the Code permits the design of such members by the following equation:

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \leqslant 1{\cdot}0$$

where

M_{ux}, M_{uy} are the moments about x and y axes respectively due to design loads,

M_{ux1}, M_{uy1} are the maximum uniaxial moment capacities with an axial load P_u, bending about x and y axes respectively, and

α_n is an exponent whose value depends on P_u/P_{uz} (*see* table below) where $P_{uz} = 0{\cdot}45\,f_{ck}\,A_c + 0{\cdot}75\,f_y\,A_s$:

P_u/P_{uz}	α_n
$\leqslant 0{\cdot}2$	1·0
$\geqslant 0{\cdot}8$	2·0

For intermediate values, linear interpolation may be done. *Chart 63* can be used for evaluating P_{uz}.

For different values of P_u/P_{uz}, the appropriate value of α_n has been taken and curves for the equation $\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = 1{\cdot}0$ have been plotted in *Chart 64*.

Example 8 Rectangular Column with Biaxial Bending

Determine the reinforcement to be provided in a short column subjected to biaxial bending, with the following data:

Size of column	40 × 60 cm
Concrete mix	M 15
Characteristic strength of reinforcement	415 N/mm²
Factored load, P_u	1 600 kN
Factored moment acting parallel to the larger dimension, M_{ux}	120 kN
Factored moment acting parallel to the shorter dimension, M_{uy}	90 kN

Moments due to minimum eccentricity are less than the values given above.

Reinforcement is distributed equally on four sides.

As a first trial assume the reinforcement percentage, $p = 1{\cdot}2$

$p/f_{ck} = 1{\cdot}2/15 = 0{\cdot}08$

Uniaxial moment capacity of the section about xx-axis:

$$d'/D = \frac{5{\cdot}25}{60} = 0{\cdot}087\,5$$

Chart for $d'/D = 0{\cdot}1$ will be used.

$$P_u/f_{ck}\,bD = \frac{1\,600 \times 10^3}{15 \times 40 \times 60 \times 10^2} = 0{\cdot}444$$

Referring to *Chart 44*,

$M_u/f_{ck}\,bD^2 = 0{\cdot}09$

$$\therefore \quad M_{ux1} = 0{\cdot}09 \times 15 \times 40 \times 60^2 \times 10^3/10^6 = 194{\cdot}4 \text{ kN.m}$$

Uniaxial moment capacity of the section about yy-axis:

$$d'/D = \frac{5{\cdot}25}{40} = 0{\cdot}131$$

Chart for $d'/D = 0{\cdot}15$ will be used.

Referring to *Chart 45*,

$M_u/f_{ck}\,bD^2 = 0{\cdot}083$

$$\therefore \quad M_{uy1} = 0{\cdot}083 \times 15 \times 60 \times 40^2 \times 10^3/10^6 = 119{\cdot}52 \text{ kN.m}$$

Calculation of P_{uz}:

Referring to *Chart 63* corresponding to $p = 1{\cdot}2, f_y = 415$ and $f_{ck} = 15$,

$$\frac{P_{uz}}{A_g} = 10{\cdot}3 \text{ N/mm}^2$$

$$\therefore \quad P_{uz} = 10{\cdot}3\,A_g = 10{\cdot}3 \times 40 \times 60 \times 10^2/10^3 \text{ kN} = 2\,472 \text{ kN}$$

$$\frac{P_u}{P_{uz}} = \frac{1\,600}{2\,472} = 0{\cdot}647$$

$$\frac{M_{ux}}{M_{ux1}} = \frac{120}{194{\cdot}4} = 0{\cdot}617$$

$$\frac{M_{uy}}{M_{uy1}} = \frac{90}{119{\cdot}52} = 0{\cdot}753$$

Referring to *Chart 64*, the permissible value of $\frac{M_{ux}}{M_{ux1}}$ corresponding to the above values of $\frac{M_{uy}}{M_{uy1}}$ and $\frac{P_u}{P_{uz}}$ is equal to 0·58.

The actual value of 0·617 is only slightly higher than the value read from the Chart. This can be made up by slight increase in reinforcement.

$$A_s = \frac{1{\cdot}2 \times 40 \times 60}{100} = 28{\cdot}8 \text{ cm}^2$$

12 bars of 18 mm will give $A_s = 30{\cdot}53 \text{ cm}^2$

Reinforcement percentage provided,

$$p = \frac{30{\cdot}53 \times 100}{60 \times 40} = 1{\cdot}27$$

With this percentage, the section may be rechecked as follows:

$p/f_{ck} = 1{\cdot}27/15 = 0{\cdot}084\,7$

Referring to *Chart 44*,

$$\frac{M_u}{f_{ck}\,bD^2} = 0{\cdot}095$$

$$\therefore \quad M_{ux1} = 0{\cdot}095 \times 15 \times 40 \times 60^2 \times 10^3/10^6 = 205{\cdot}2 \text{ kN.m}$$

Referring to *Chart 45*

$$\frac{M_u}{f_{ck}\,bD^2} = 0{\cdot}085$$

$$\therefore \quad M_{uy1} = 0{\cdot}085 \times 15 \times 60 \times 40^2 \times 10^3/10^6 = 122{\cdot}4 \text{ kN.m}$$

Referring to *Chart 63*,

$$\frac{P_{uz}}{A_g} = 10{\cdot}4 \text{ N/mm}^2$$

$$\therefore \quad P_{uz} = 10{\cdot}4 \times 60 \times 40 \times 10^2/10^3 = 2\,496 \text{ kN}$$

$$P_u/P_{uz} = \frac{1\,600}{2\,496} = 0{\cdot}641$$

$$M_{ux}/M_{ux1} = \frac{120}{205.2} = 0{\cdot}585$$

$$M_{uy}/M_{uy1} = \frac{90}{122{\cdot}4} = 0{\cdot}735$$

Referring to *Chart 64*,

Corresponding to the above values of $\frac{M_{uy}}{M_{uy1}}$ and $\frac{P_u}{P_{uz}}$, the permissible value of $\frac{M_{ux}}{M_{ux1}}$ is 0·6.

Hence the section is O.K.

3.4 SLENDER COMPRESSION MEMBERS

When the slenderness ratio $\frac{l_{ex}}{D}$ or $\frac{l_{ey}}{b}$ of a compression member exceeds 12, it is considered to be a slender compression member (*see 24.1.2* of the Code); l_{ex} and l_{ey} being the effective lengths with respect to the major axis and minor axis respectively. When a compression member is slender with respect to the major axis, an additional moment M_{ax} given by the following equation (modified as indicated later) should be taken into account in the design (*see 38.7.1* of the Code):

$$M_{ax} = \frac{P_u D}{2\,000}\left(\frac{l_{ex}}{D}\right)^2$$

Similarly, if the column is slender about the minor axis an additional moment M_{ay} should be considered.

$$M_{ay} = \frac{P_u b}{2\,000}\left(\frac{l_{ey}}{b}\right)^2$$

The expressions for the additional moments can be written in the form of eccentricities of load, as follows:

$$M_{ax} = P_u\, e_{ax}$$

where

$$e_{ax} = \frac{D}{2\,000}\left(\frac{l_{ex}}{D}\right)^2$$

$$\frac{e_{ax}}{D} = \frac{1}{2\,000}\left(\frac{l_{ex}}{D}\right)^2$$

Table I gives the values $\frac{e_{ax}}{D}$ or $\frac{e_{ay}}{b}$ for different values of slenderness ratio.

TABLE I ADDITIONAL ECCENTRICITY FOR SLENDER COMPRESSION MEMBERS

(*Clause* 3.4)

l_{ex}/D or l_{ey}/b	e_{ax}/D or e_{ay}/b	l_{ex}/D or l_{ey}/b	e_{ax}/D or e_{ay}/b
(1)	(2)	(3)	(4)
12	0·072	25	0·313
13	0·085	30	0·450
14	0·098	35	0·613
15	0·113	40	0·800
16	0·128	45	1·013
17	0·145	50	1·250
18	0·162	55	1·513
19	0·181	60	1·800
20	0·200		

In accordance with *38.7.1.1* of the Code, the additional moments may be reduced by the multiplying factor k given below:

$$k = \frac{P_{uz} - P_u}{P_{uz} - P_b} \leqslant 1$$

where

$P_{uz} = 0{\cdot}45\, f_{ck}\, A_c + 0{\cdot}75\, f_y\, A_s$, which may be obtained from *Chart 63*, and P_b is the axial load corresponding to the condition of maximum compressive strain of 0·003 5 in concrete and tensile strain of 0·002 in outermost layer of tension steel.

Though this modification is optional according to the Code, it should always be taken advantage of, since the value of k could be substantially less than unity.

The value of P_b will depend on arrangement of reinforcement and the cover ratio d'/D, in addition to the grades of concrete and steel. The values of the coefficients required for evaluating P_b for various cases are given in *Table 60*. The values given in *Table 60* are based on the same assumptions as for members with axial load and uniaxial bending.

The expression for k can be written as follows:

$$k = \frac{1 - P_u/P_{uz}}{1 - P_b/P_{uz}} \leqslant 1$$

Chart 65 can be used for finding the ratio of k after calculating the ratios P_u/P_{uz} and P_b/P_{uz}.

Example 9 Slender Column (with biaxial bending)

Determine the reinforcement required for a column which is restrained against sway, with the following data:

Size of column	40 × 30 cm
Concrete grade	M 30
Characteristic strength of reinforcement	415 N/mm²
Effective length for bending parallel to larger dimension, l_{ex}	6·0 m
Effective length for bending parallel to shorter dimension, l_{ey}	5·0 m
Unsupported length	7·0 m
Factored load	1 500 kN
Factored moment in the direction of larger dimension	40 kN.m at top and 22·5 kN.m at bottom

Factored moment in the direction of shorter dimension	30 kN.m at top and 20 kN.m at bottom

The column is bent in double curvature. Reinforcement will be distributed equally on four sides.

$$\frac{l_{ex}}{D} = \frac{6{\cdot}0 \times 100}{40} = 15{\cdot}0 > 12$$

$$\frac{l_{ey}}{b} = \frac{5{\cdot}0 \times 100}{30} = 16{\cdot}7 > 12$$

Therefore the column is slender about both the axes.

From Table I,

For $\frac{l_{ex}}{D} = 15$, $e_x/D = 0{\cdot}113$

For $\frac{l_{ey}}{b} = 16{\cdot}7$, $e_y/b = 0{\cdot}140$

Additional moments:

$$M_{ax} = P_u e_x = 1\,500 \times 0{\cdot}113 \times \frac{40}{100} = 67{\cdot}8 \text{ kN.m}$$

$$M_{ay} = P_u e_y = 1\,500 \times 0{\cdot}14 \times \frac{30}{100} = 63{\cdot}0 \text{ kN.m}$$

The above moments will have to be reduced in accordance with *38.7.1.1* of the Code; but multiplication factors can be evaluated only if the reinforcement is known.

For first trial, assume $p = 3{\cdot}0$ (with reinforcement equally on all the four sides).

$A_g = 40 \times 30 = 1\,200 \text{ cm}^2$

From *Chart 63*, $P_{uz}/A_g = 22{\cdot}5 \text{ N/mm}^2$

$\therefore\ P_{uz} = 22{\cdot}5 \times 1\,200 \times 10^2/10^3 = 2\,700 \text{ kN}$

Calculation of P_b:

Assuming 25 mm dia bars with 40 mm cover

d'/D (about xx-axis) $= \frac{5{\cdot}25}{40} = 0{\cdot}13$

Chart or Table for $d'/d = 0{\cdot}15$ will be used.

d'/D (about yy-axis) $= \frac{5{\cdot}25}{30} = 0{\cdot}17$

Chart or Table for $d'/d = 0{\cdot}20$ will be used.

From *Table 60*,

$$P_b \text{ (about } xx\text{-axis)} = \left(k_1 + k_2 \frac{p}{f_{ck}}\right) f_{ck} bD$$

$$P_{bx} = \left(0{\cdot}196 + 0{\cdot}203 \times \frac{3}{30}\right) \times 30 \times 30 \times 40 \times 10^2/10^3 = 779 \text{ kN}$$

$$P_b \text{ (about } yy\text{-axis)} = \left(0{\cdot}184 + \frac{0{\cdot}028 \times 3}{30}\right) \times 40 \times 30 \times 30 \times 10^2/10^3$$

$P_{by} = 672$ kN

$$\therefore\ k_x = \frac{P_{uz} - P_u}{P_{uz} - P_{bx}} = \frac{2\,700 - 1\,500}{2\,700 - 779} = 0{\cdot}625$$

$$k_y = \frac{P_{uz} - P_u}{P_{uz} - P_{by}} = \frac{2\,700 - 1\,500}{2\,700 - 672} = 0{\cdot}592$$

The additional moments calculated earlier, will now be multiplied by the above values of k.

$M_{ax} = 67{\cdot}8 \times 0{\cdot}625 = 42{\cdot}4$ kN.m
$M_{ay} = 63{\cdot}0 \times 0{\cdot}592 = 37{\cdot}3$ kN.m

The additional moments due to slenderness effects should be added to the initial moments after modifying the initial moments as follows (*see* Note 1 under *38.7.1* of the Code):

$M_{ux} = (0{\cdot}6 \times 40 - 0{\cdot}4 \times 22{\cdot}5) = 15{\cdot}0$ kN.m
$M_{uy} = (0{\cdot}6 \times 30 - 0{\cdot}4 \times 20) = 10{\cdot}0$ kN.m

The above actual moments should be compared with those calculated from minimum eccentricity consideration (*see 24.4* of the Code) and greater value is to be taken as the initial moment for adding the additional moments.

$$e_x = \frac{l}{500} + \frac{D}{30} = \frac{700}{500} + \frac{40}{30} = 2{\cdot}73 \text{ cm}$$

$$e_y = \frac{l}{500} + \frac{b}{30} = \frac{700}{500} + \frac{30}{30} = 2{\cdot}4 \text{ cm}$$

Both e_x and e_y are greater than 2·0 cm.

Moments due to minimum eccentricity:

$$M_{ux} = 1\,500 \times \frac{2{\cdot}73}{100} = 41{\cdot}0 \text{ kN.m} > 15{\cdot}0 \text{ kN.m}$$

$$M_{uy} = 1\,500 \times \frac{2{\cdot}4}{100} = 36{\cdot}0 \text{ kN.m} > 10{\cdot}0 \text{ kN.m}$$

$\therefore$ Total moments for which the column is to be designed are:

$M_{ux} = 41{\cdot}0 + 42{\cdot}4 = 83{\cdot}4$ kN.m
$M_{uy} = 36{\cdot}0 + 37{\cdot}3 = 73{\cdot}3$ kN.m

The section is to be checked for biaxial bending.

$$P_u/f_{ck}\,bD = \frac{1\,500 \times 10^3}{30 \times 30 \times 40 \times 10^2} = 0{\cdot}417$$

$$p/f_{ck} = \frac{3 \cdot 0}{30} = 0 \cdot 10$$

Referring to *Chart 45* ($d'/D = 0 \cdot 15$),

$$M_u/f_{ck}\,bD^2 = 0 \cdot 104$$

$$\therefore \quad M_{ux1} = 0 \cdot 104 \times 30 \times 30 \times 40 \times 40 \times 10^3/10^6$$

$$= 149 \cdot 8 \text{ kN.m}$$

Referring to *Chart 46* ($d'/D = 0 \cdot 20$),

$$M_u/f_{ck}\,bD^2 = 0 \cdot 096$$

$$\therefore \quad M_{uy1} = 0 \cdot 096 \times 30 \times 40 \times 30 \times 30 \times 10^3/10^6$$

$$= 103 \cdot 7 \text{ kN.m}$$

$$\frac{M_{ux}}{M_{ux1}} = \frac{83 \cdot 4}{149 \cdot 8} = 0 \cdot 56$$

$$\frac{M_{uy}}{M_{uy1}} = \frac{73 \cdot 3}{103 \cdot 7} = 0 \cdot 71$$

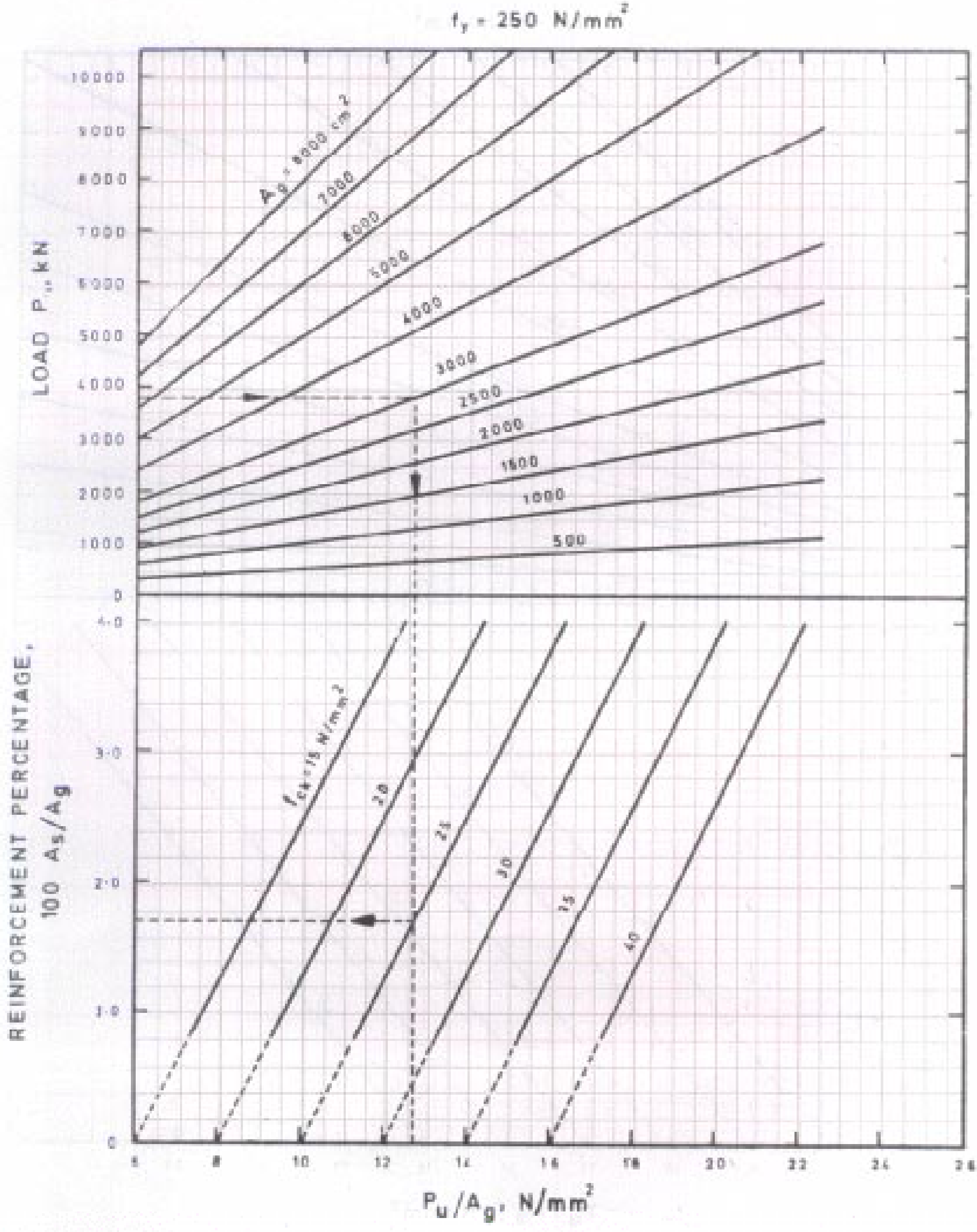
$$P_u/P_{uz} = \frac{1\,500}{2\,700} = 0 \cdot 56$$

Referring to *Chart 64*, the maximum allowable value of M_{ux}/M_{ux1} corresponding to the above values of M_{uy}/M_{uy1} and P_u/P_{uz} is 0·58 which is slightly higher than the actual value of 0·56. The assumed reinforcement of 3·0 percent is therefore satisfactory.

$$A_s = pbD/100 = 3 \cdot 0 \times 30 \times 40/100$$

$$= 36 \cdot 0 \text{ cm}^2$$

Chart 24 AXIAL COMPRESSION



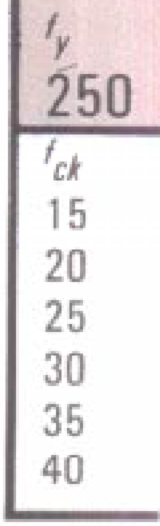
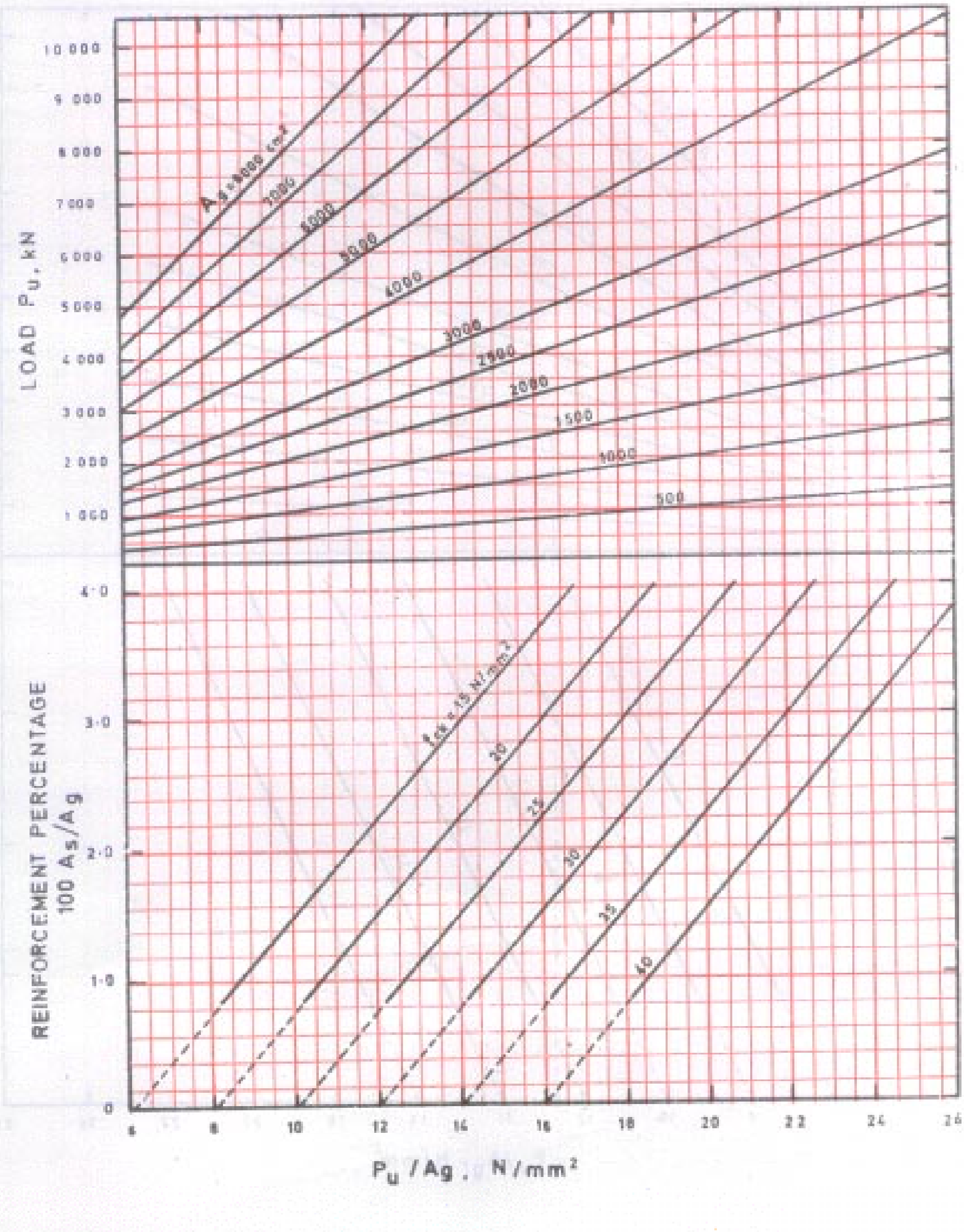


Chart 25 AXIAL COMPRESSION

$f_y = 415\ N/mm^2$



f_y
500

f_{ck}
15
20
25
30
35
40

Chart 26 AXIAL COMPRESSION

$f_y = 500$ N/mm²

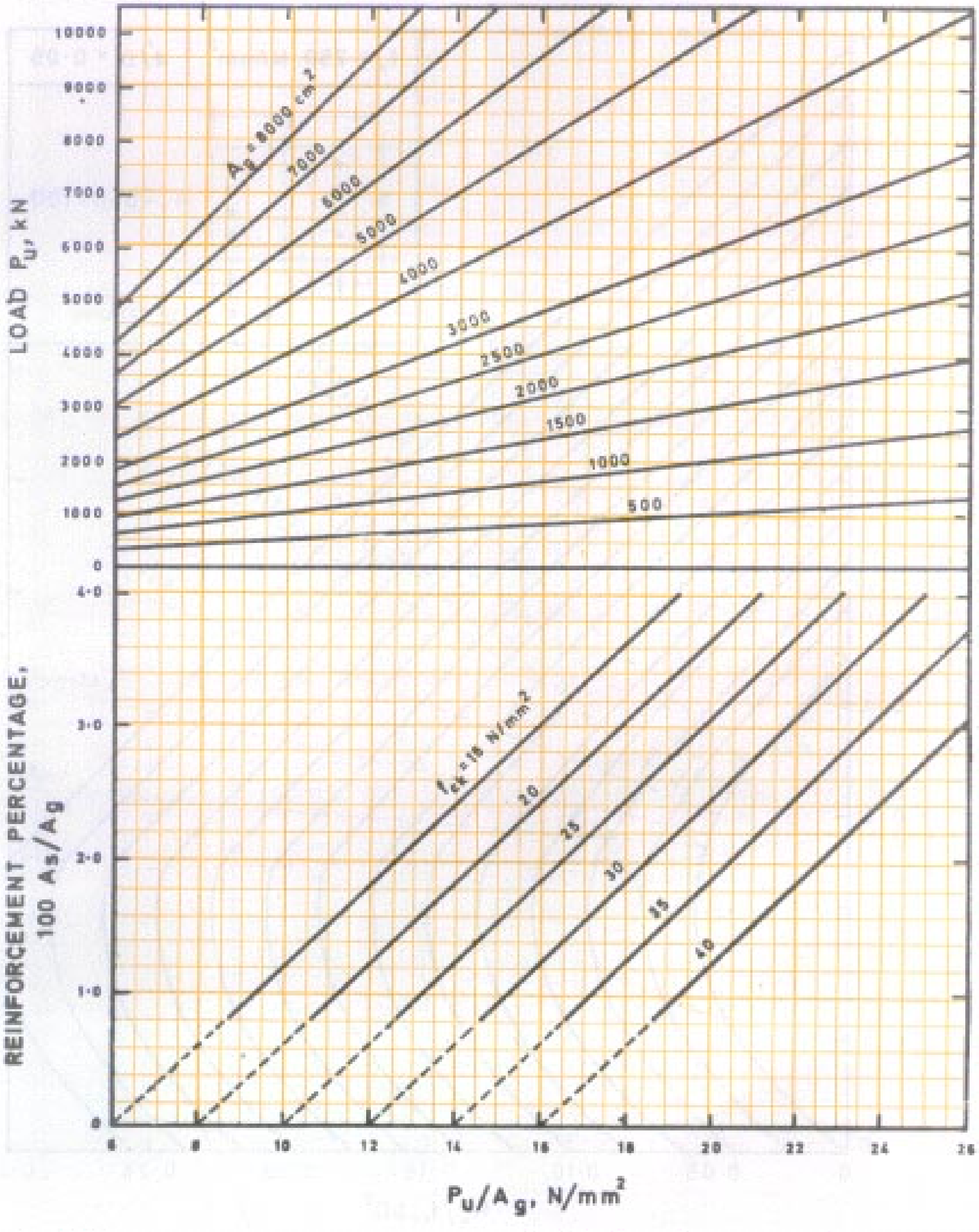


Chart 27 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

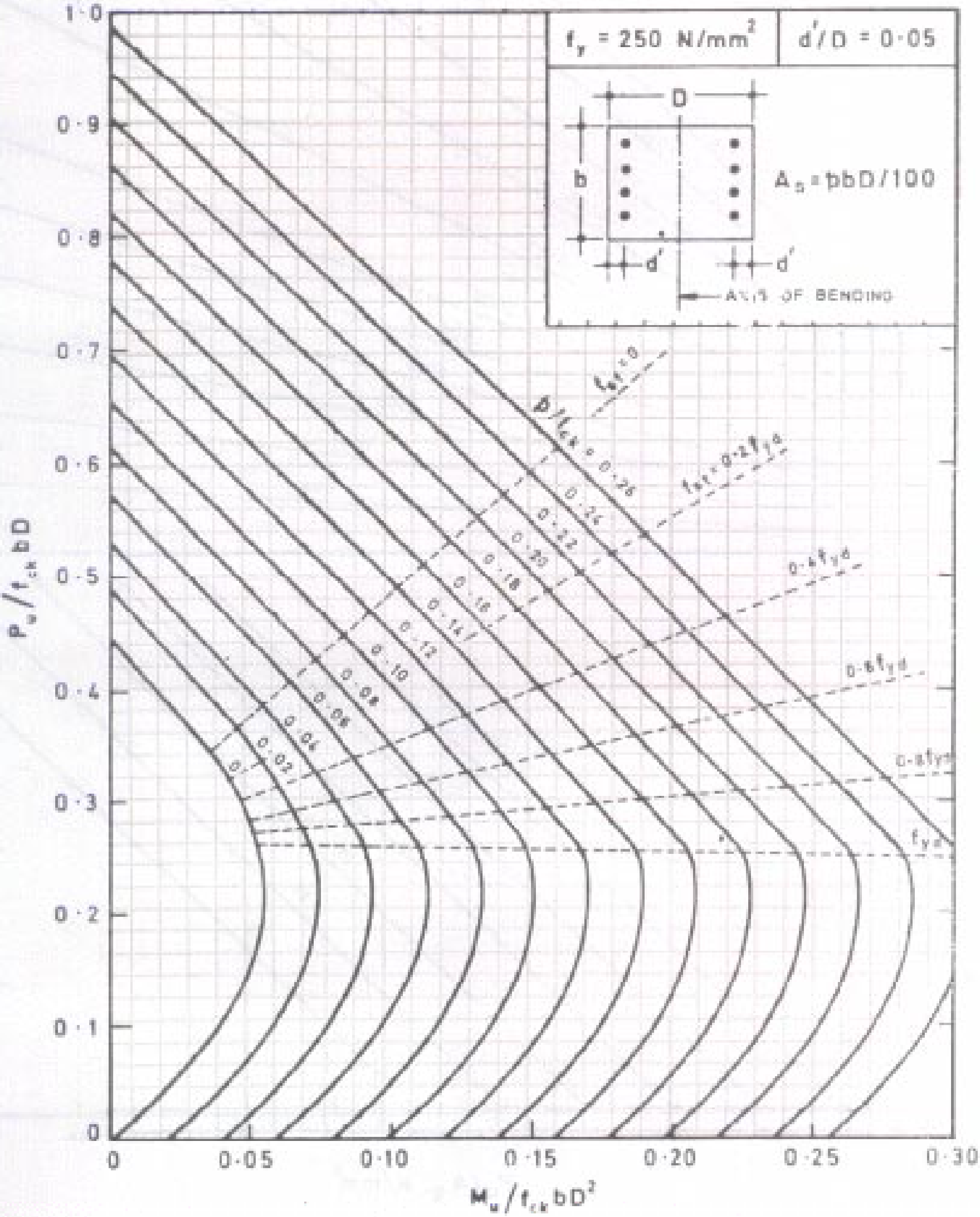


Chart 28 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

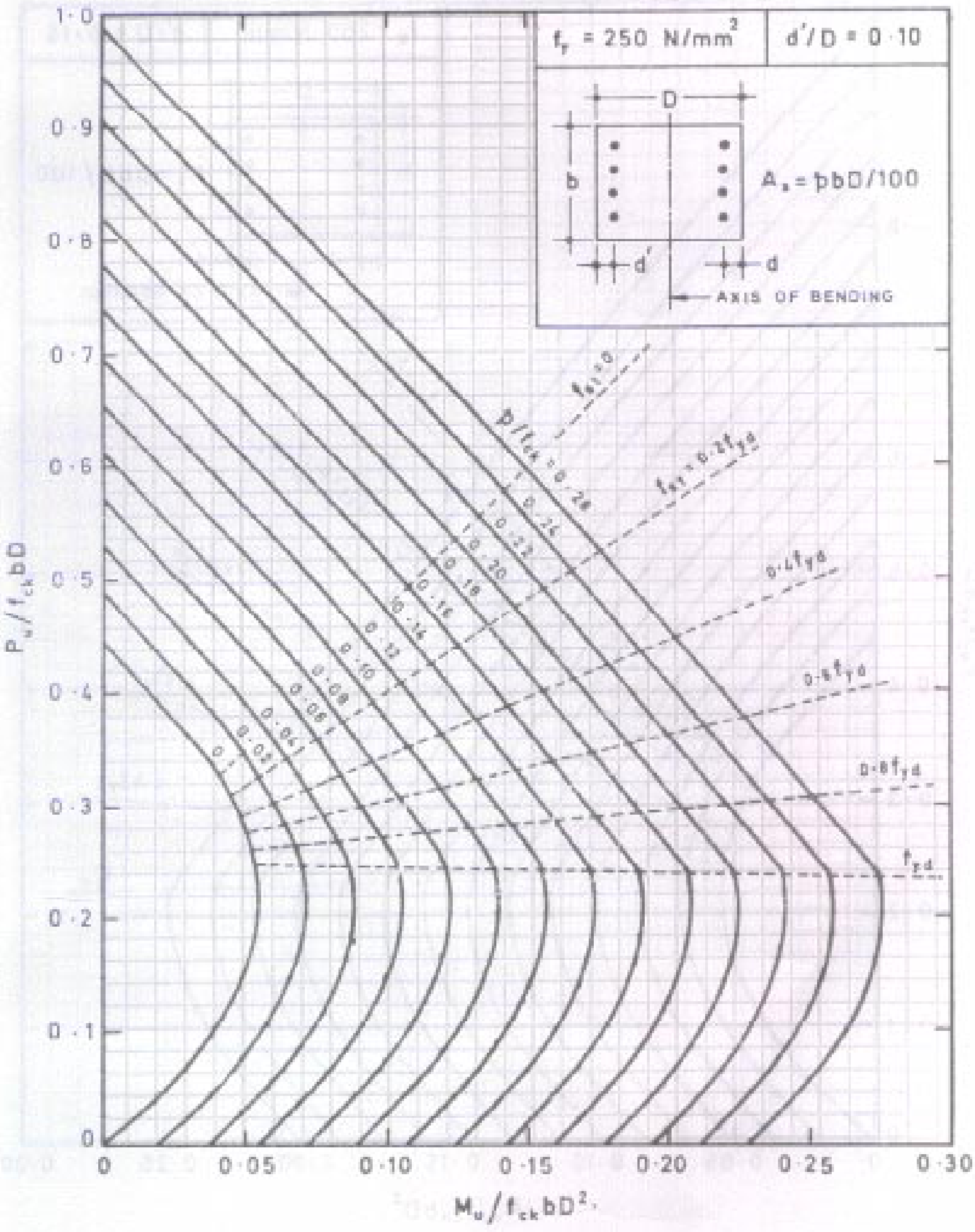


Chart 29 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

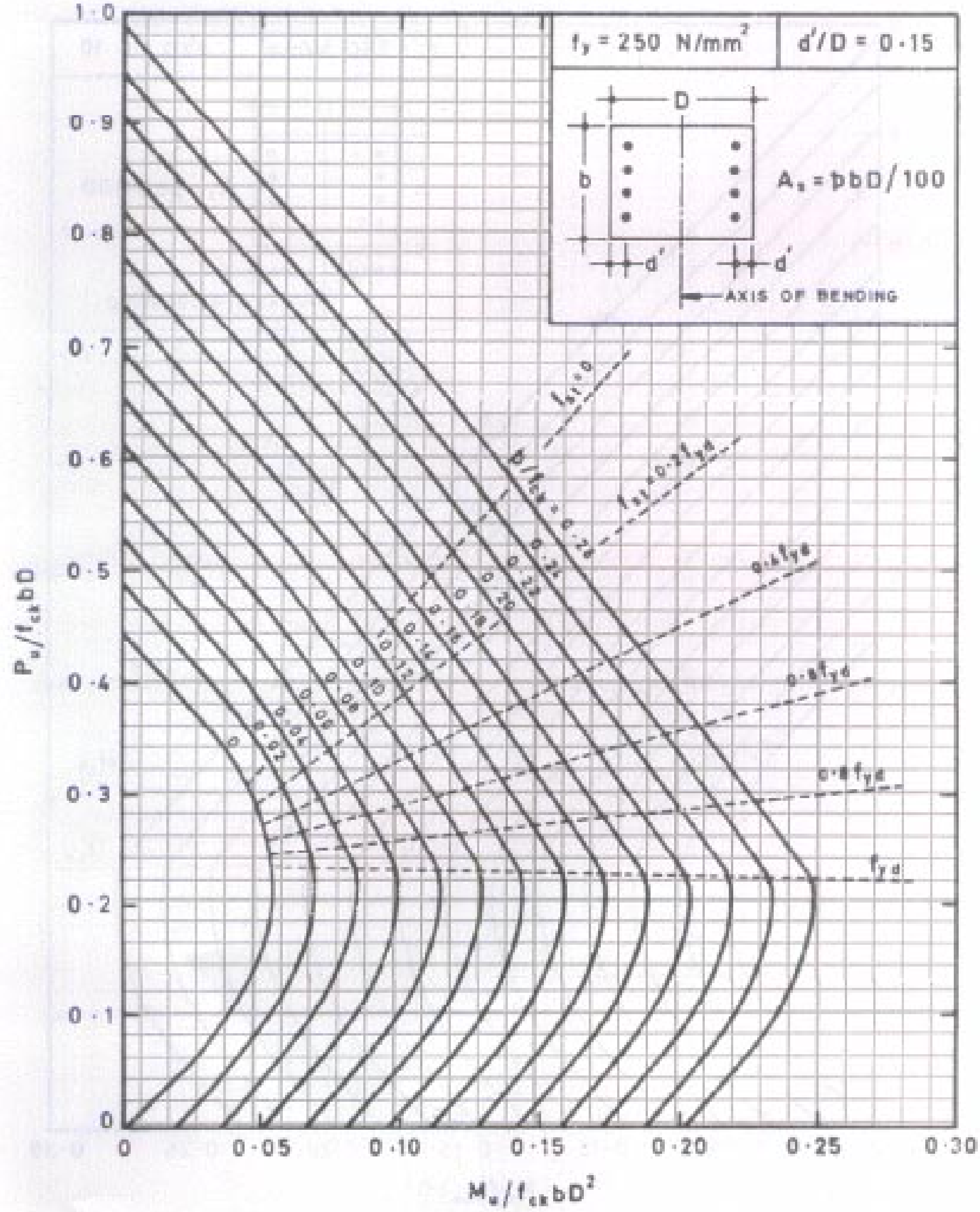


Chart 30 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

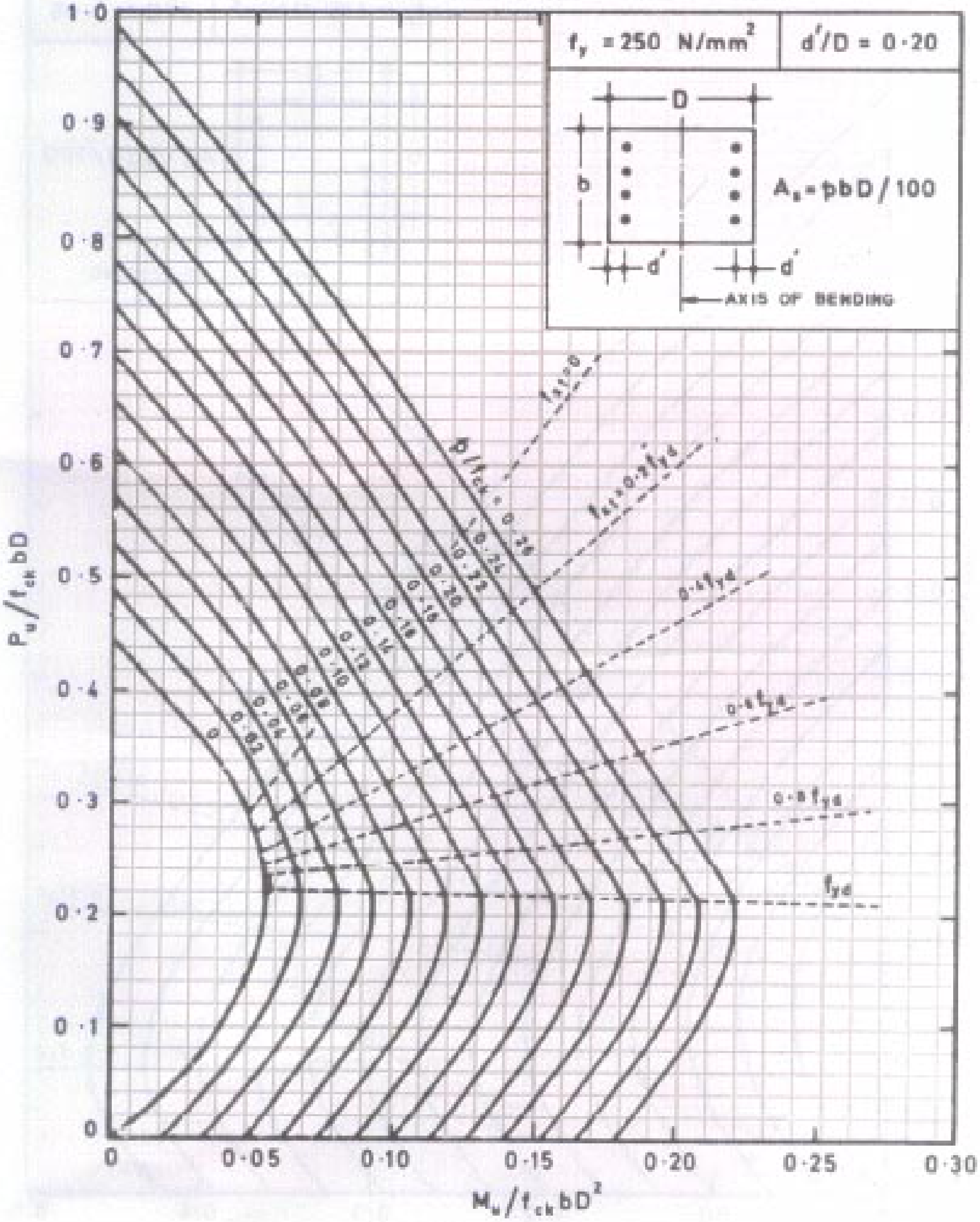


Chart 31 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

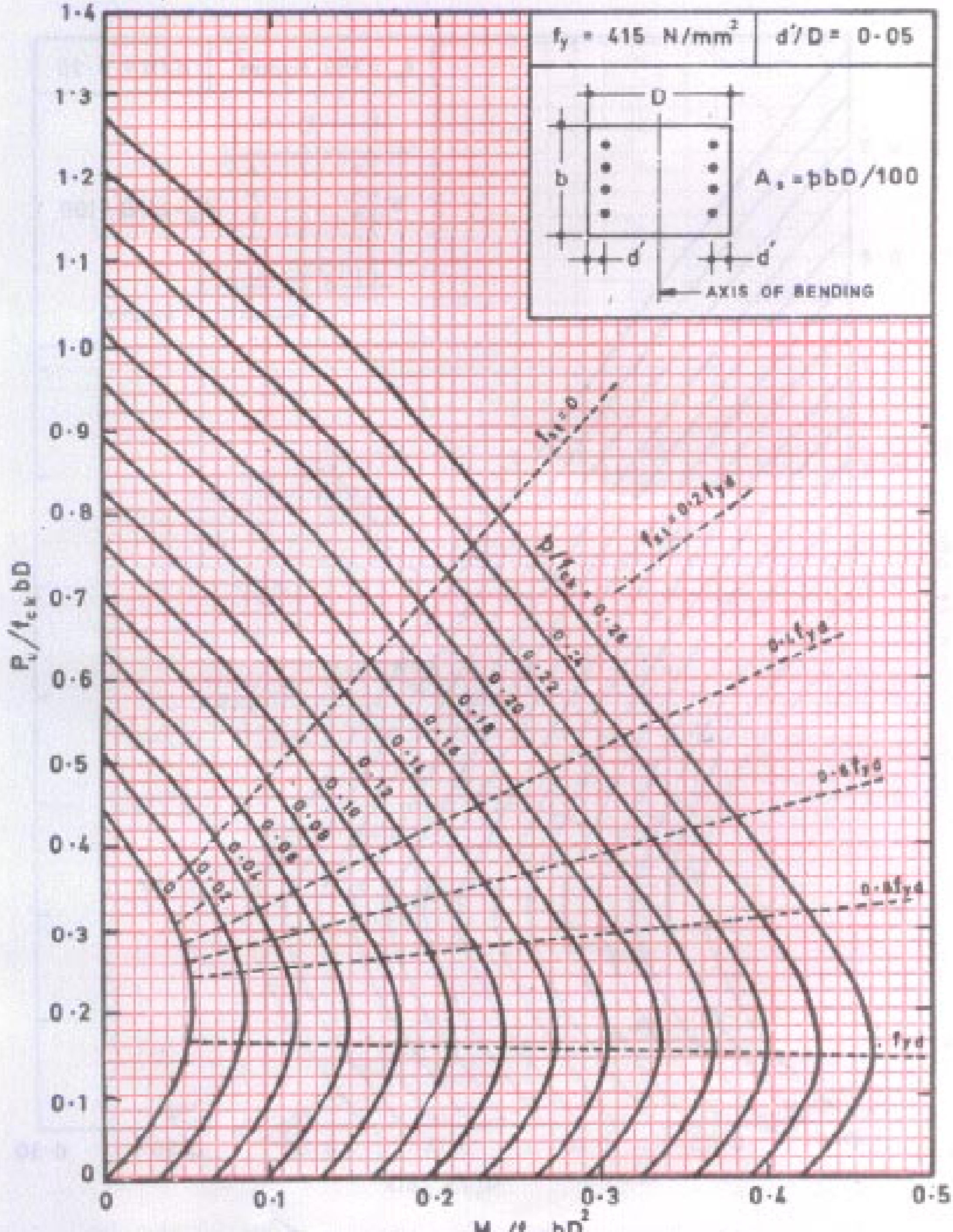


Chart 32 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributud Equally on Two Sides

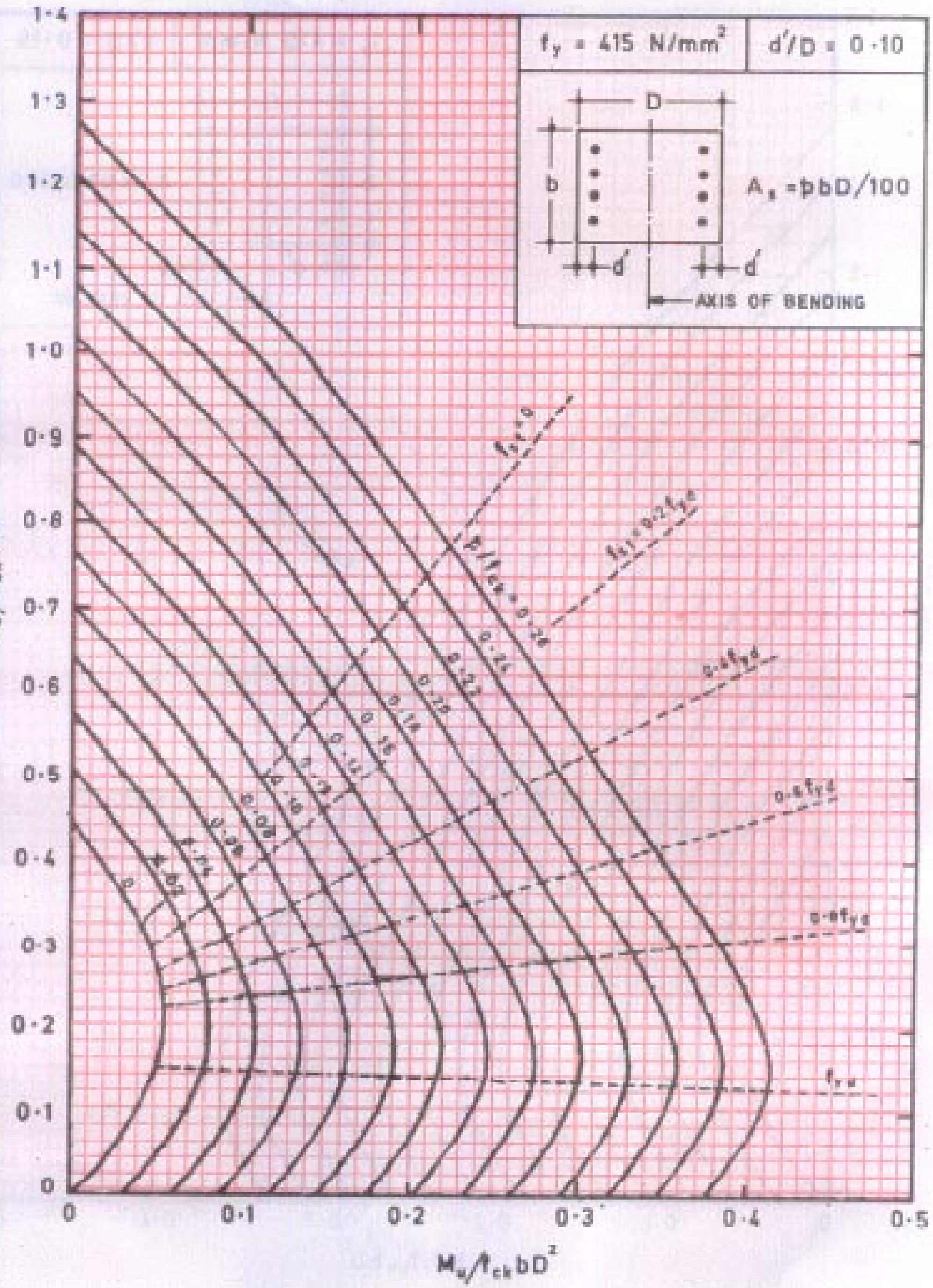


Chart 33 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

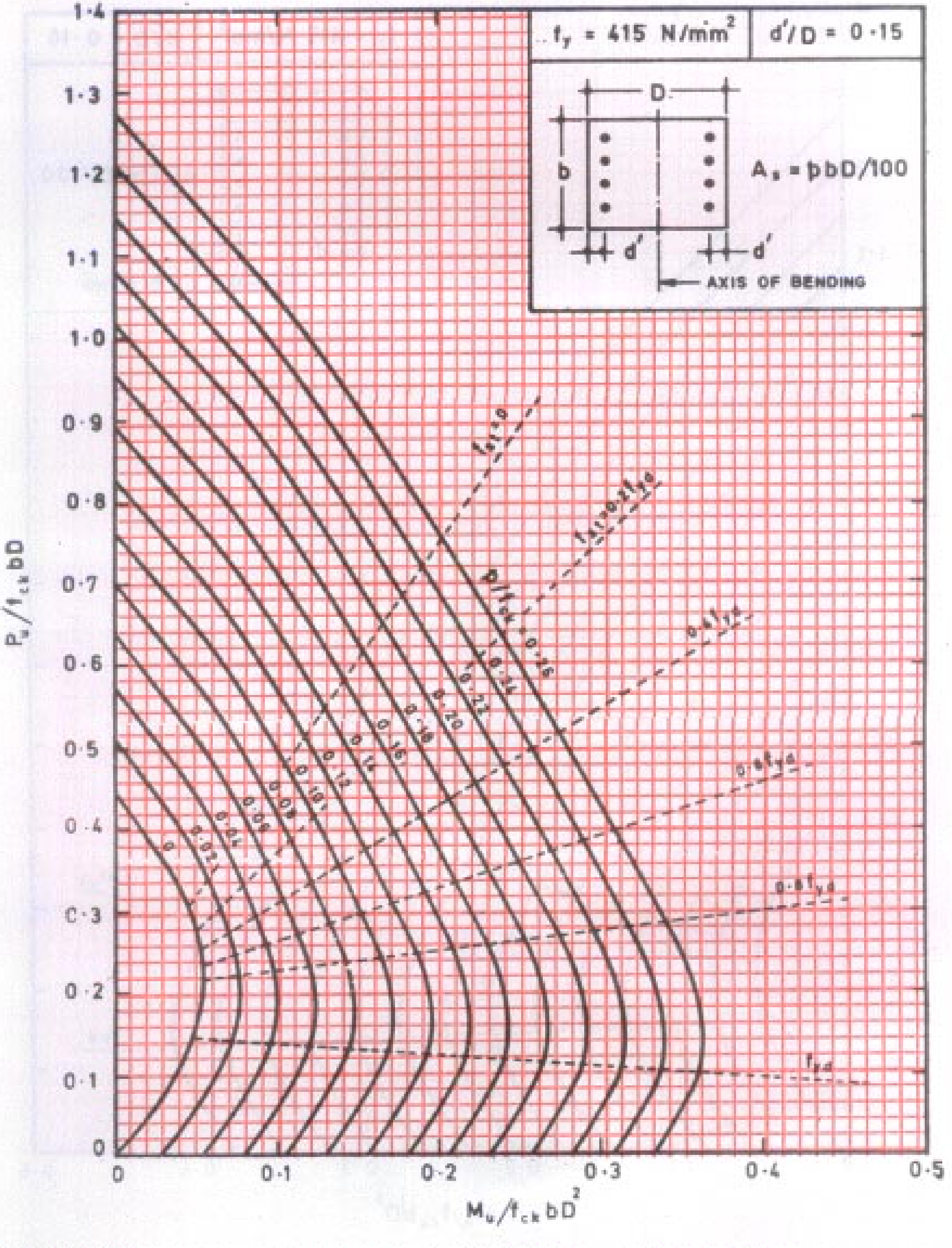


Chart 34 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

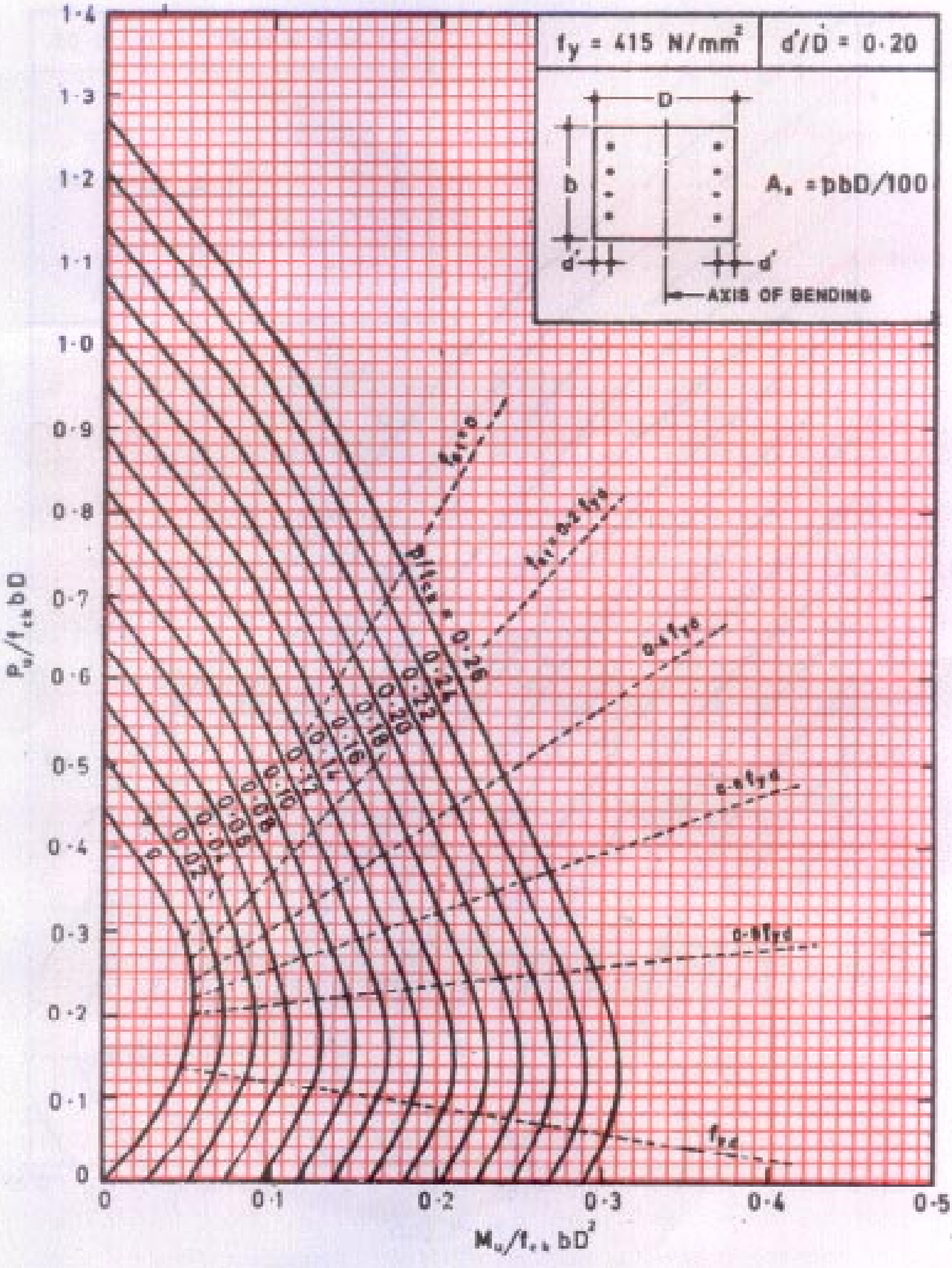


Chart 35 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

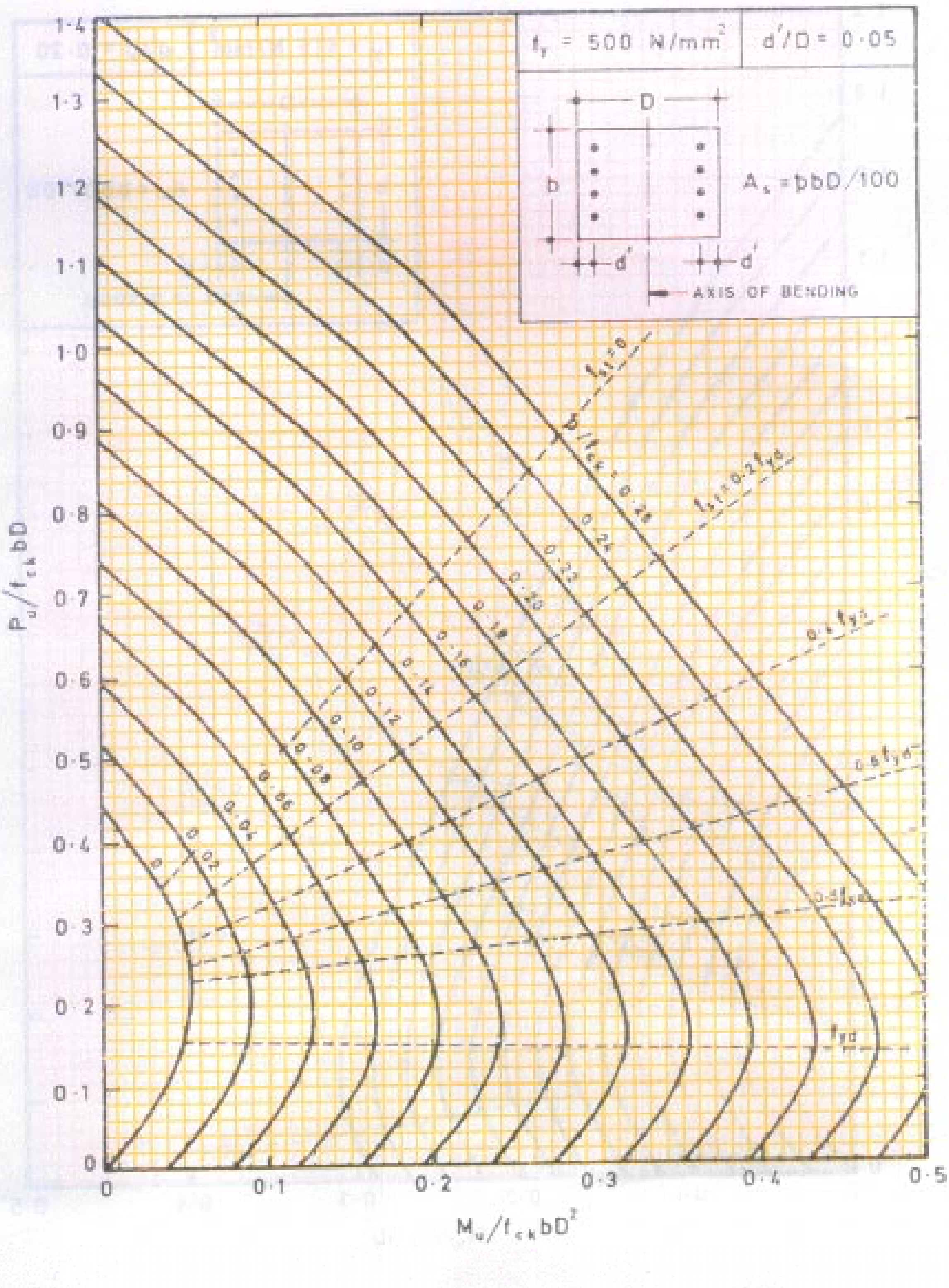


Chart 36 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

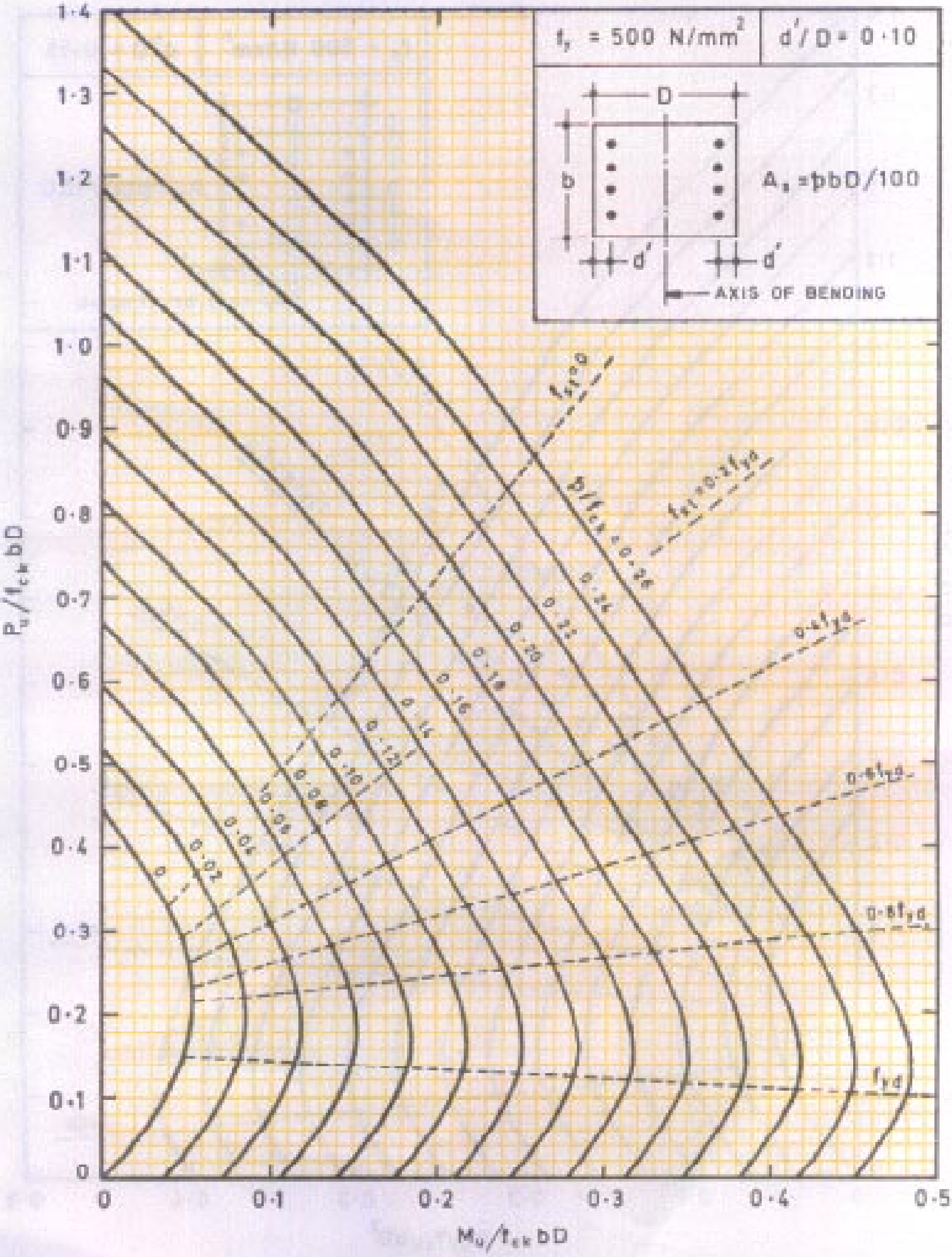


Chart 37 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

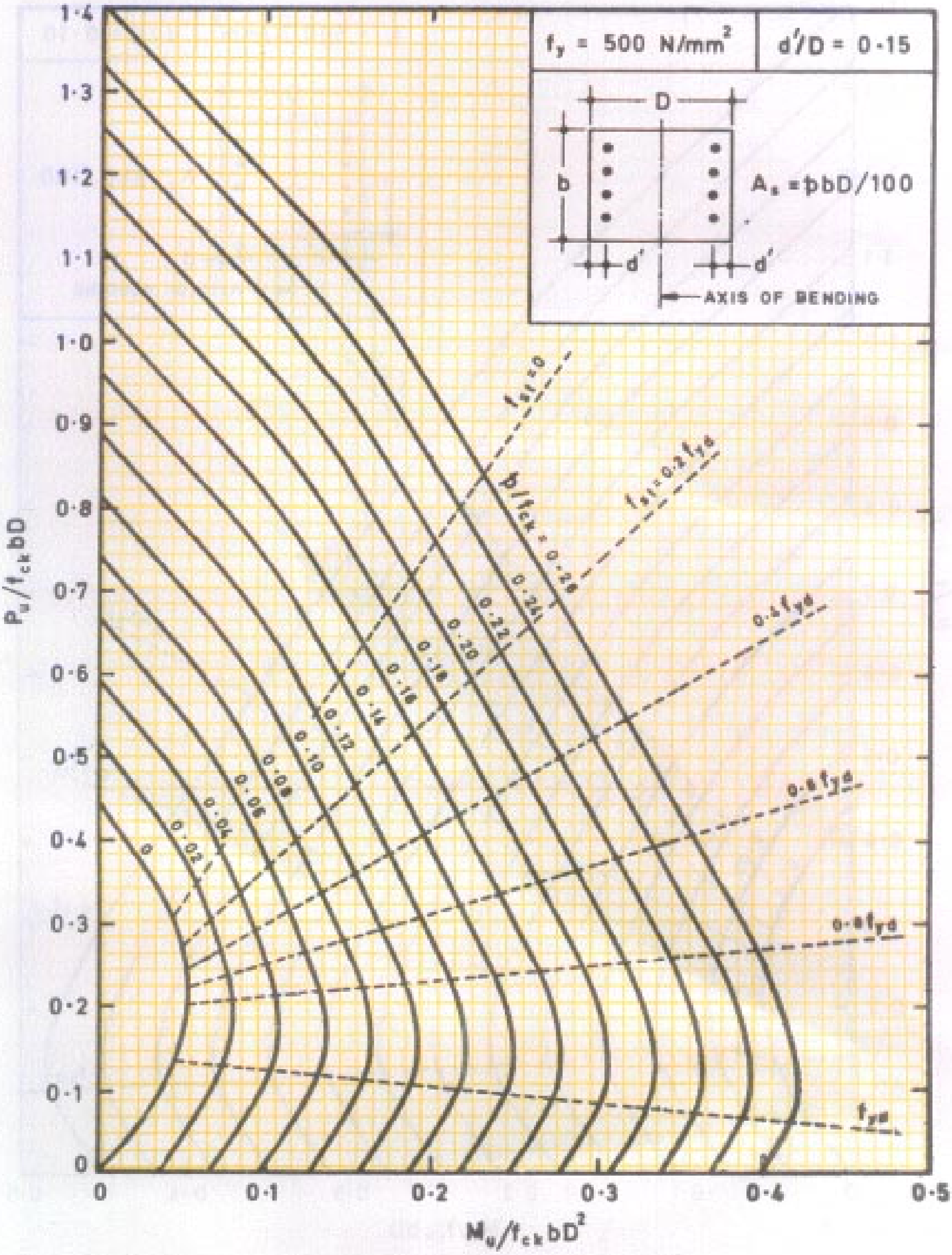


Chart 38 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

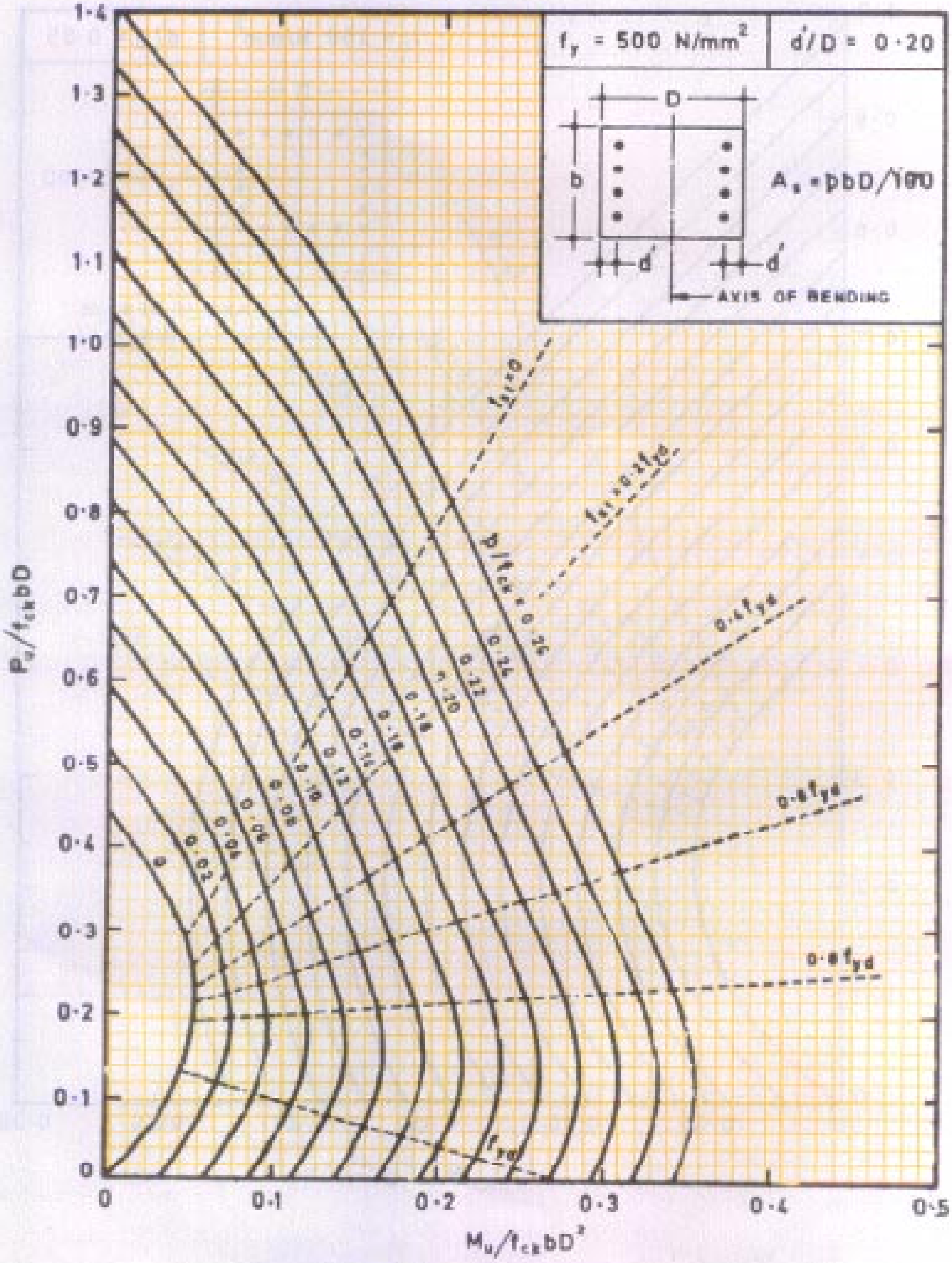


Chart 39 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

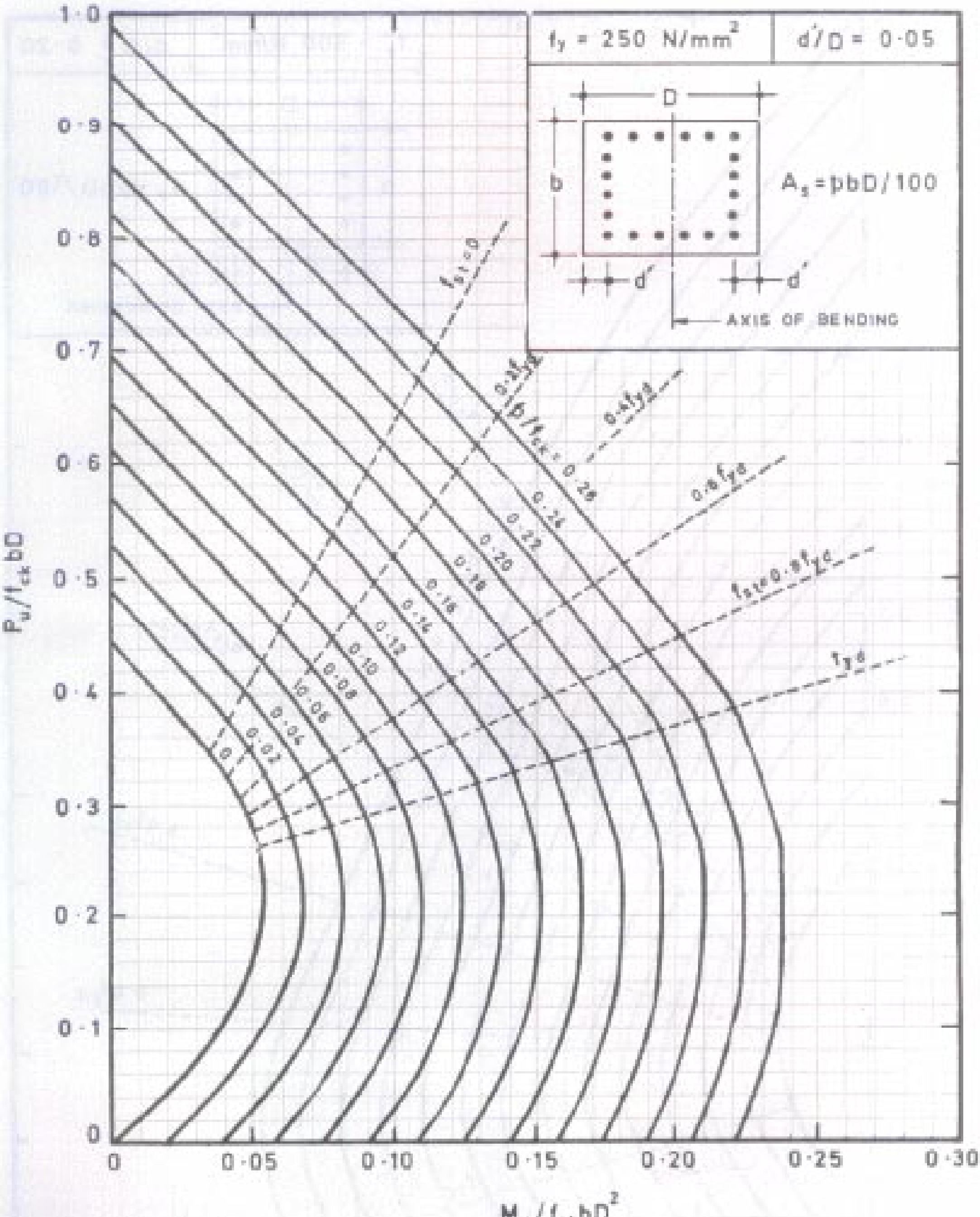


Chart 40 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

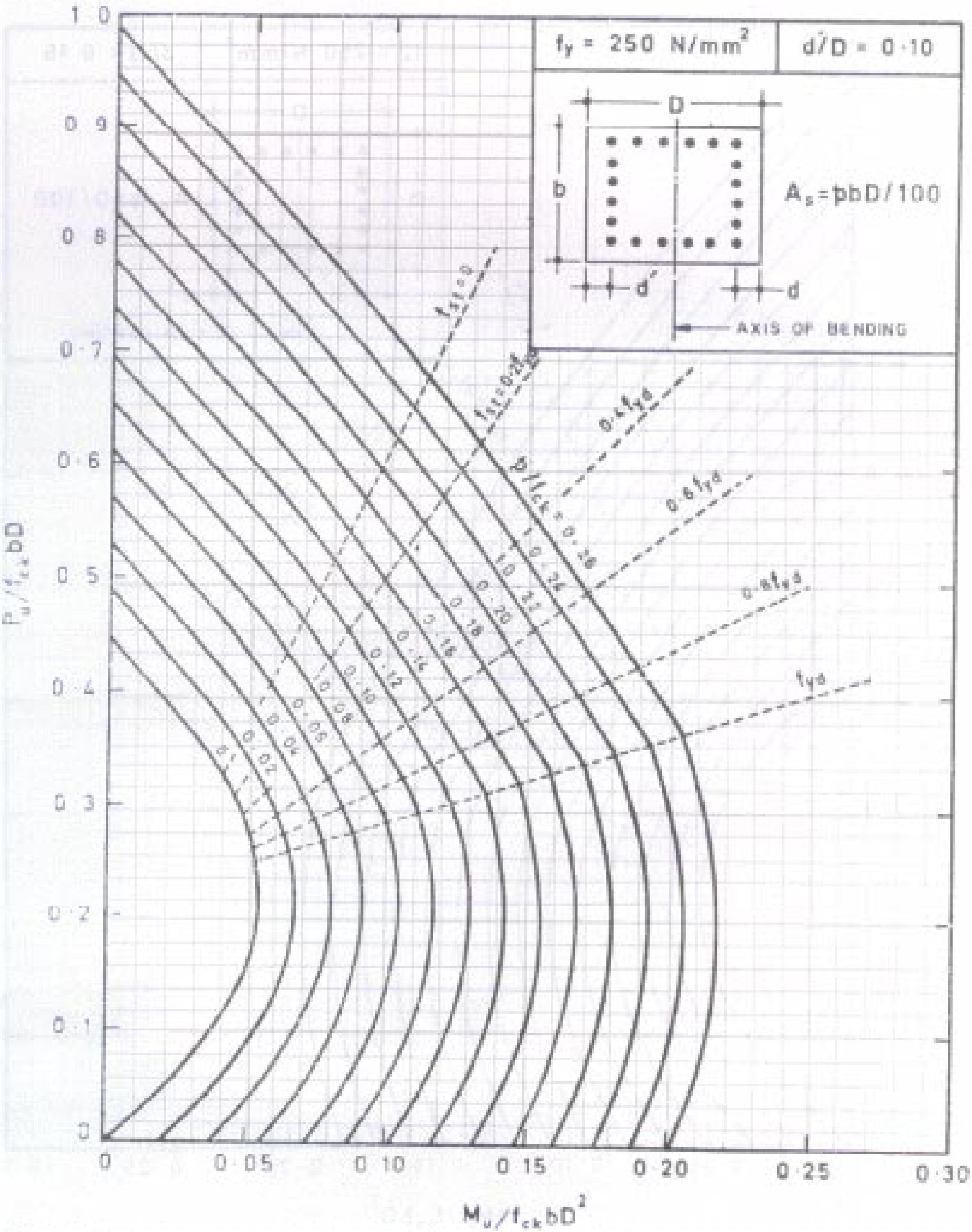


Chart 41 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

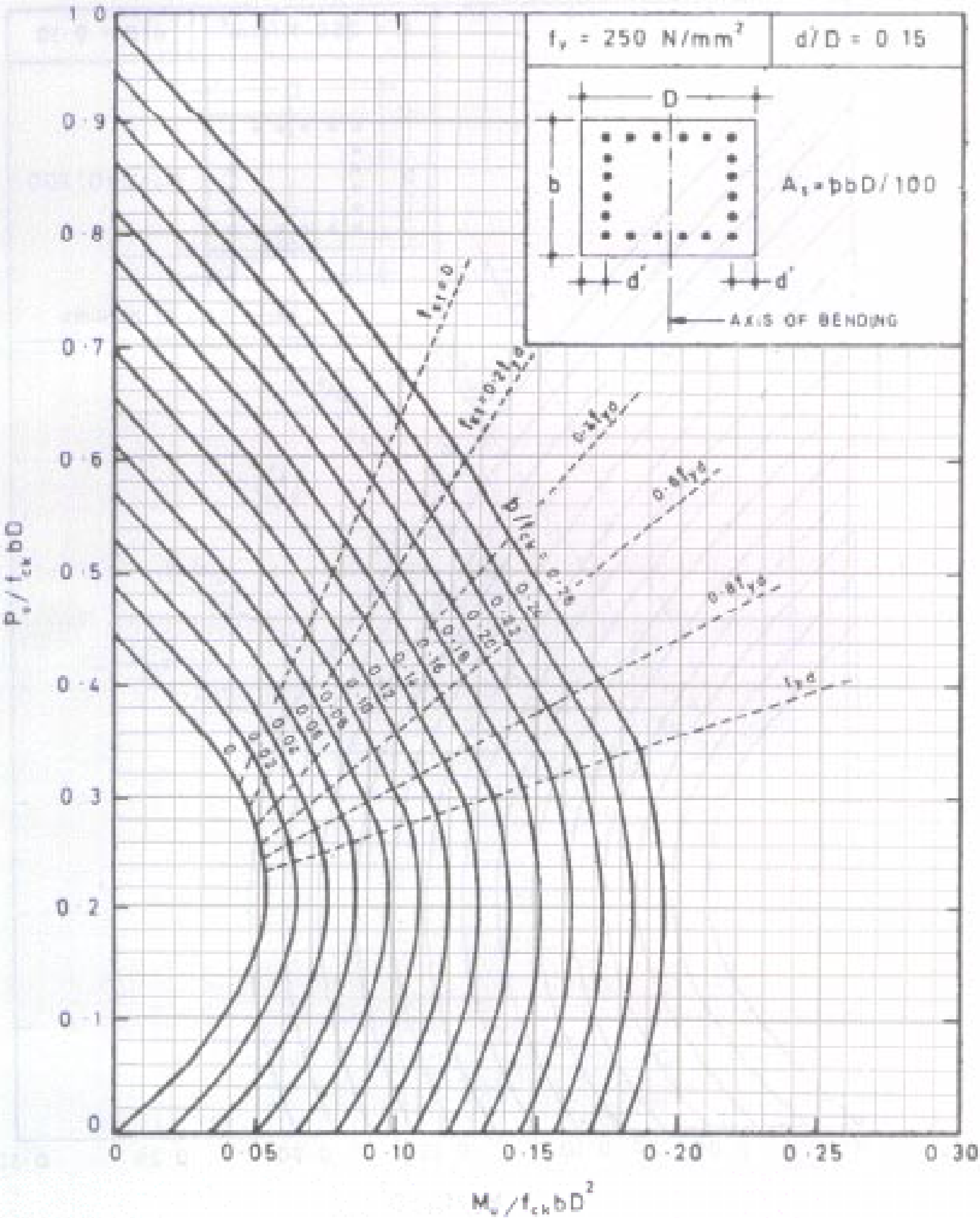
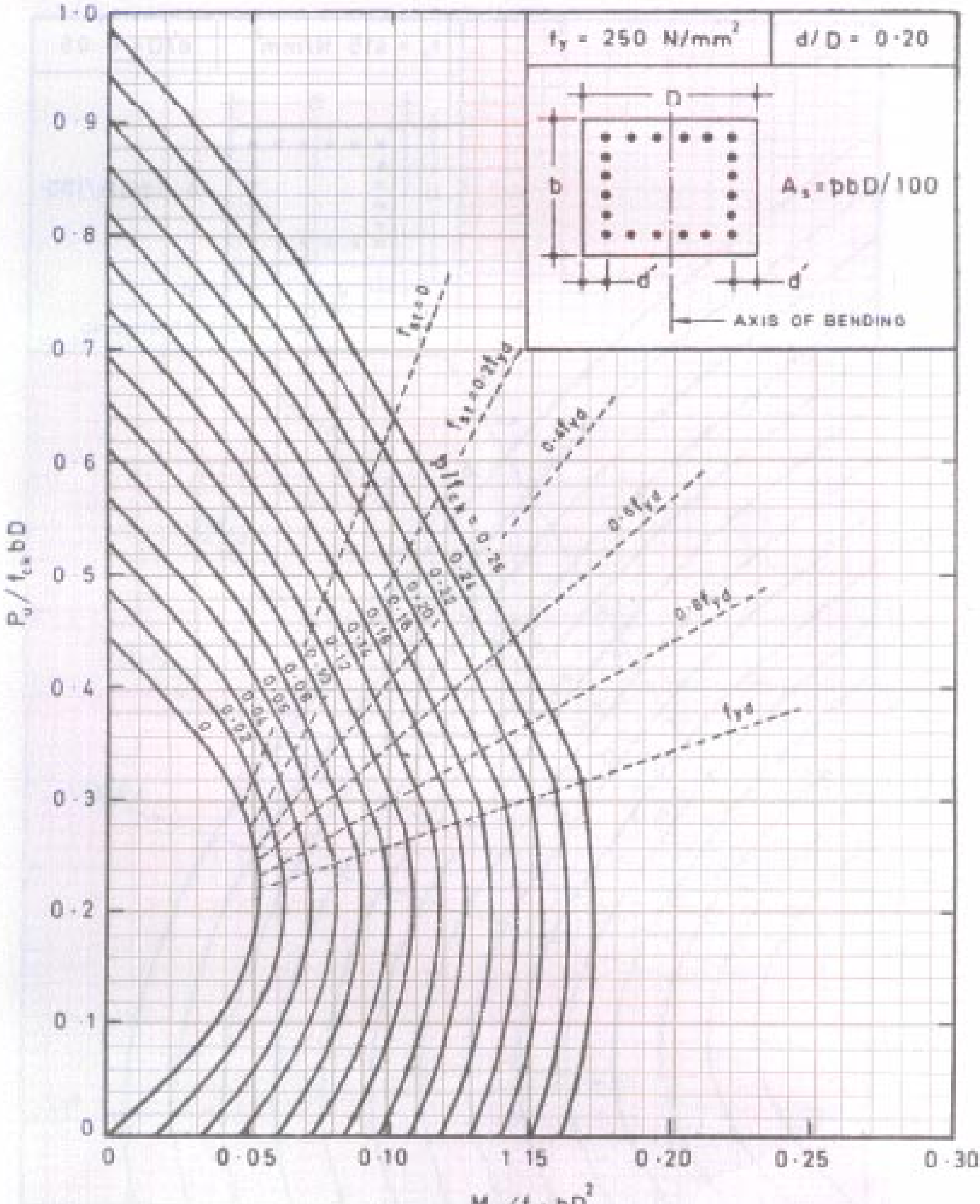


Chart 42 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides



f_y 415

Chart 43 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

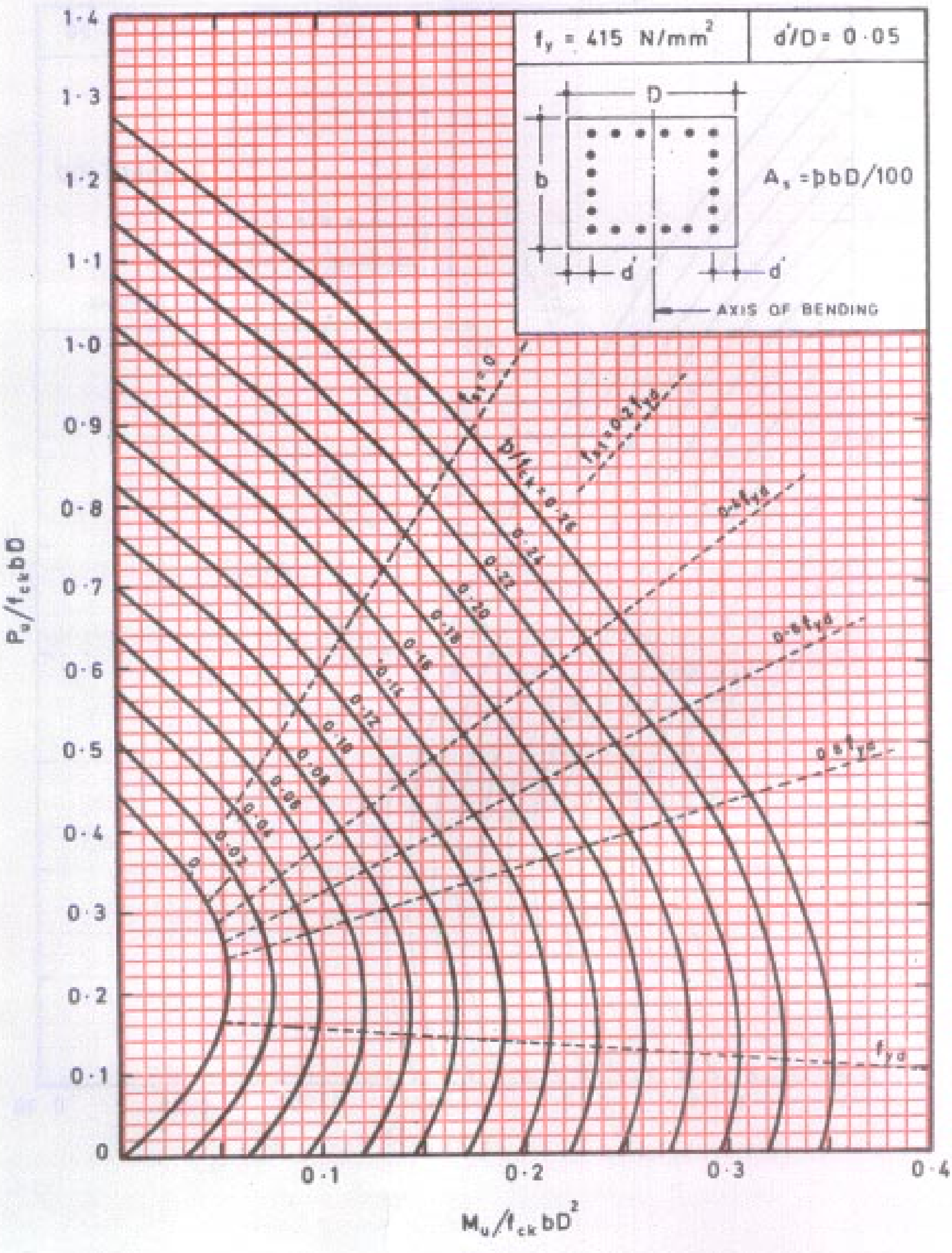


Chart 44 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

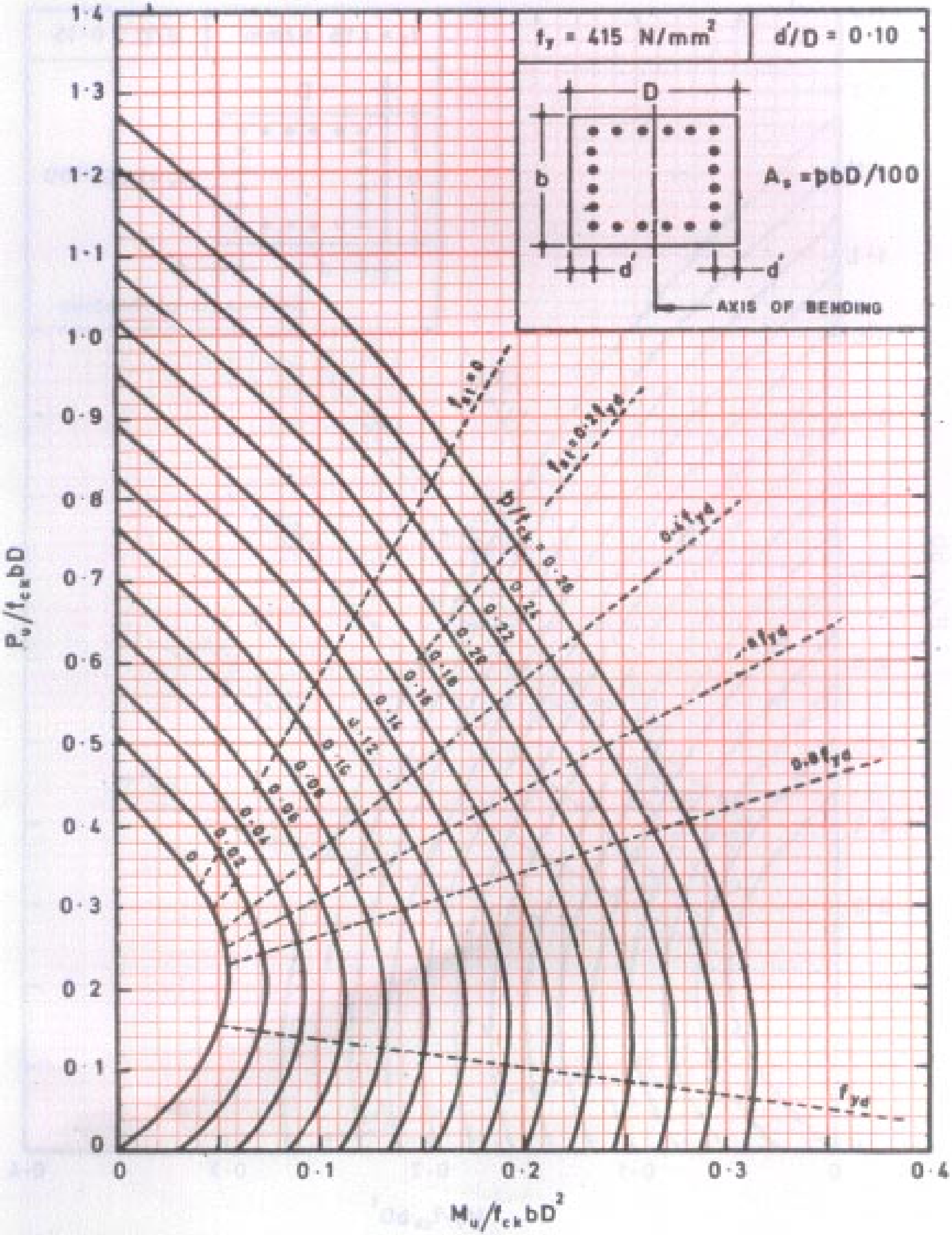


Chart 45 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

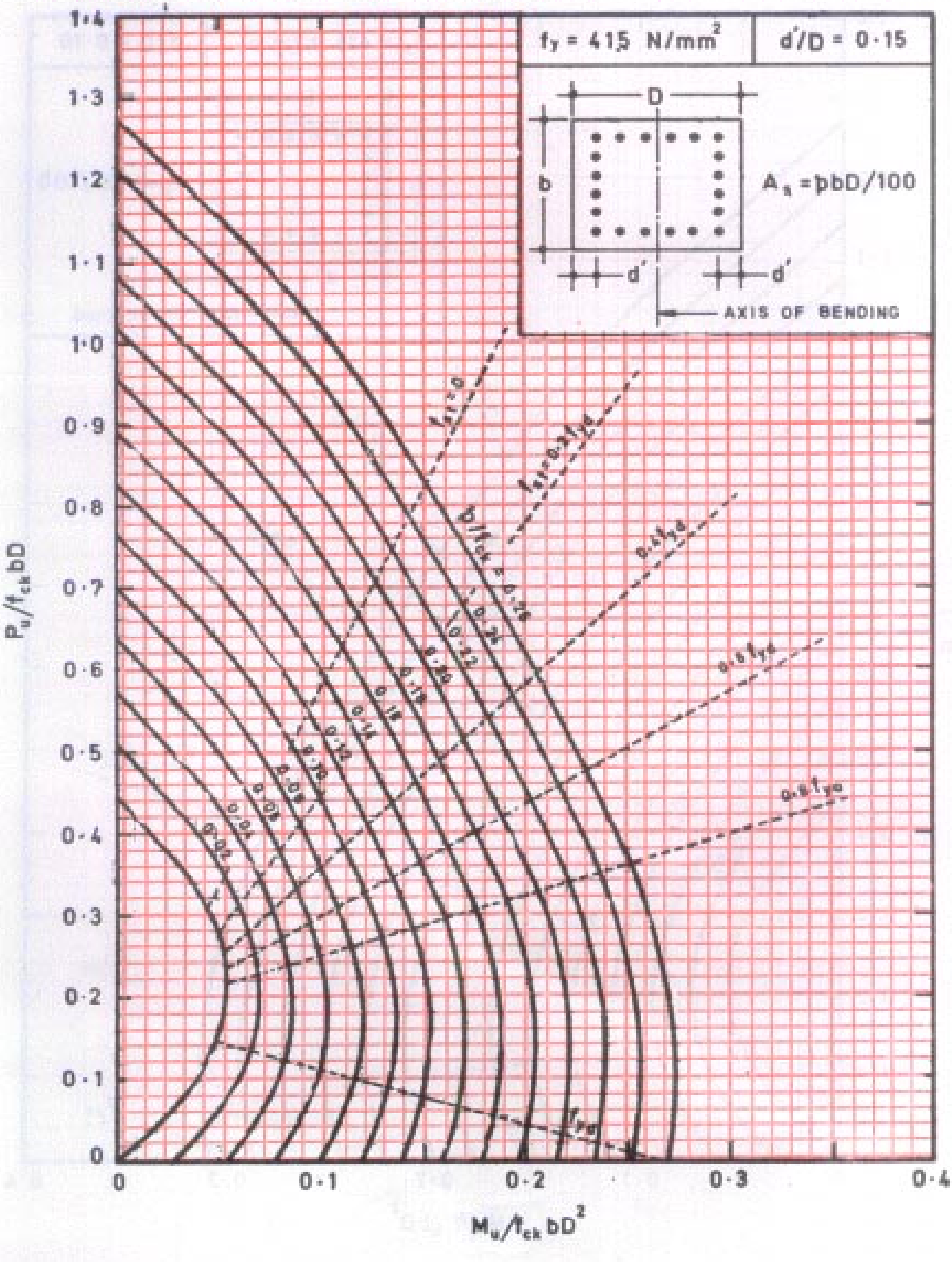


Chart 46 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

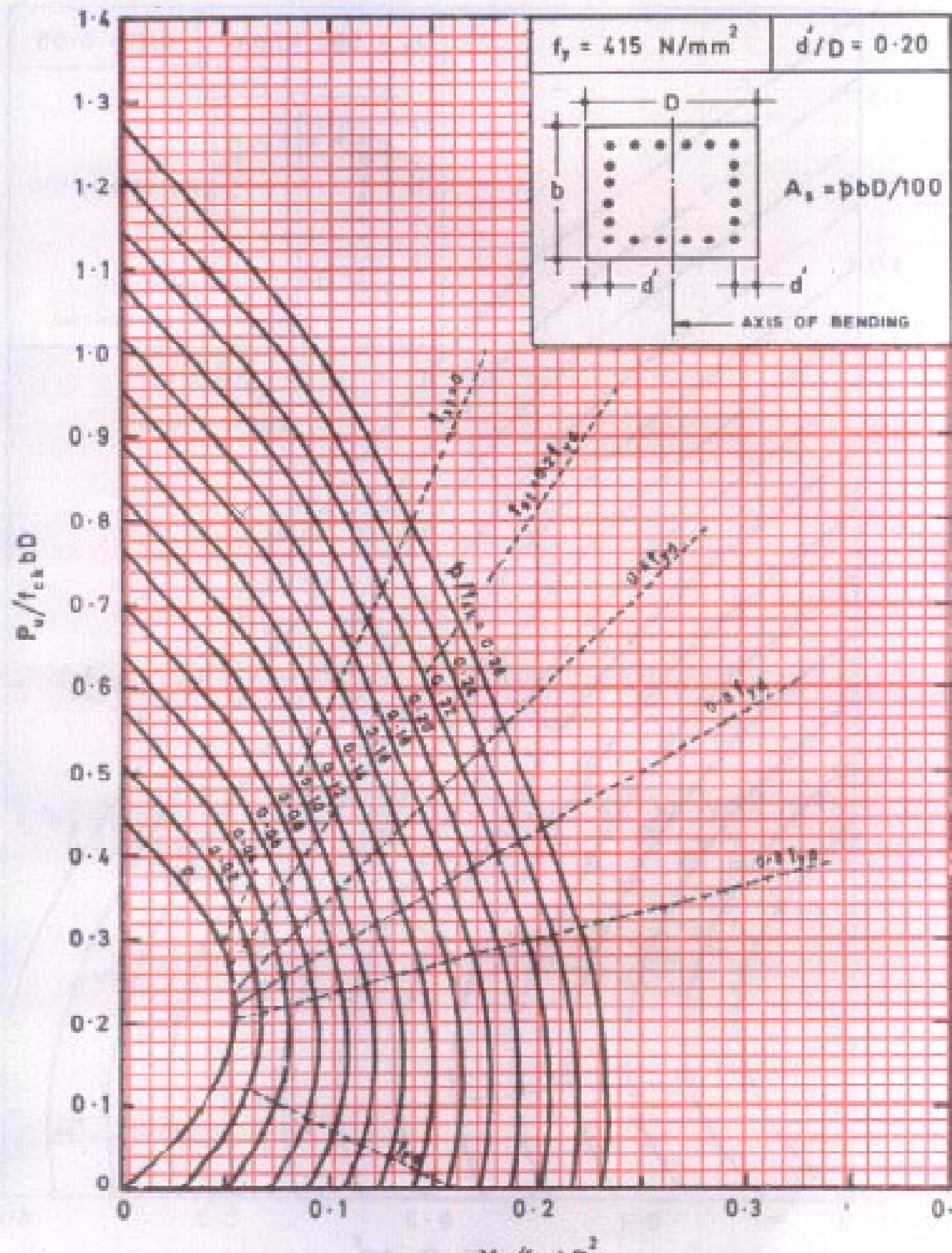


Chart 47 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

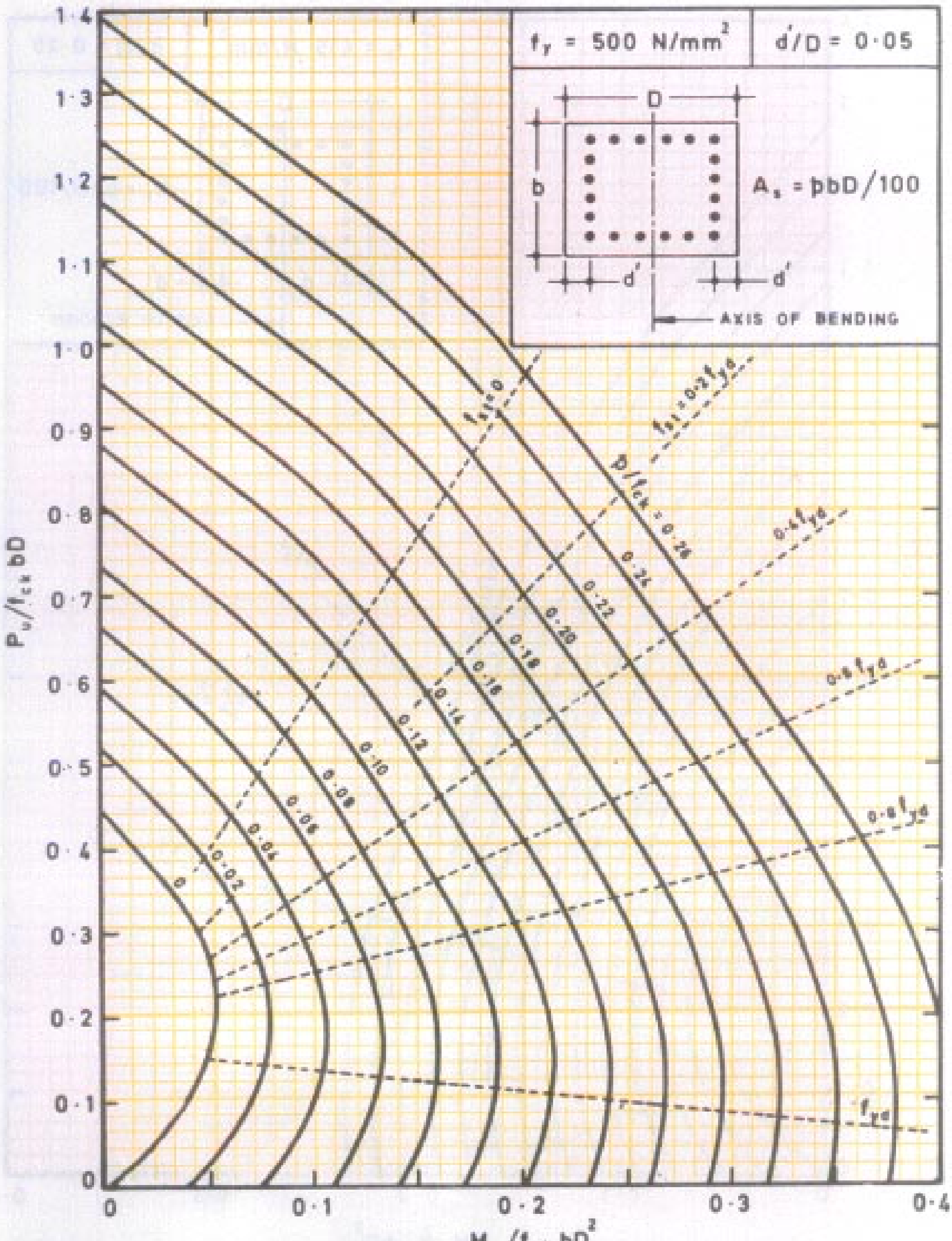


Chart 48 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

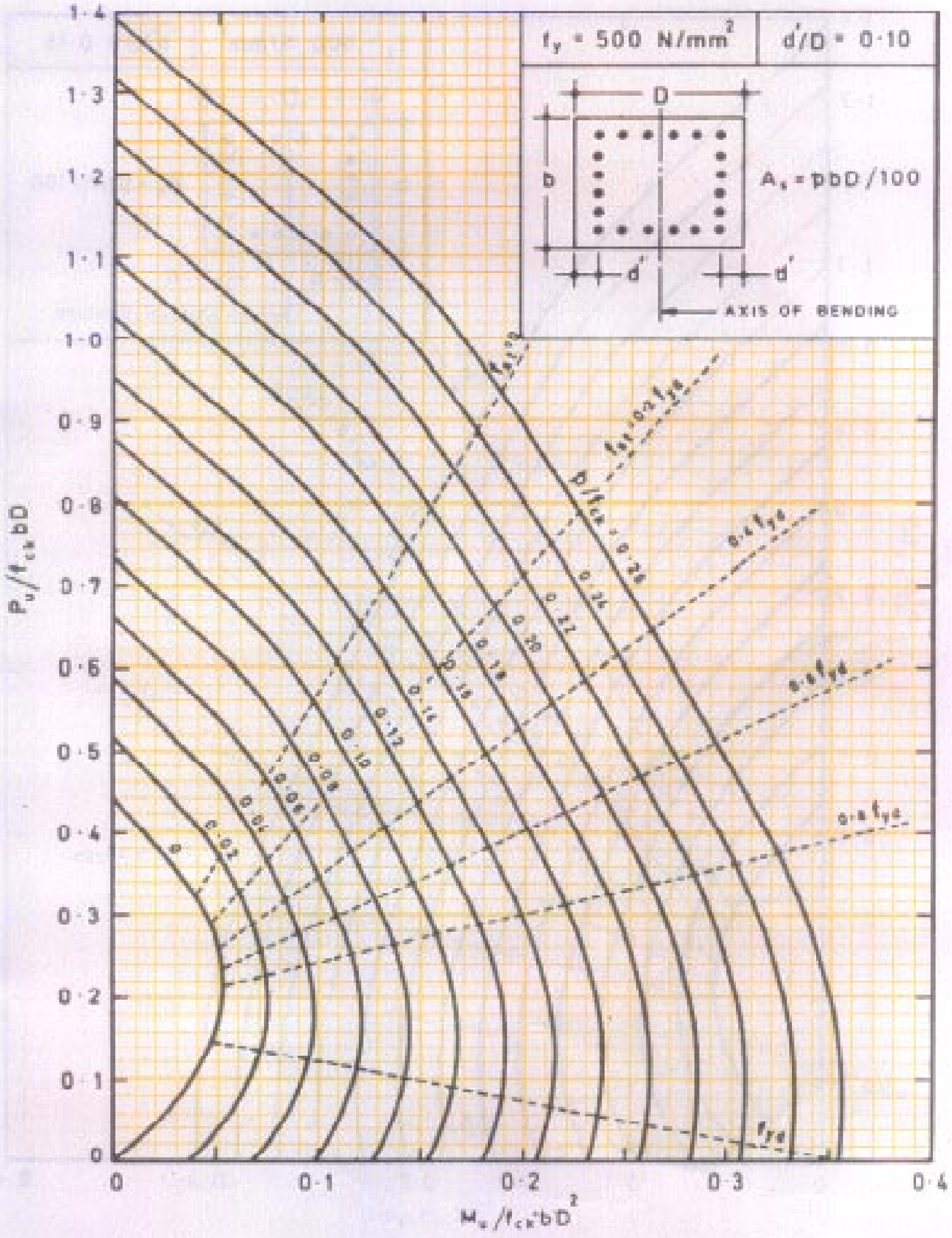


Chart 49 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

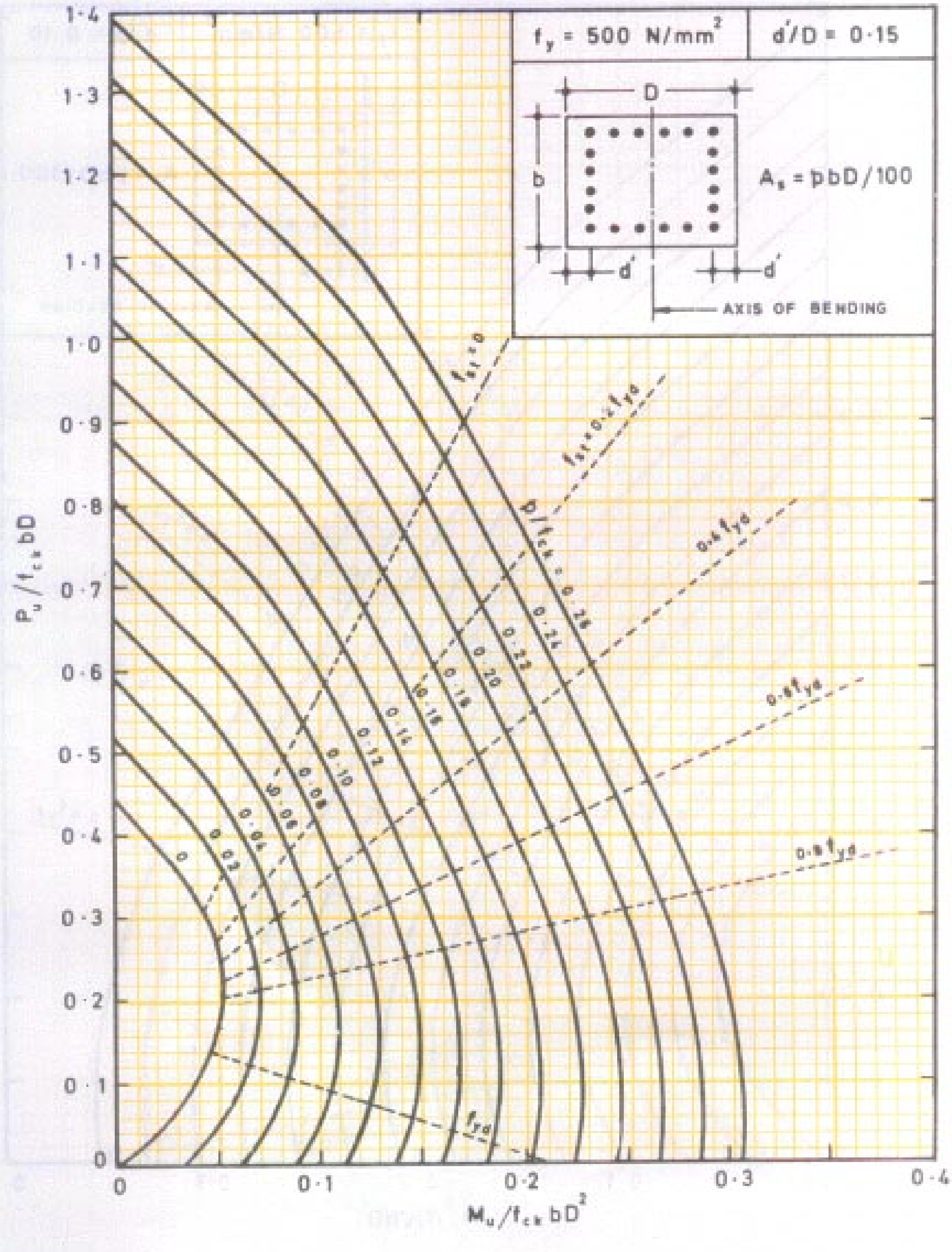


Chart 50 COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

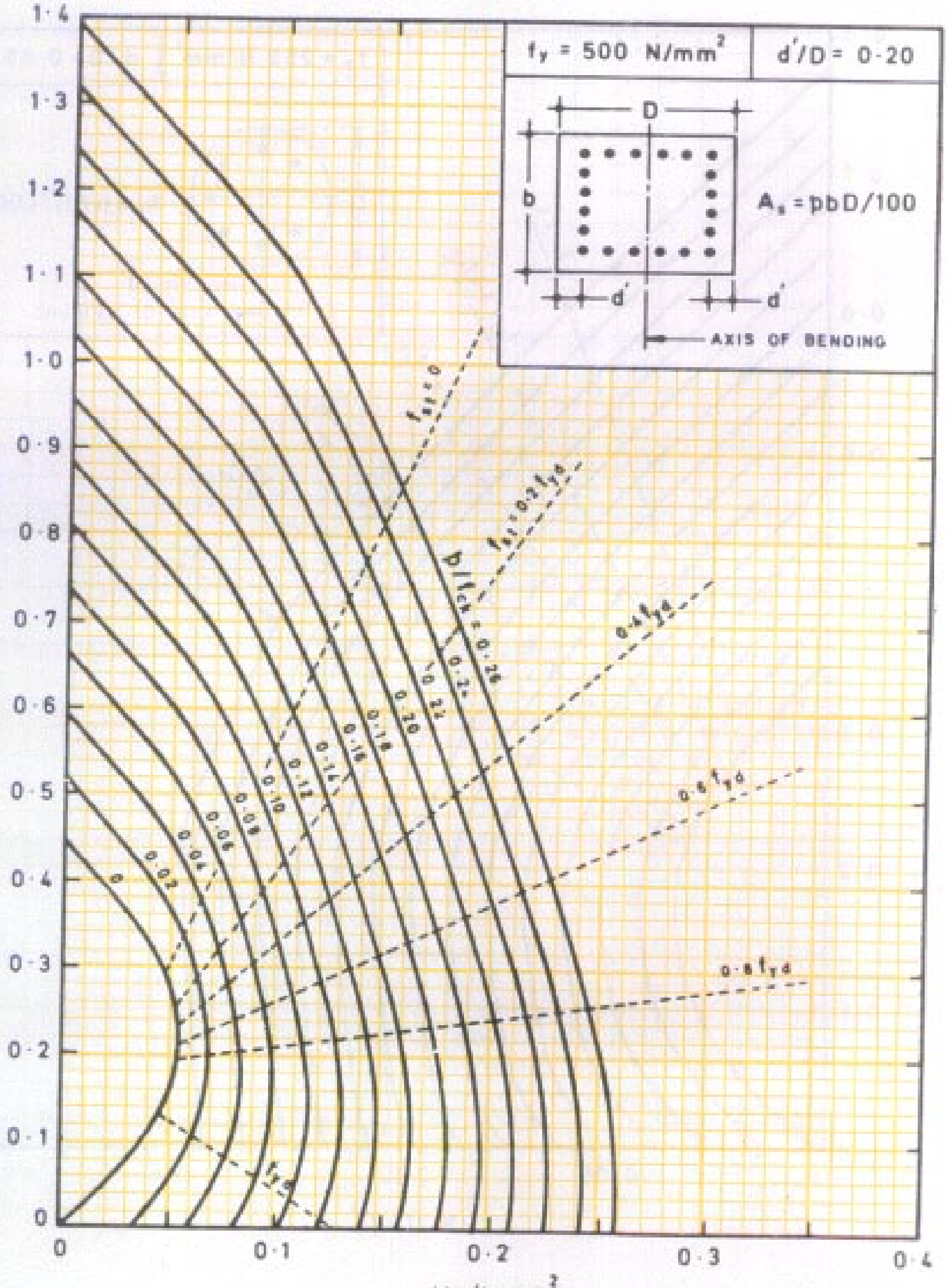


Chart 51 COMPRESSION WITH BENDING — Circular Section

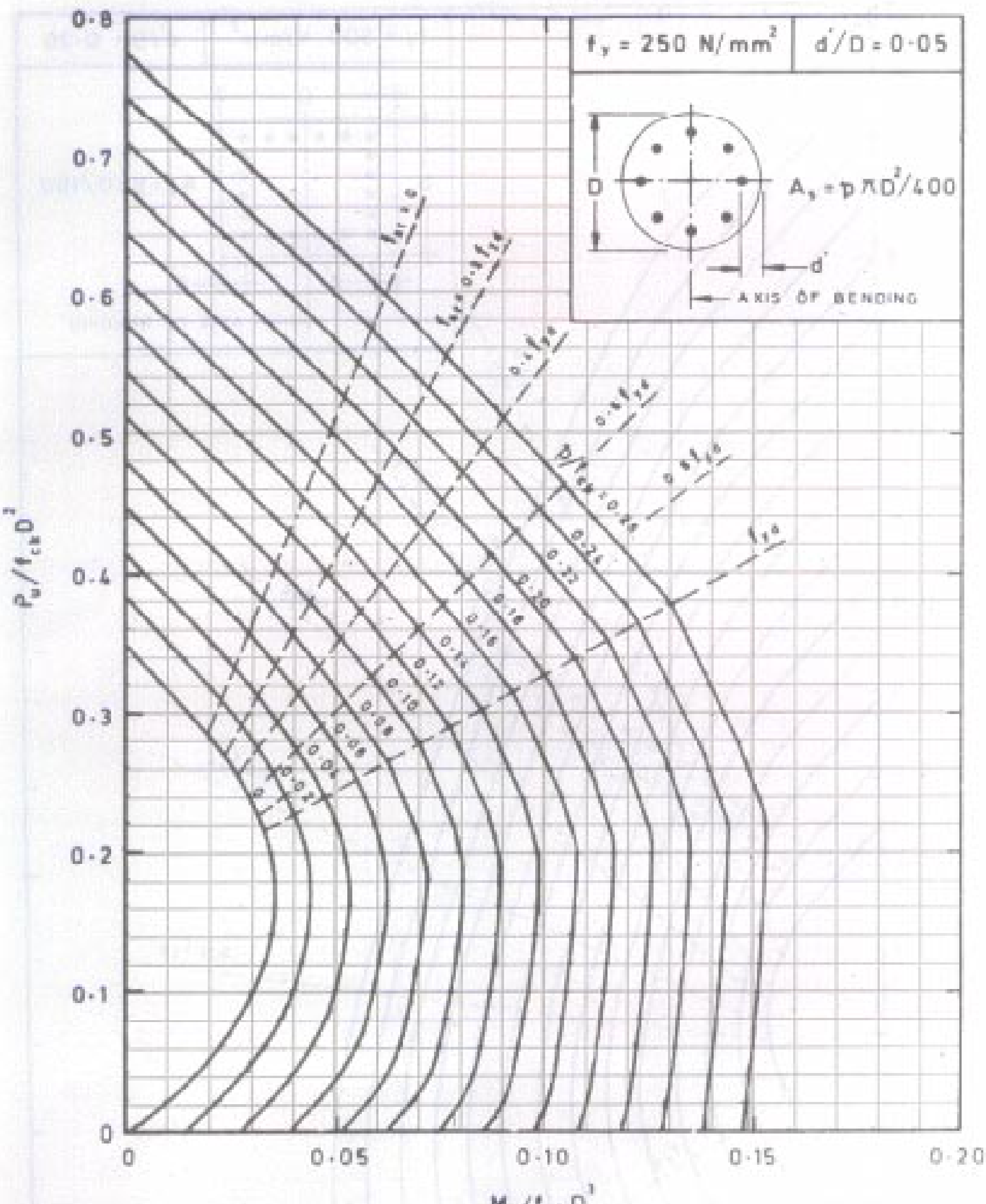


Chart 52 COMPRESSION WITH BENDING — Circular Section

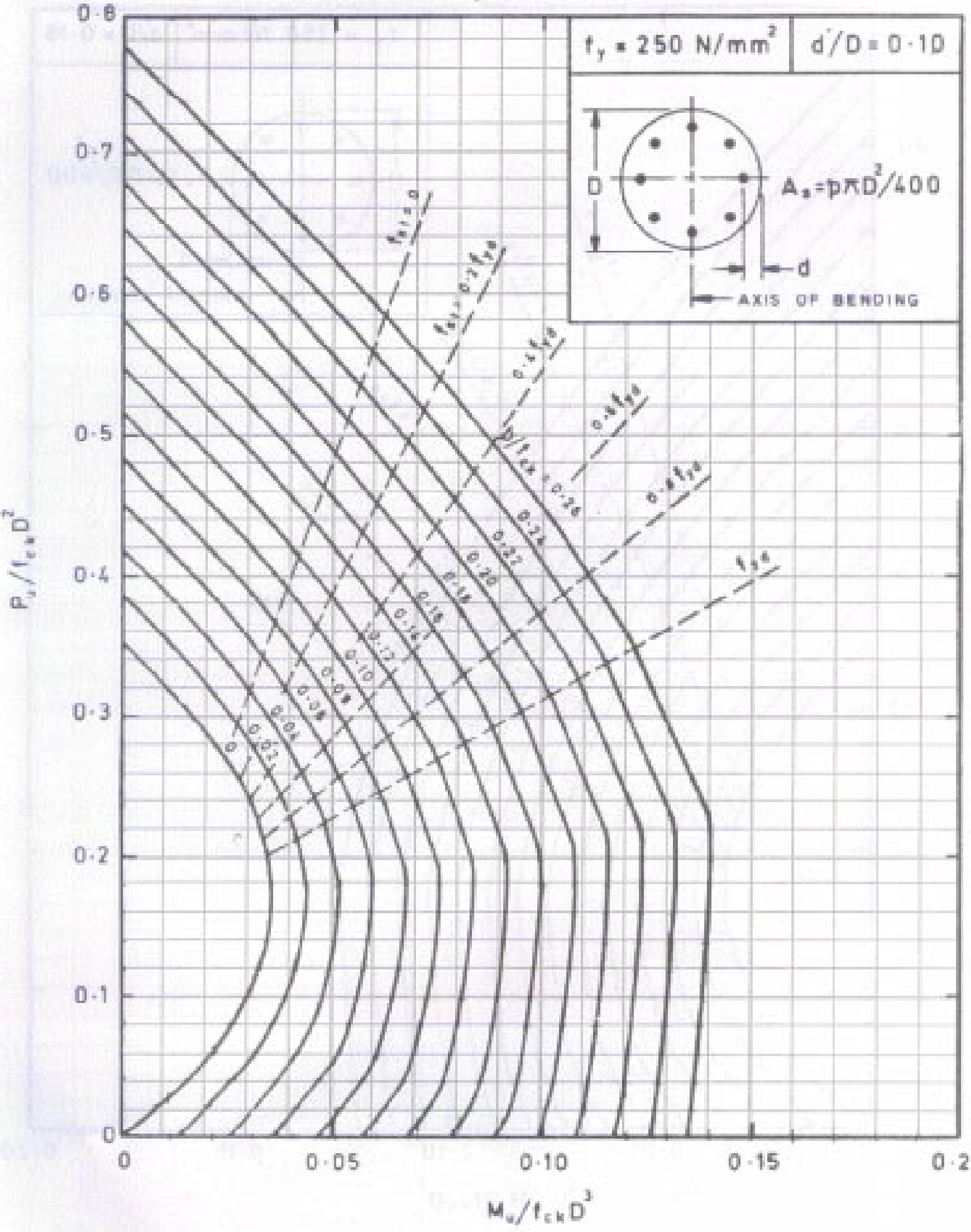


Chart 53 COMPRESSION WITH BENDING — Circular Section

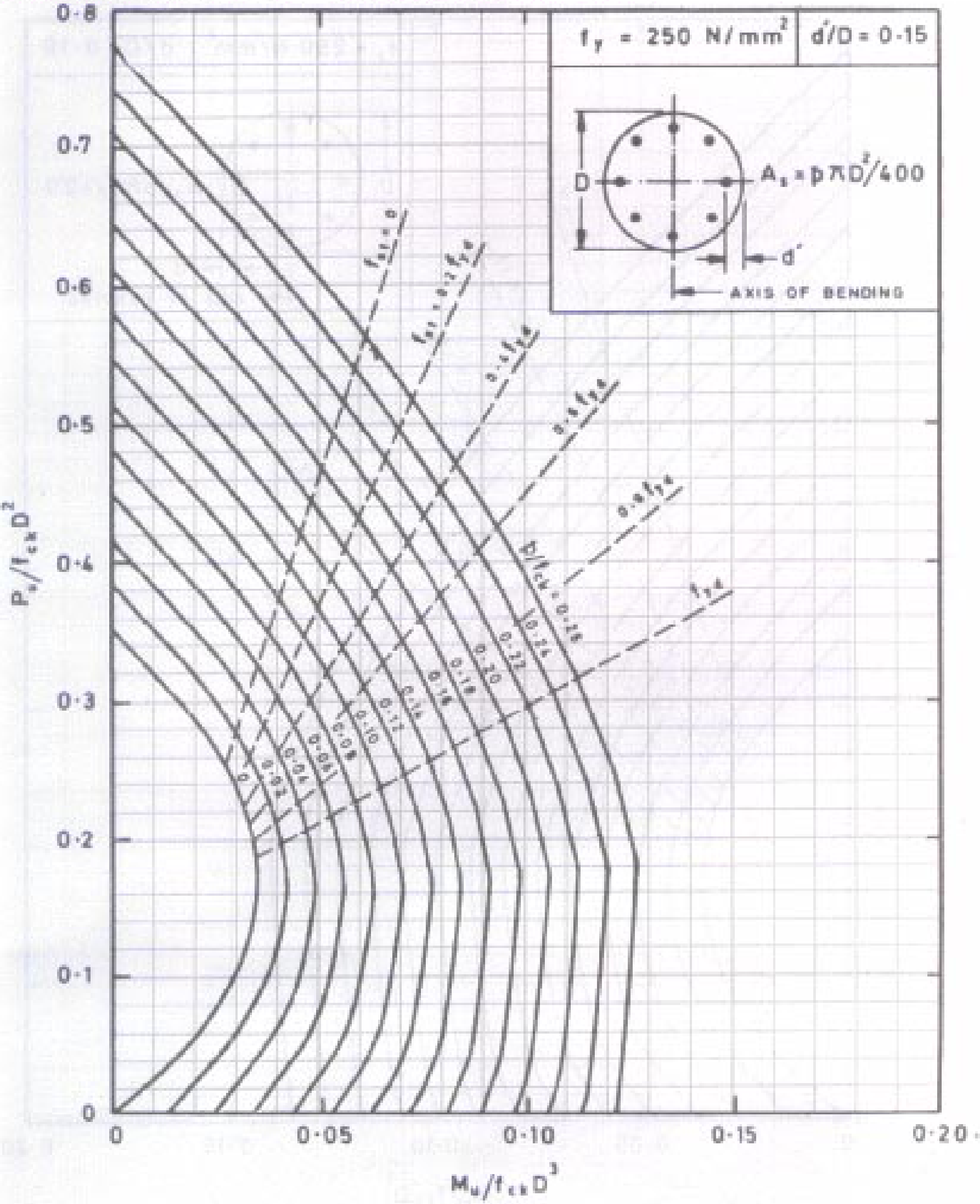


Chart 54 COMPRESSION WITH BENDING — Circular Section

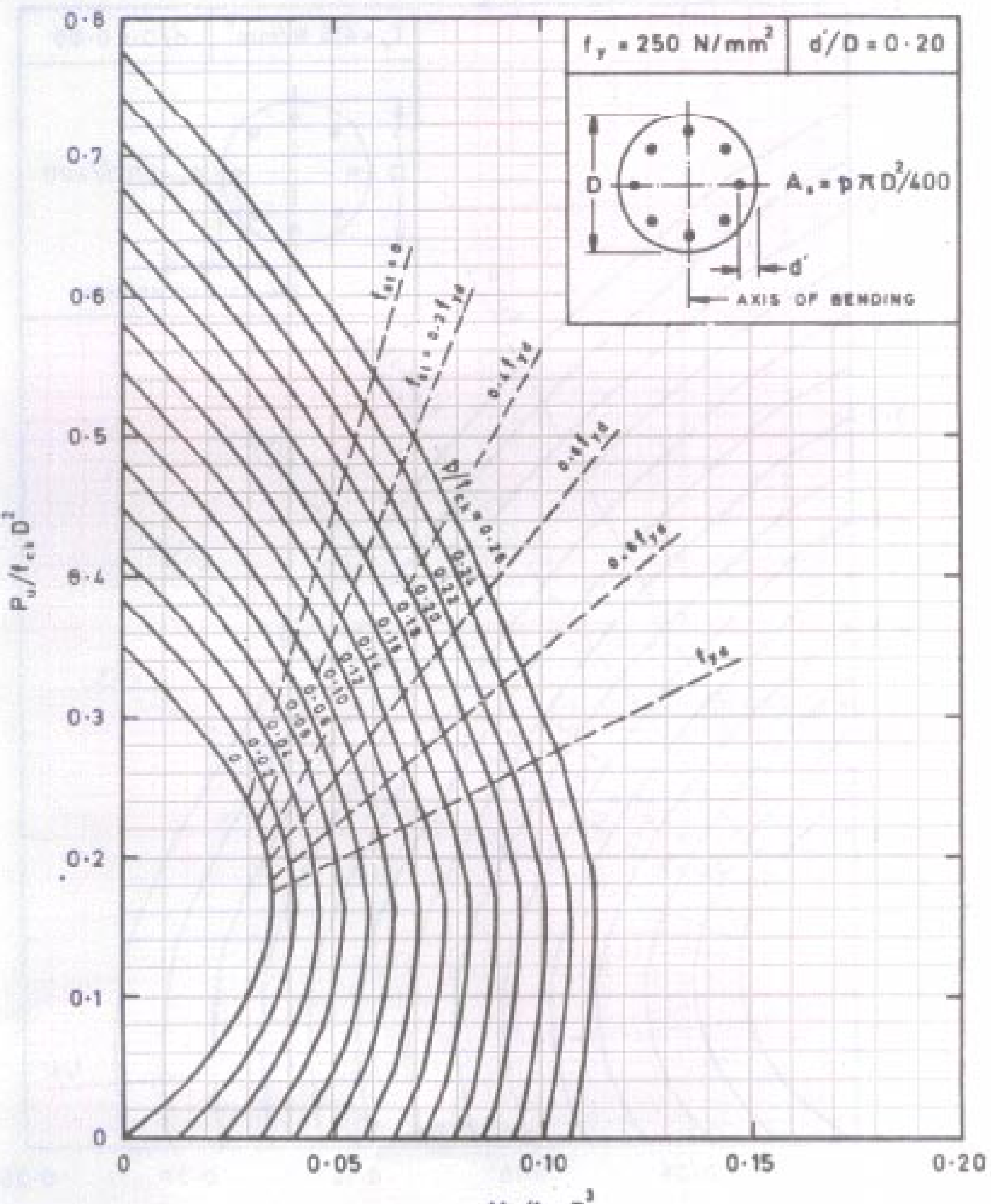


Chart 55 COMPRESSION WITH BENDING — Circular Section

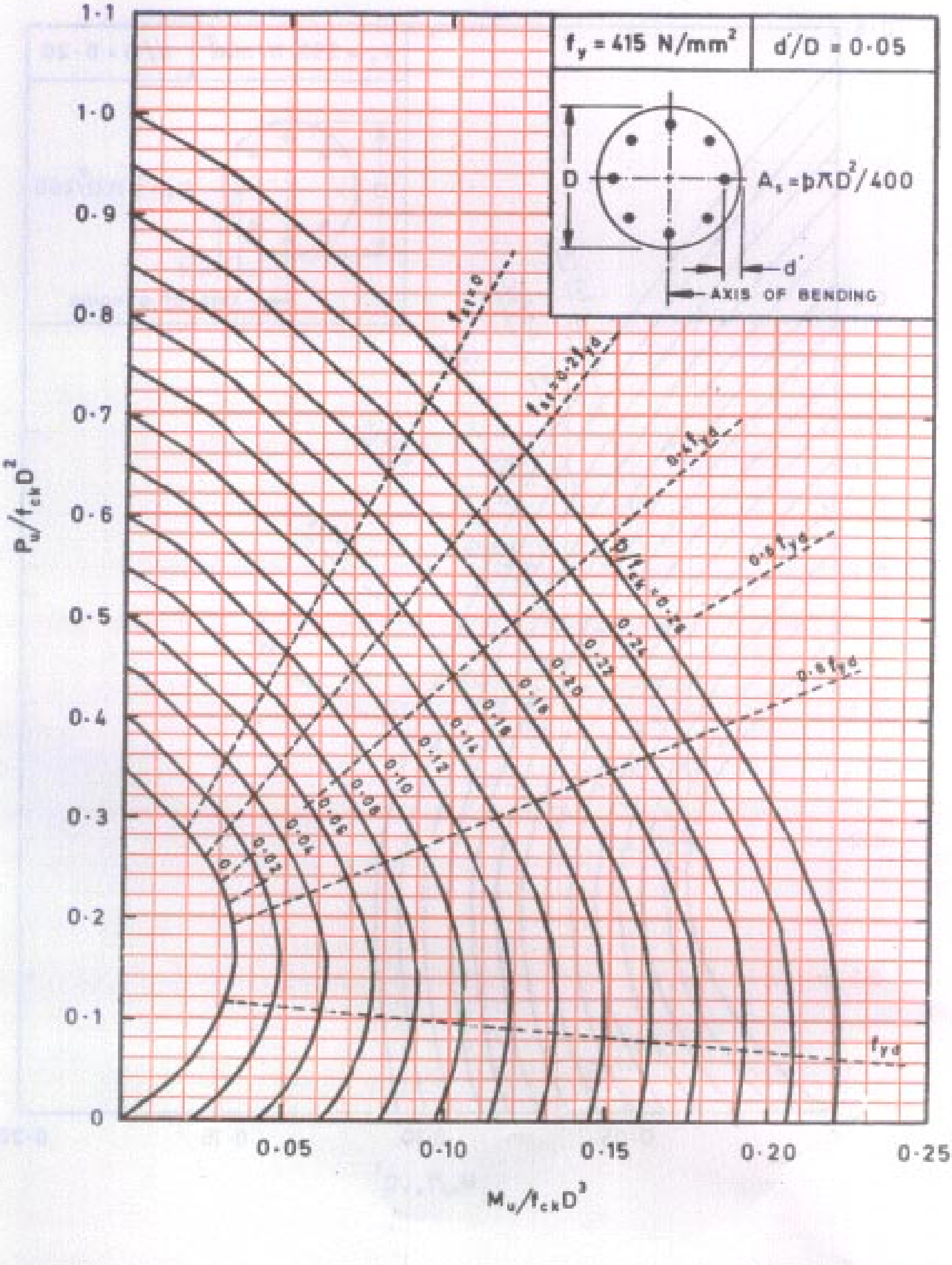


Chart 56 COMPRESSION WITH BENDING — Circular Section

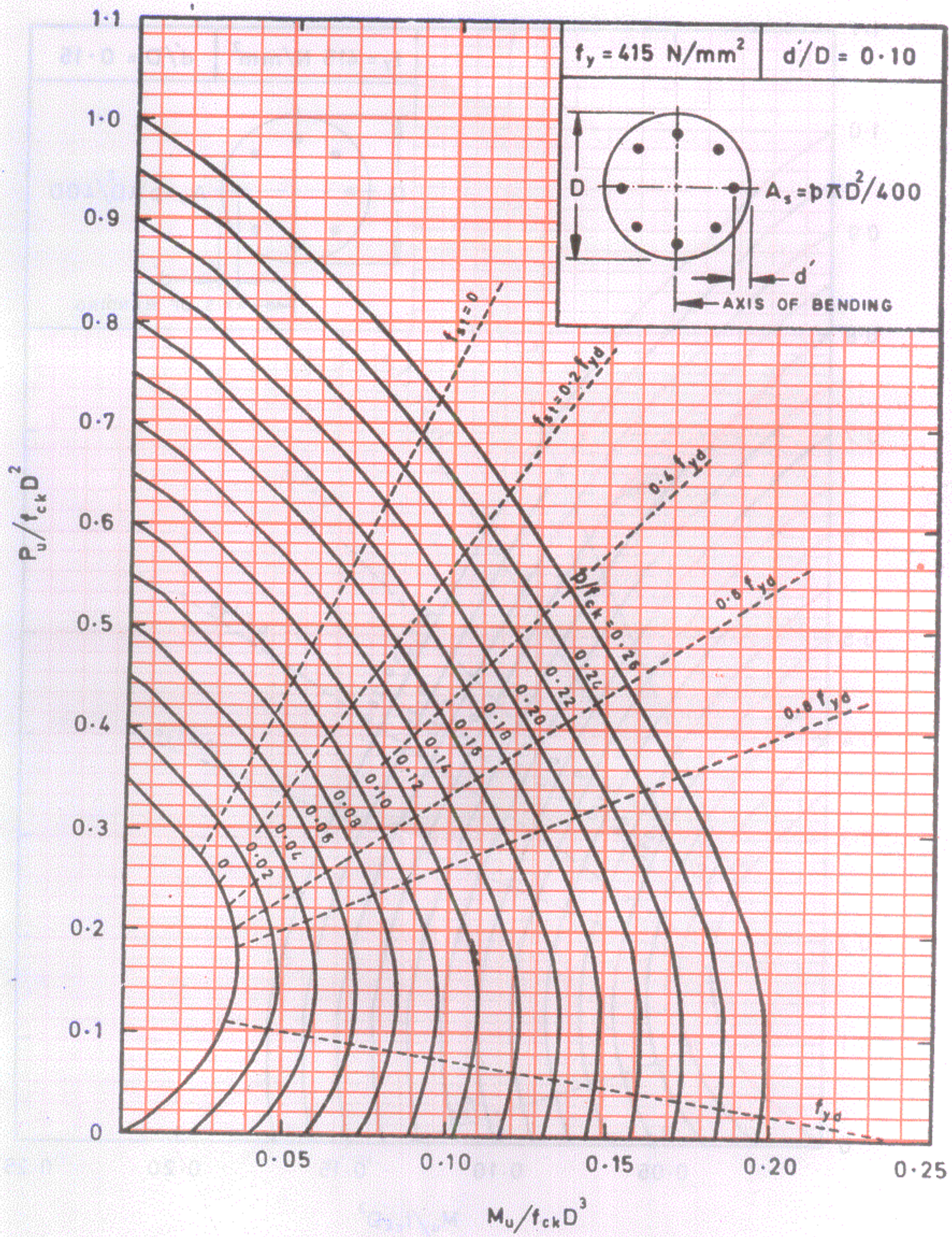


Chart 57 COMPRESSION WITH BENDING — Circular Section

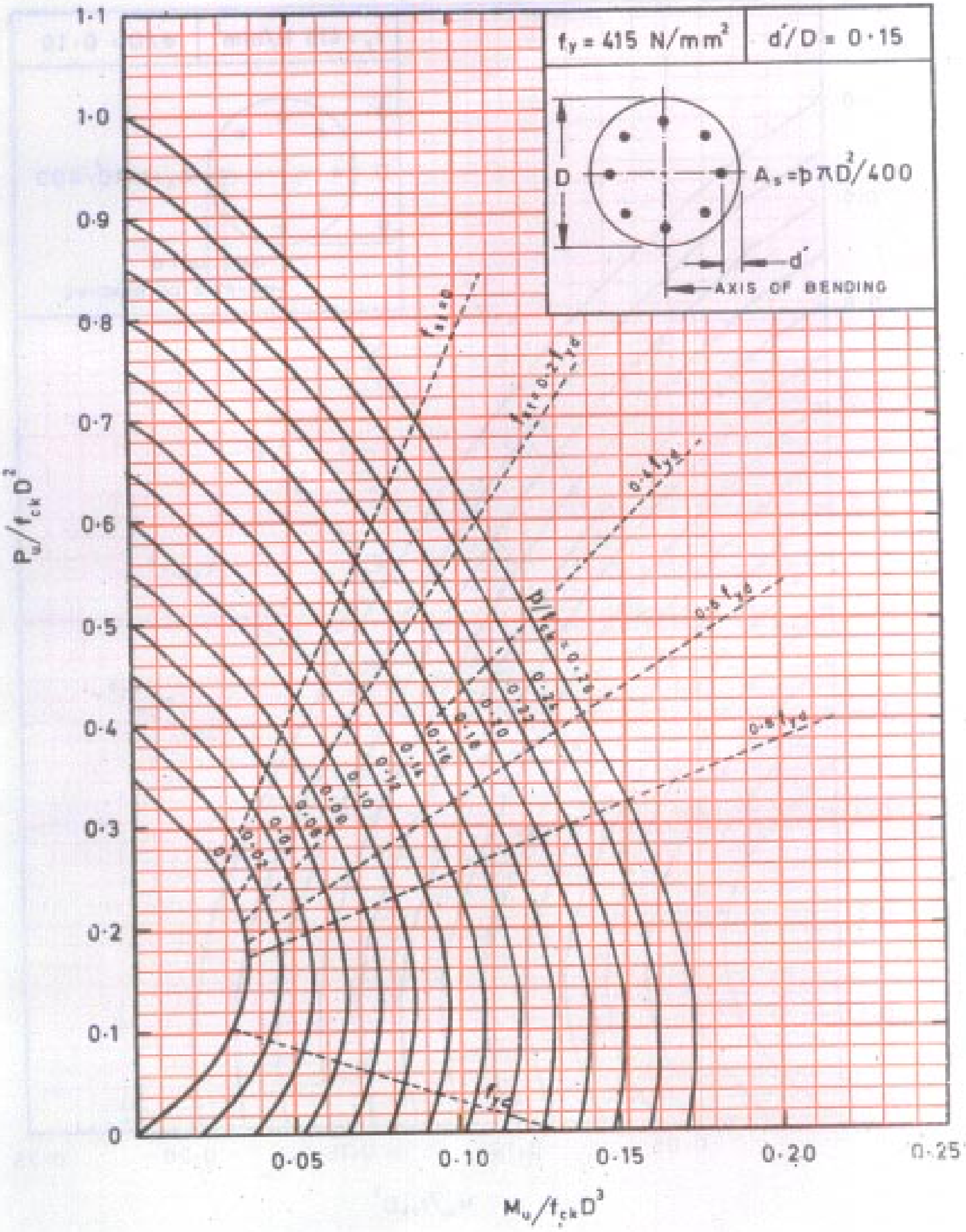


Chart 58 COMPRESSION WITH BENDING — Circular Section

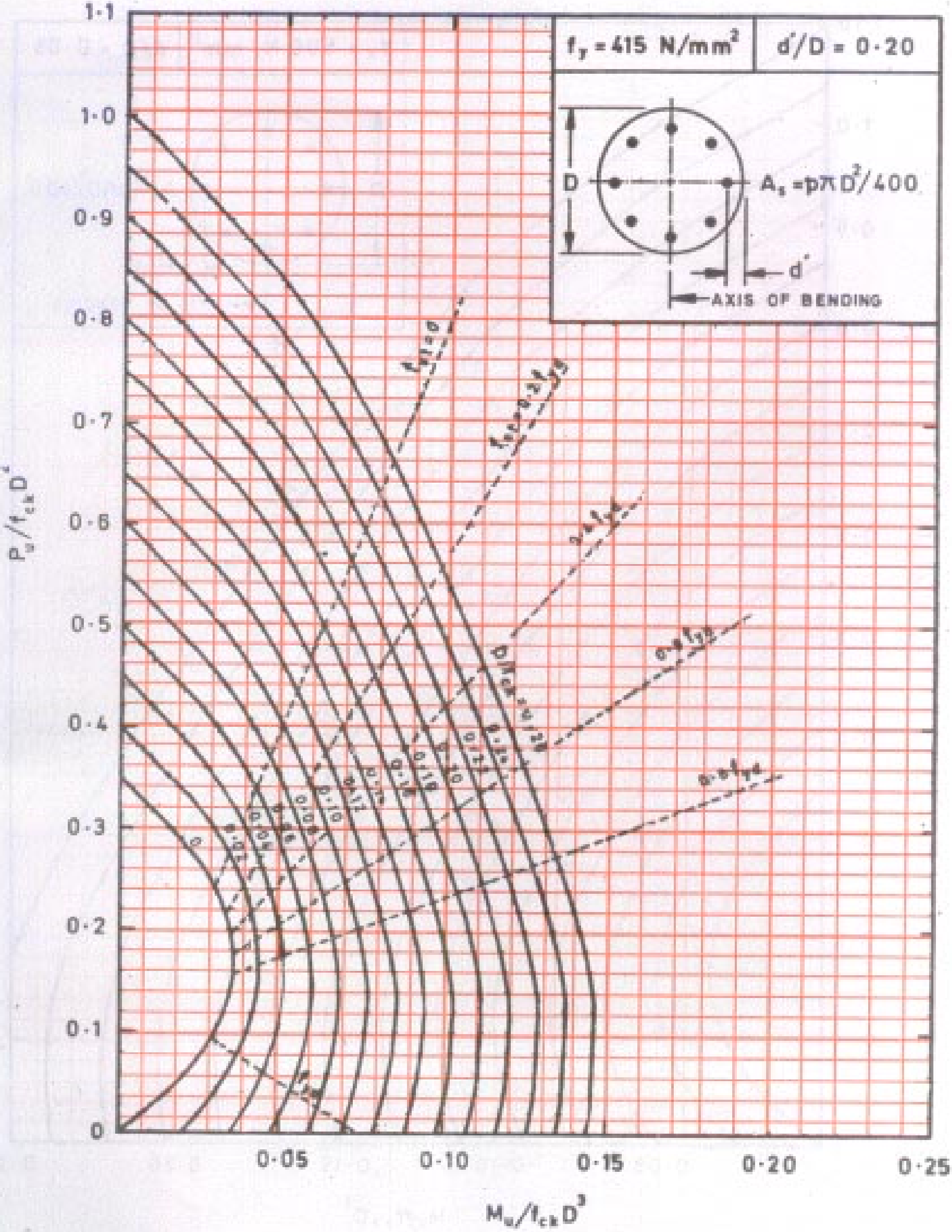


Chart 59 COMPRESSION WITH BENDING — Circular Section

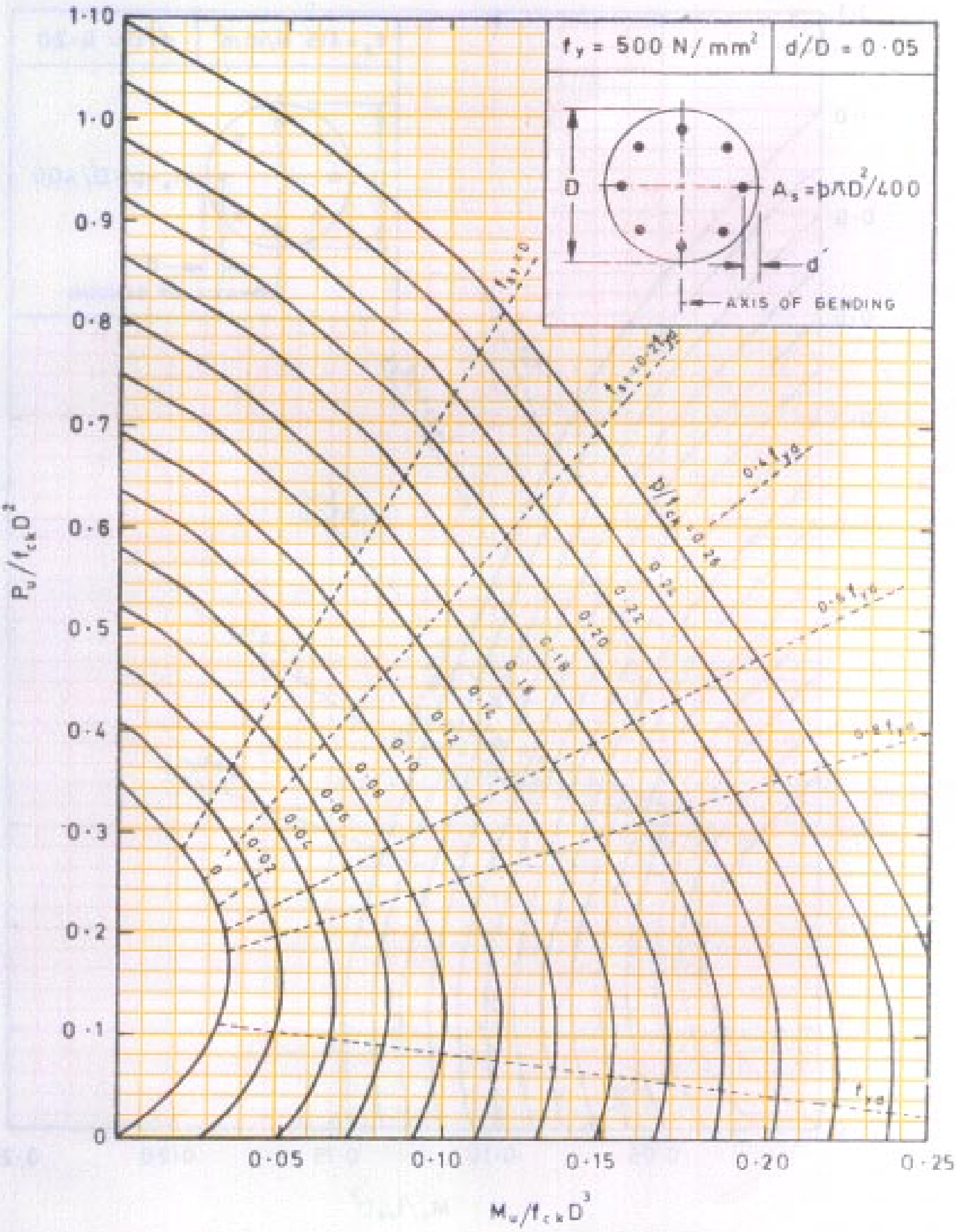


Chart 60 COMPRESSION WITH BENDING — Circular Section

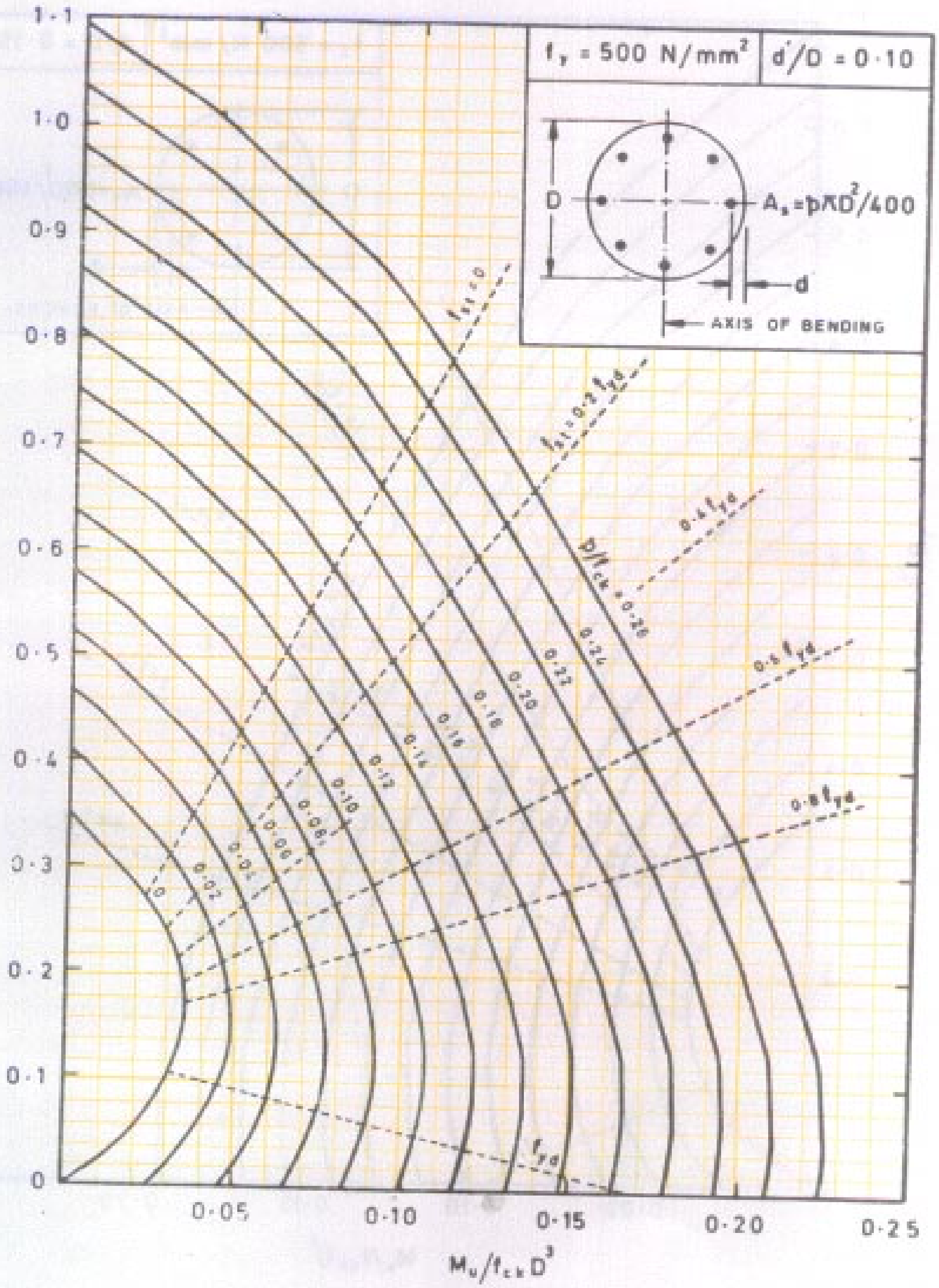


Chart 61 **COMPRESSION WITH BENDING — Circular Section**

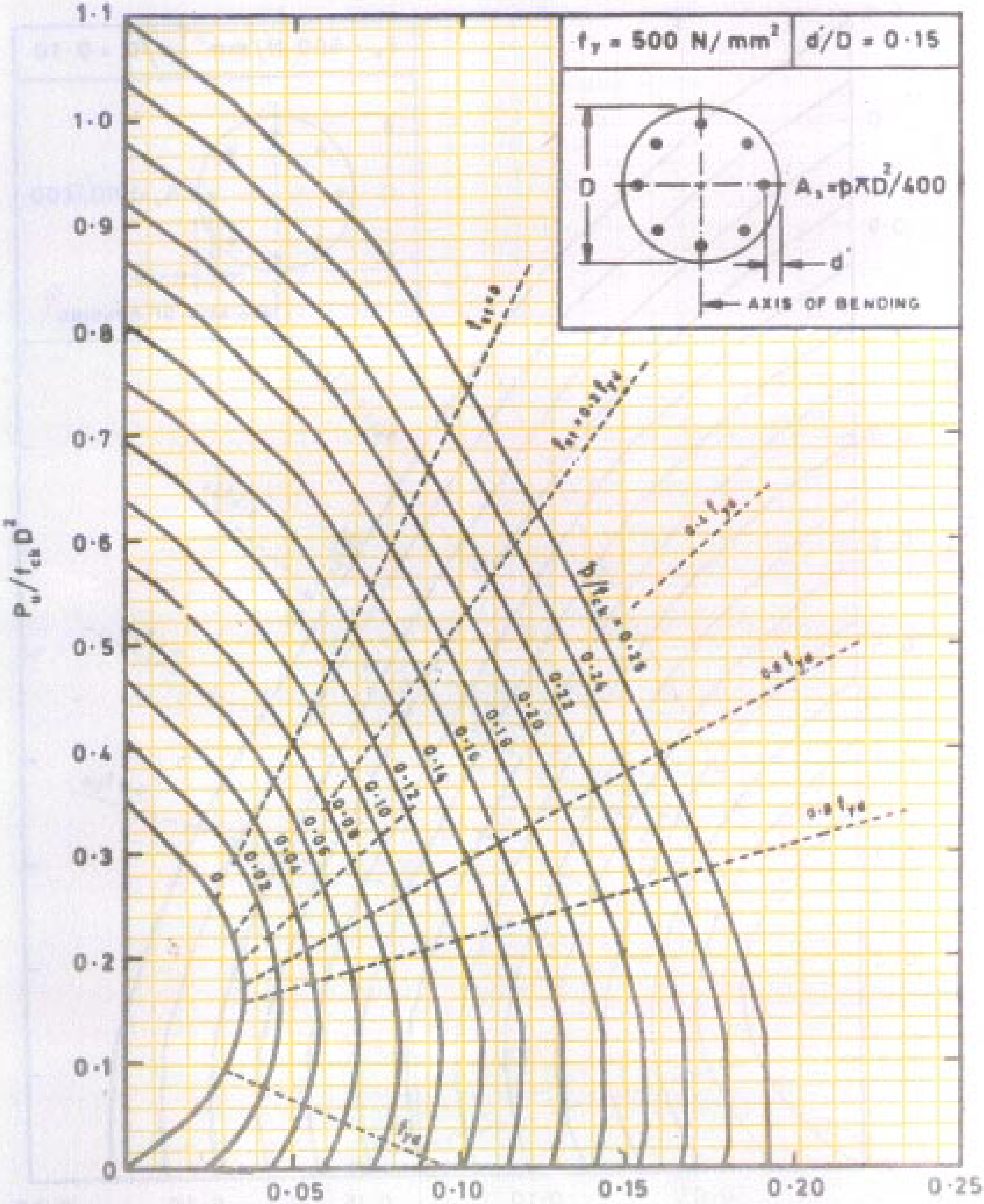


Chart 62 COMPRESSION WITH BENDING — Circular Section

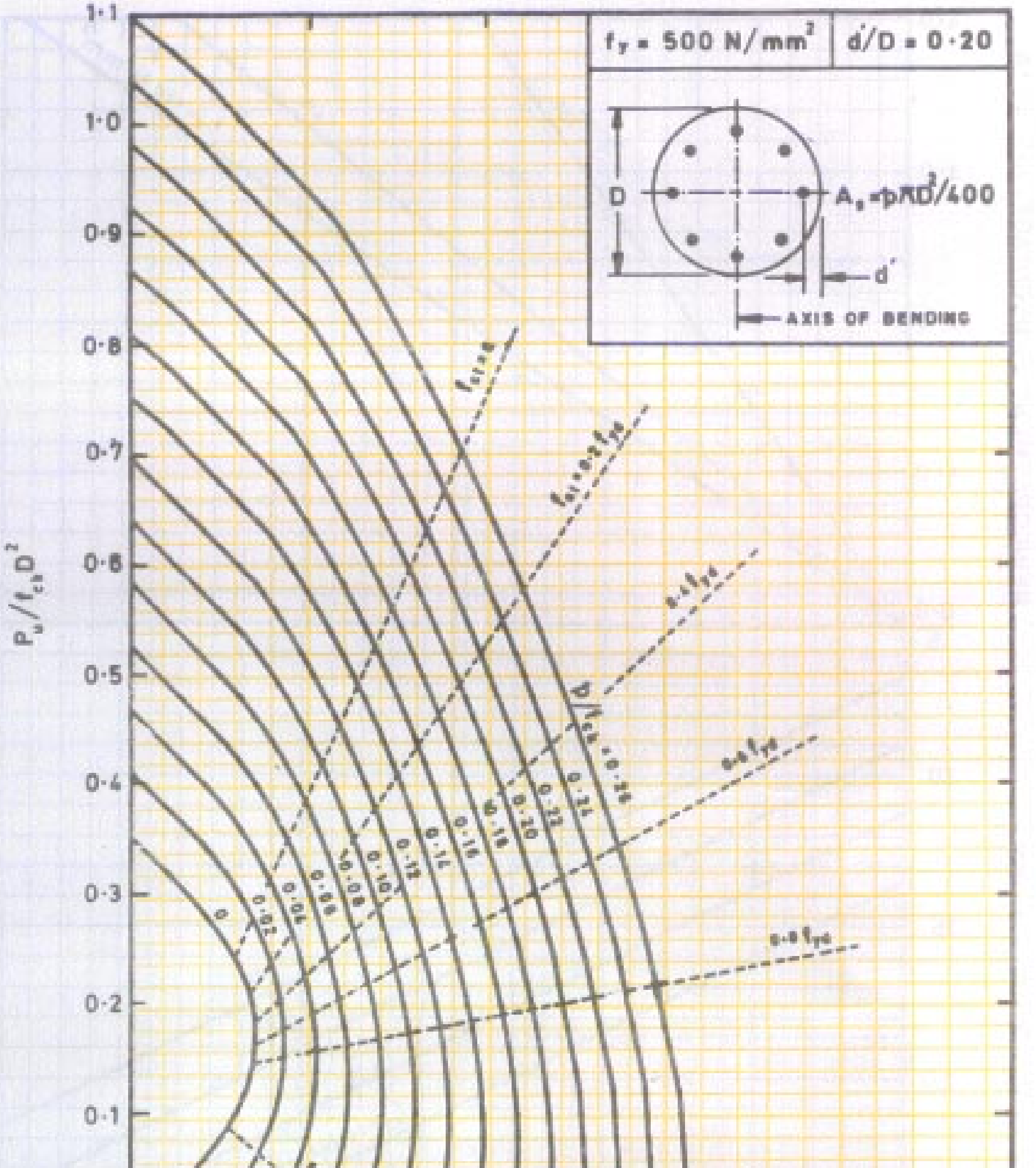


Chart 63 VALUES OF P_{uz} for COMPRESSION MEMBERS

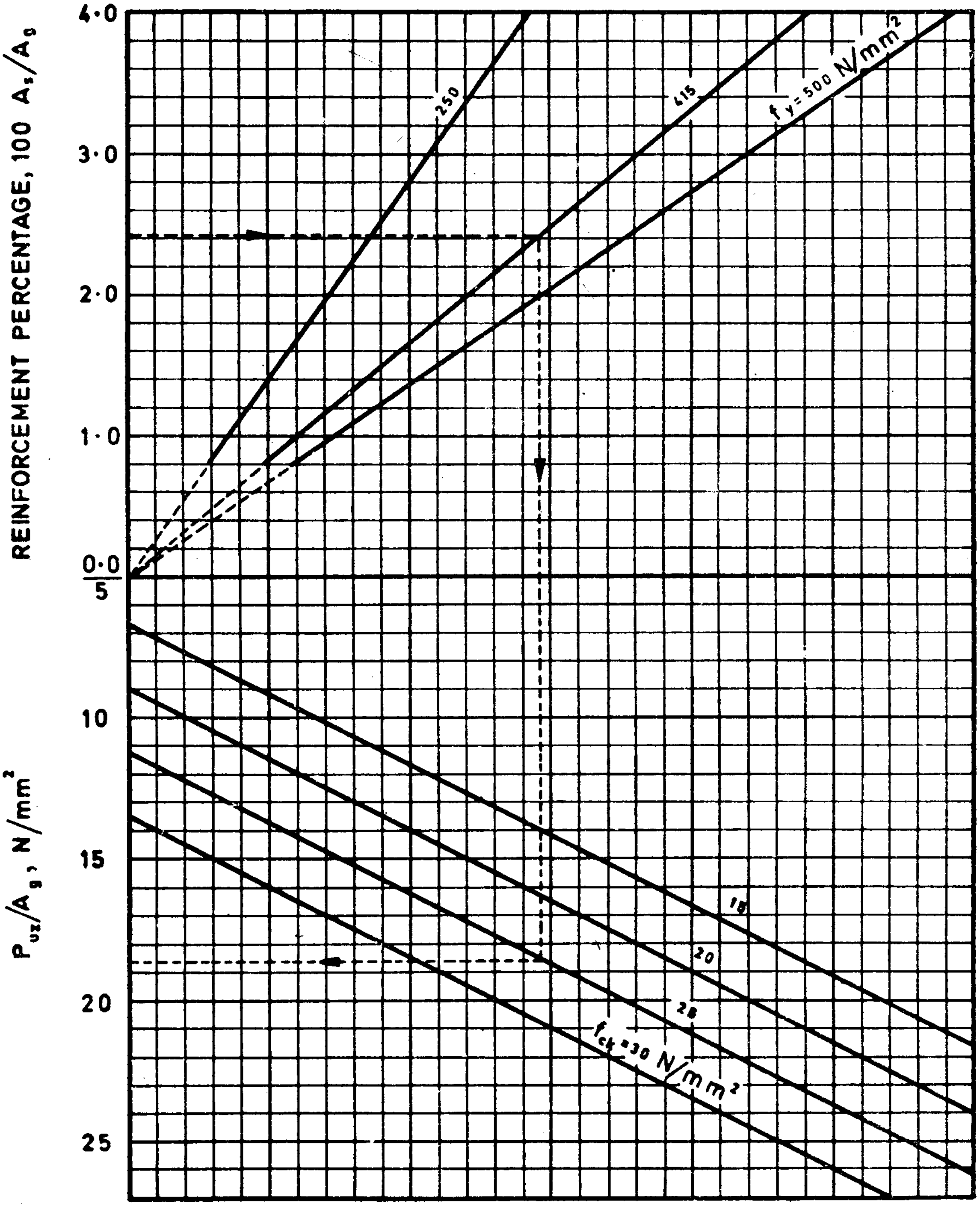


Chart 64 BIAXIAL BENDING IN COMPRESSION MEMBERS

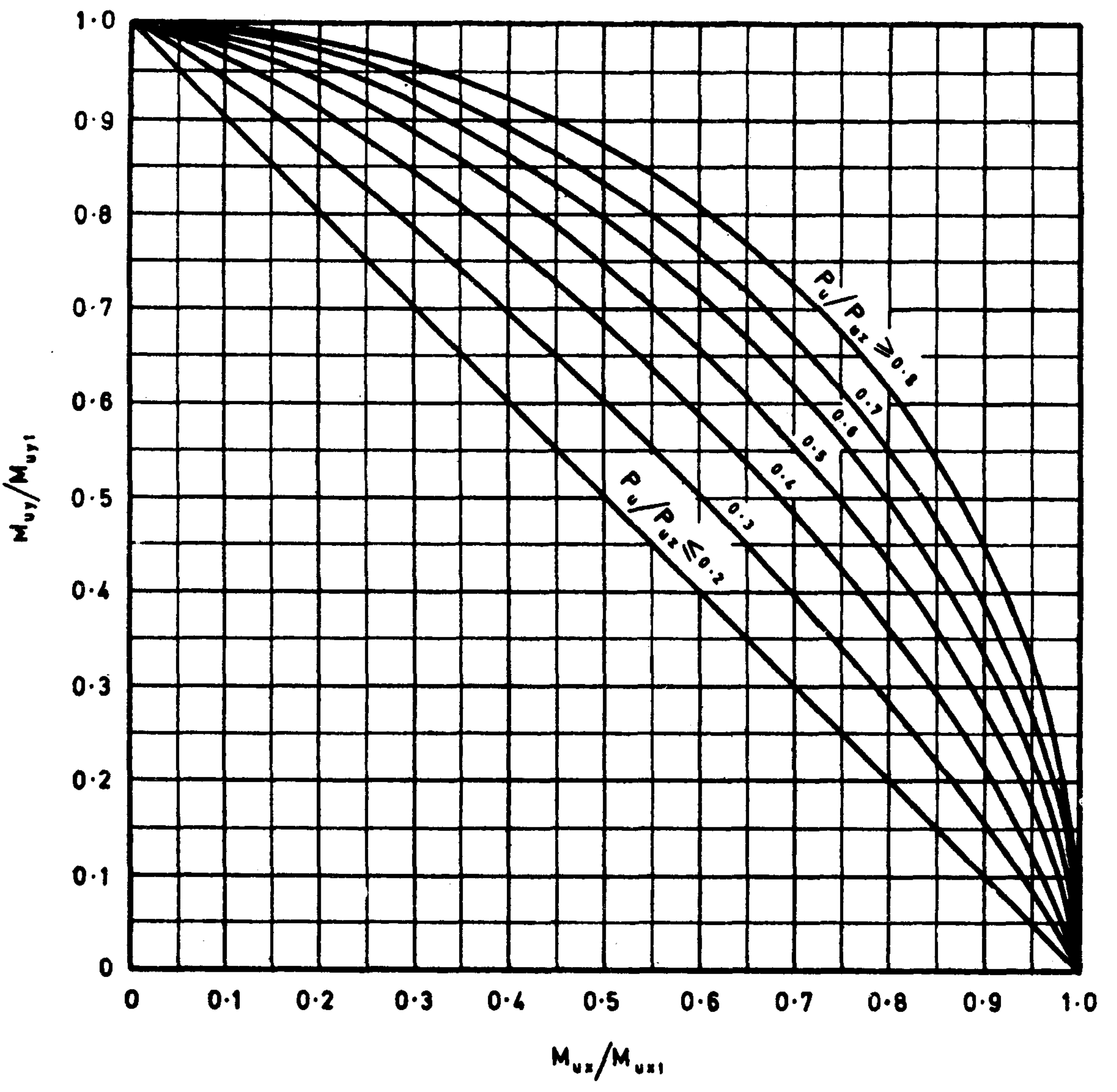


Chart 65 SLENDER COMPRESSION MEMBERS — Multiplying Factor k for Additional Moments

$$k = \frac{P_{uz} - P_u}{P_{uz} - P_b}$$

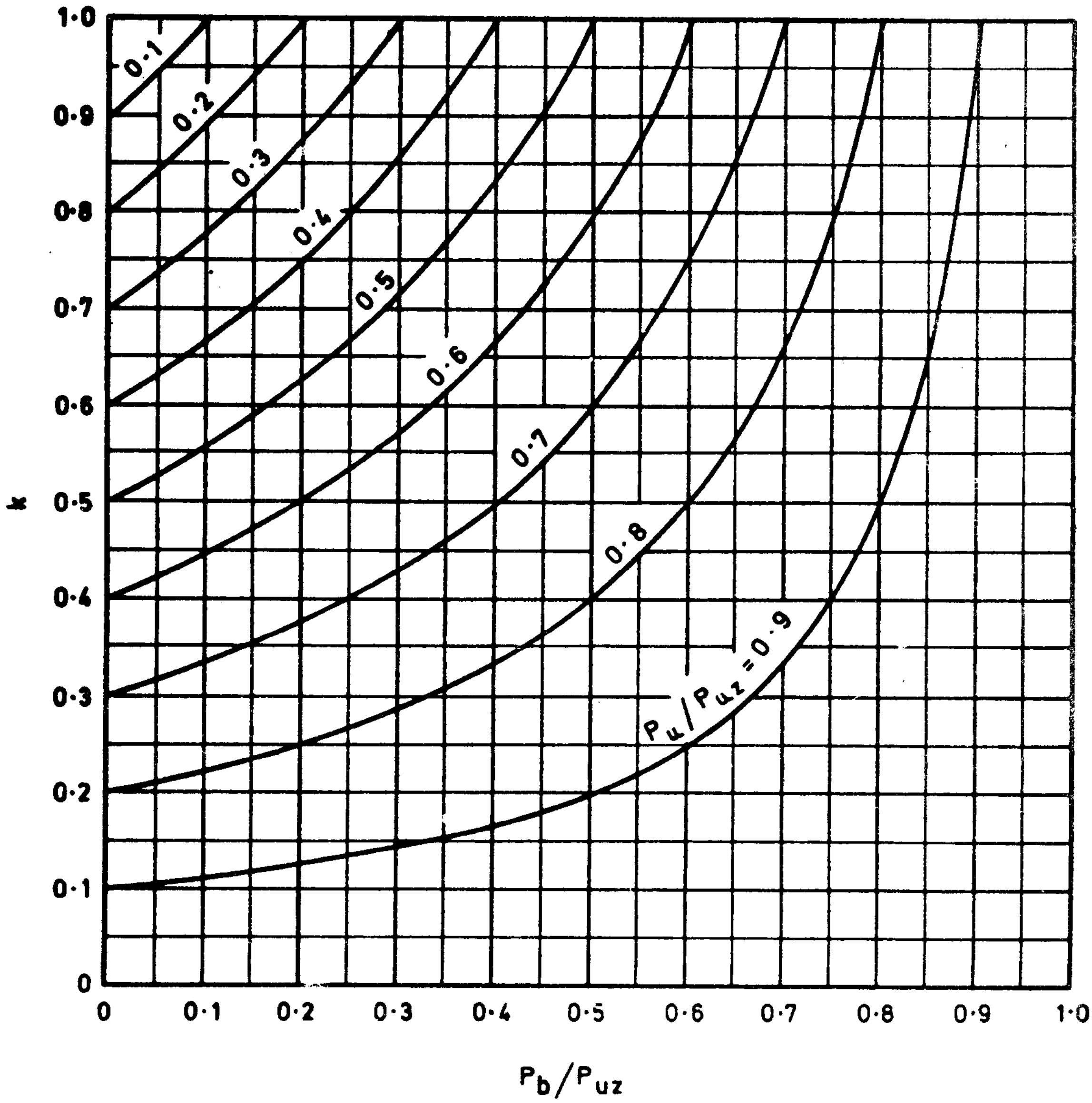
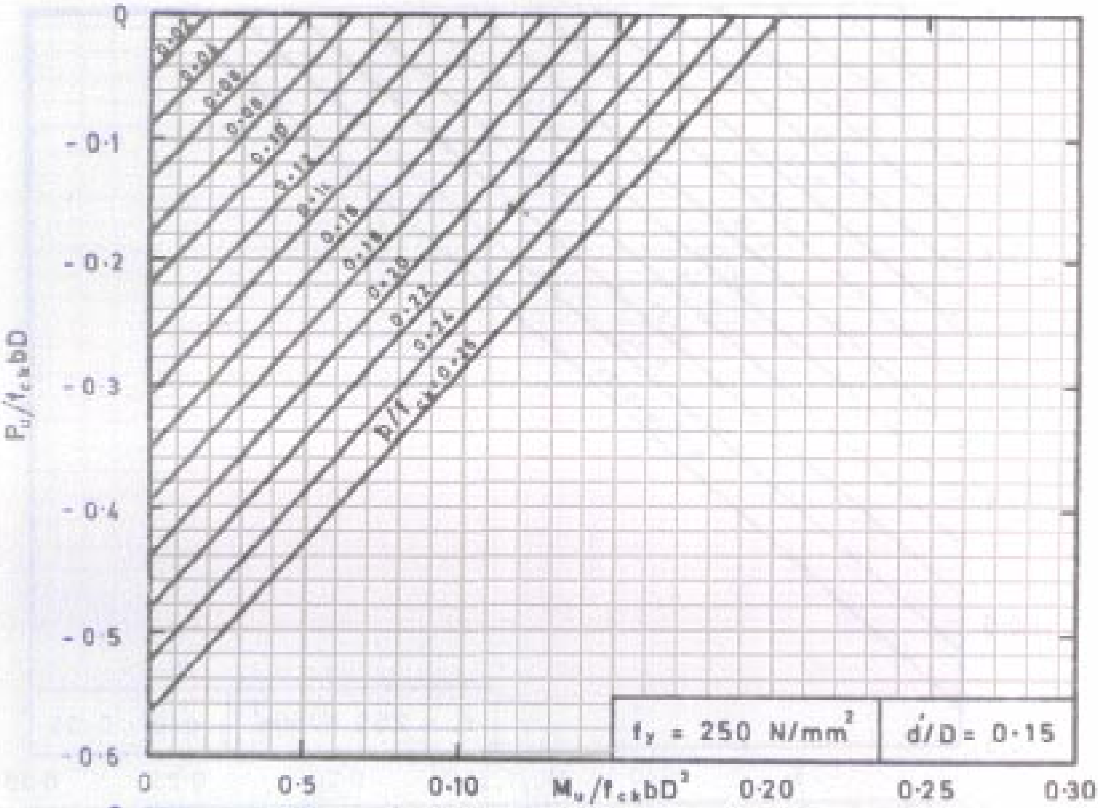


Chart 66 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides



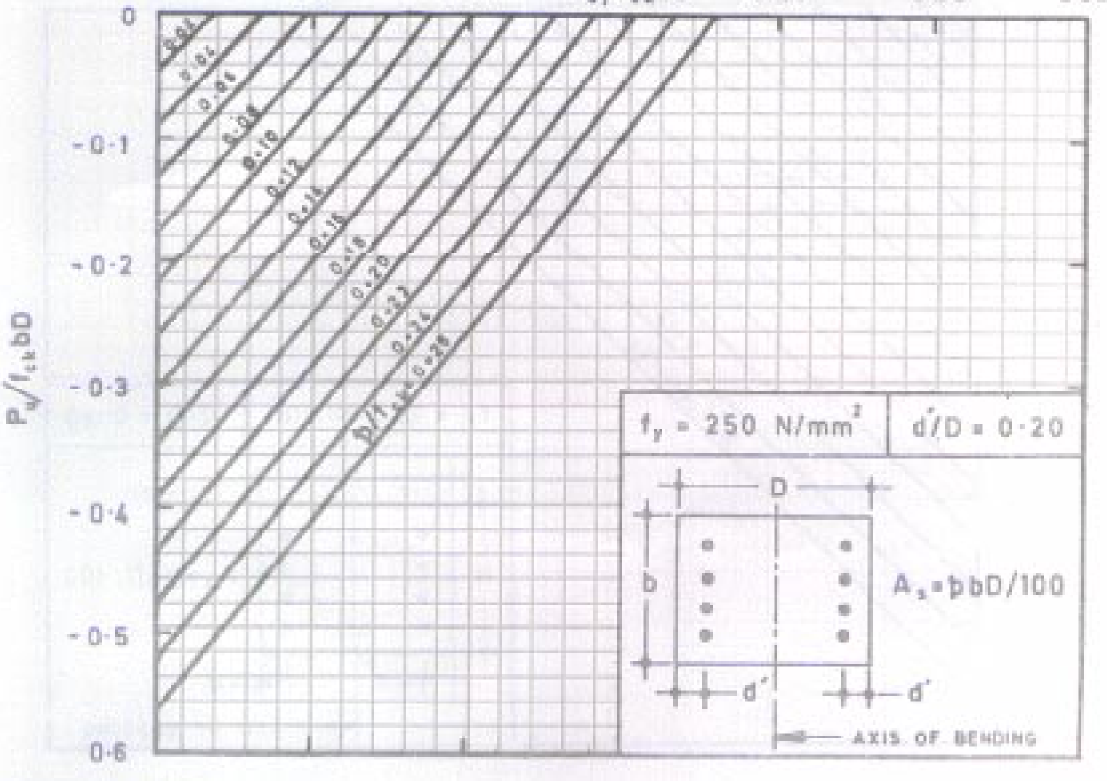
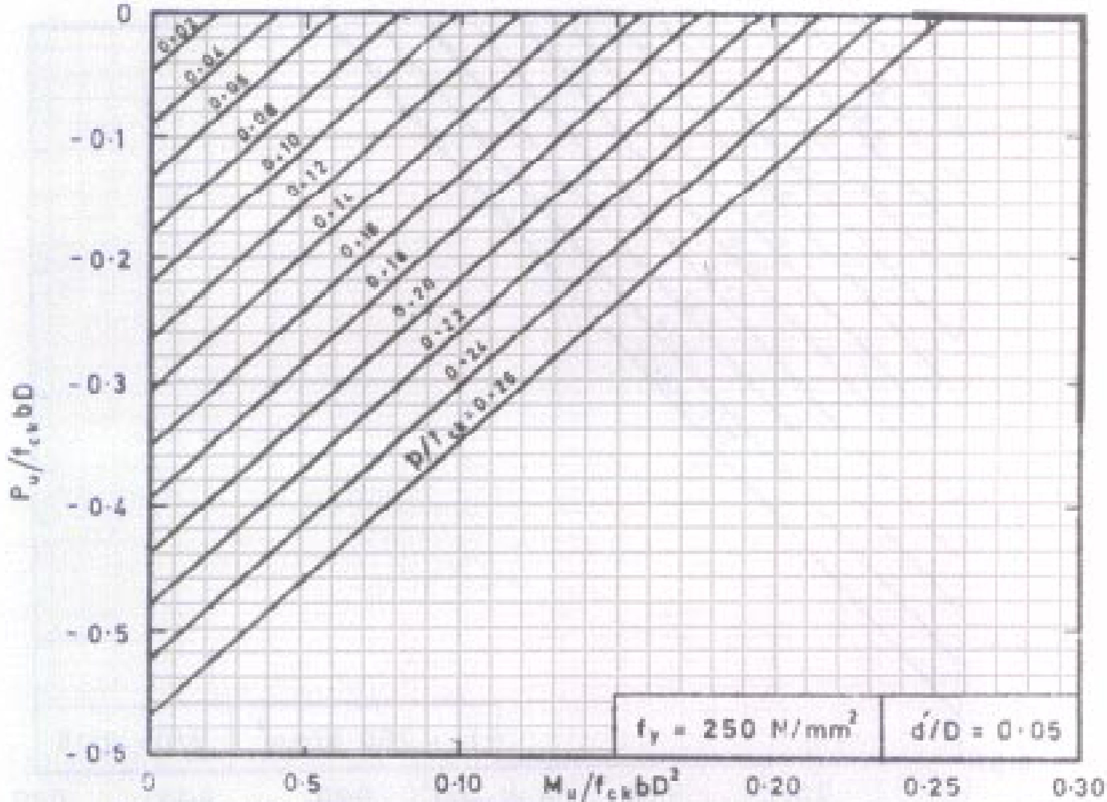


Chart 67 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides



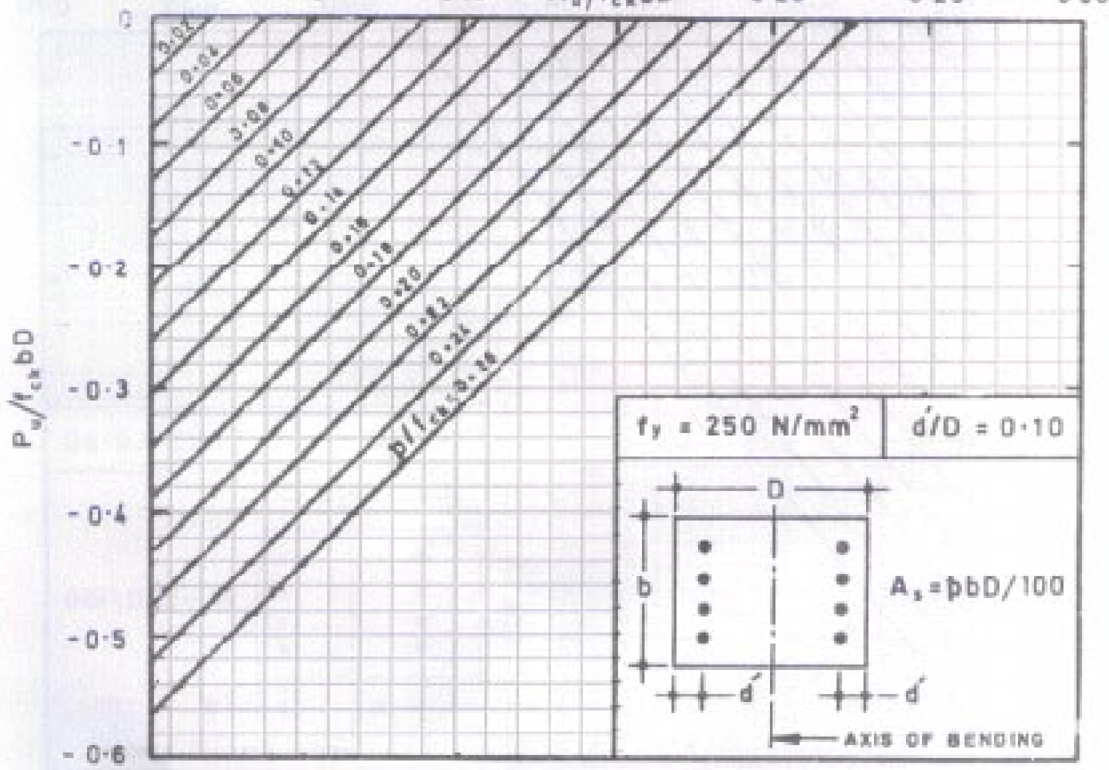


Chart 68 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

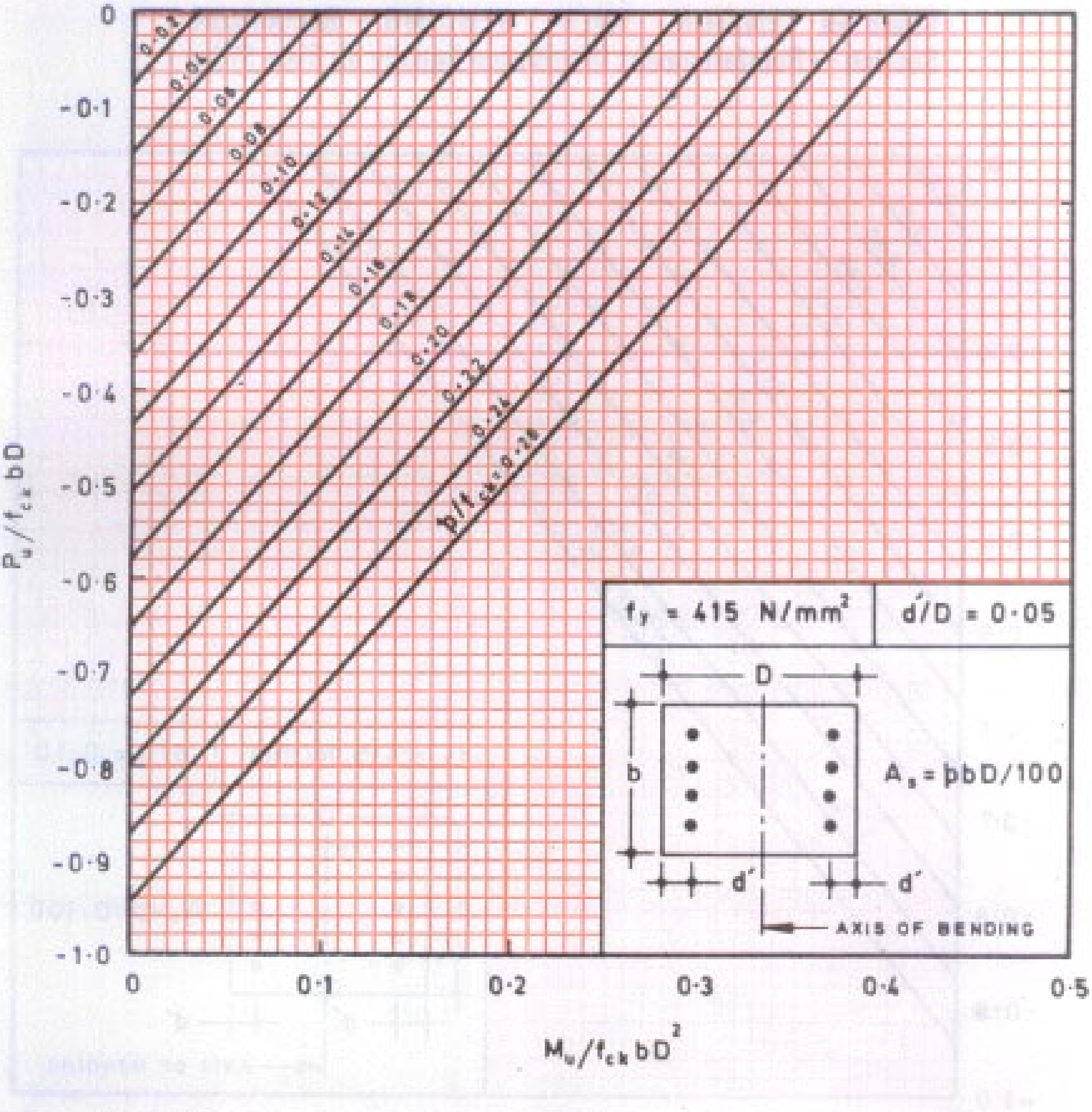


Chart 69 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

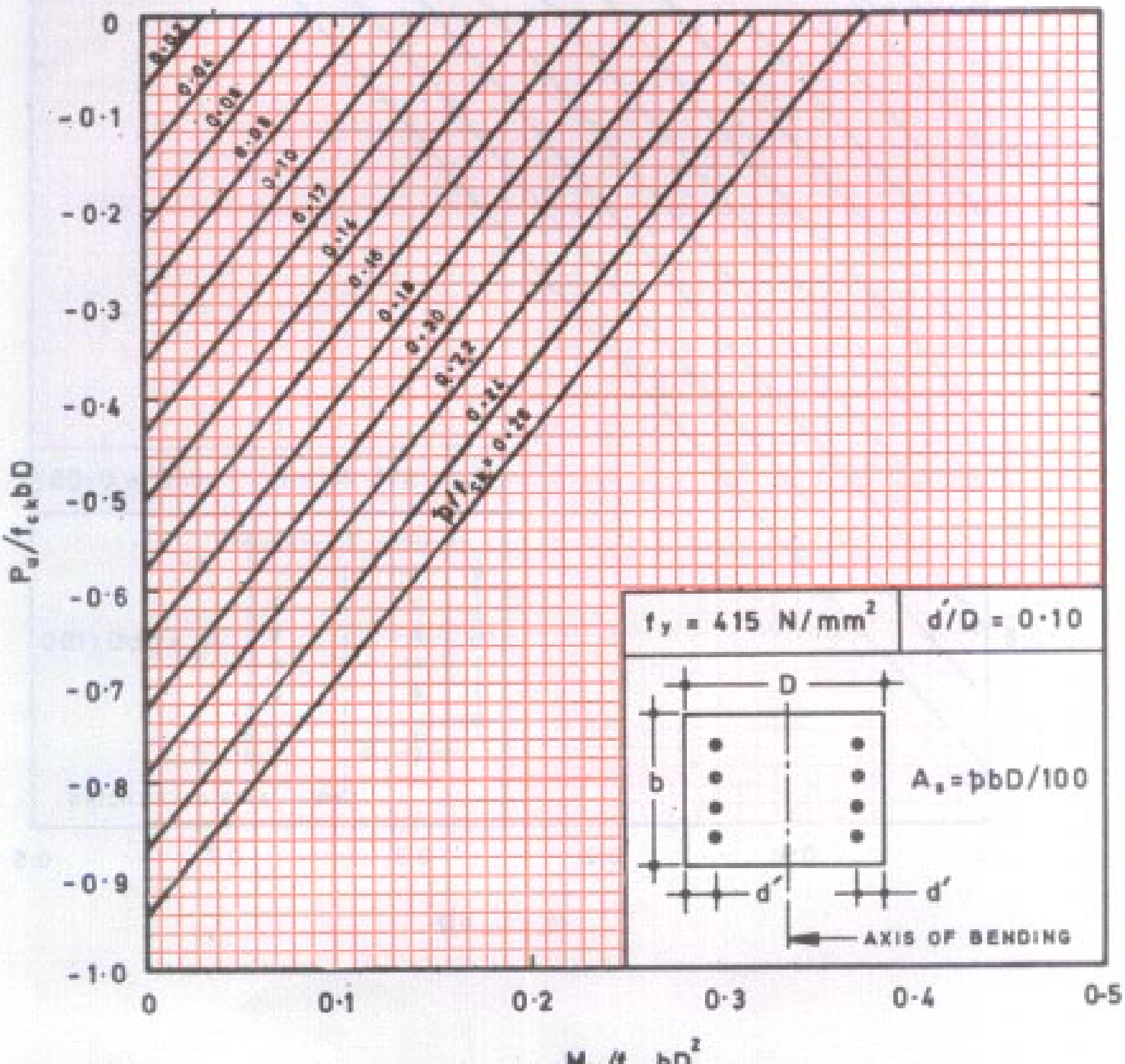


Chart 70 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

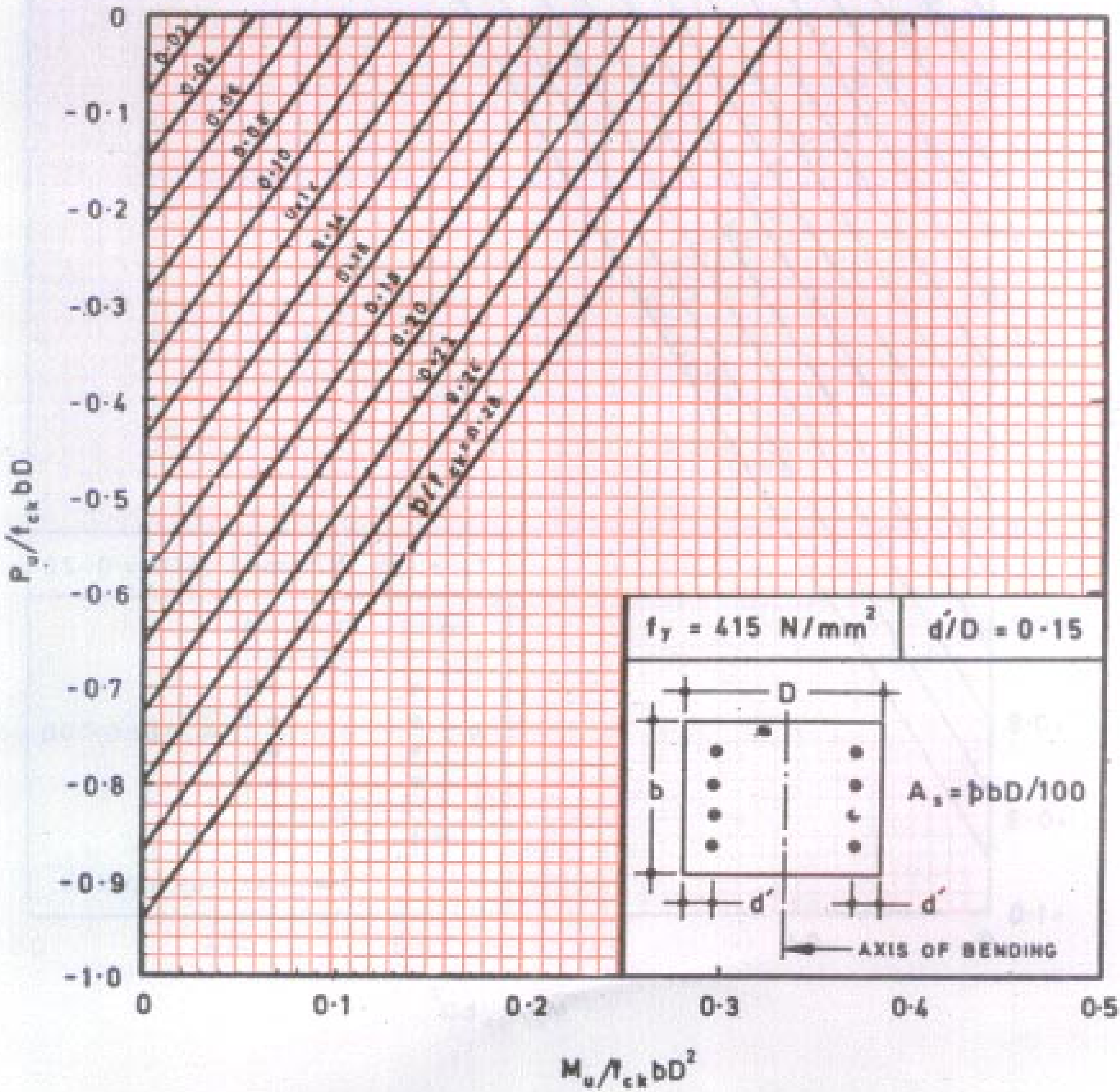


Chart 71 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

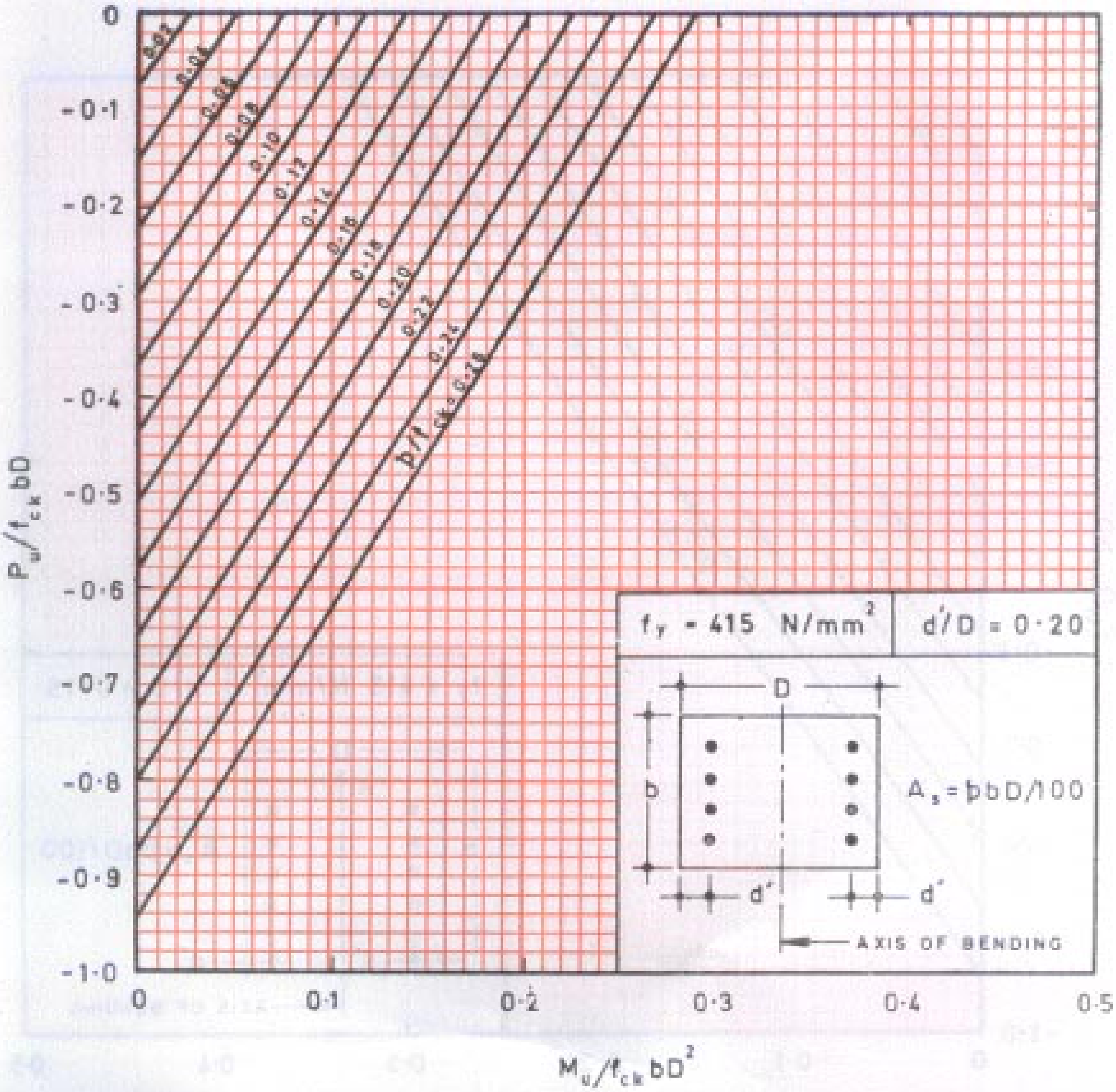


Chart 72 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

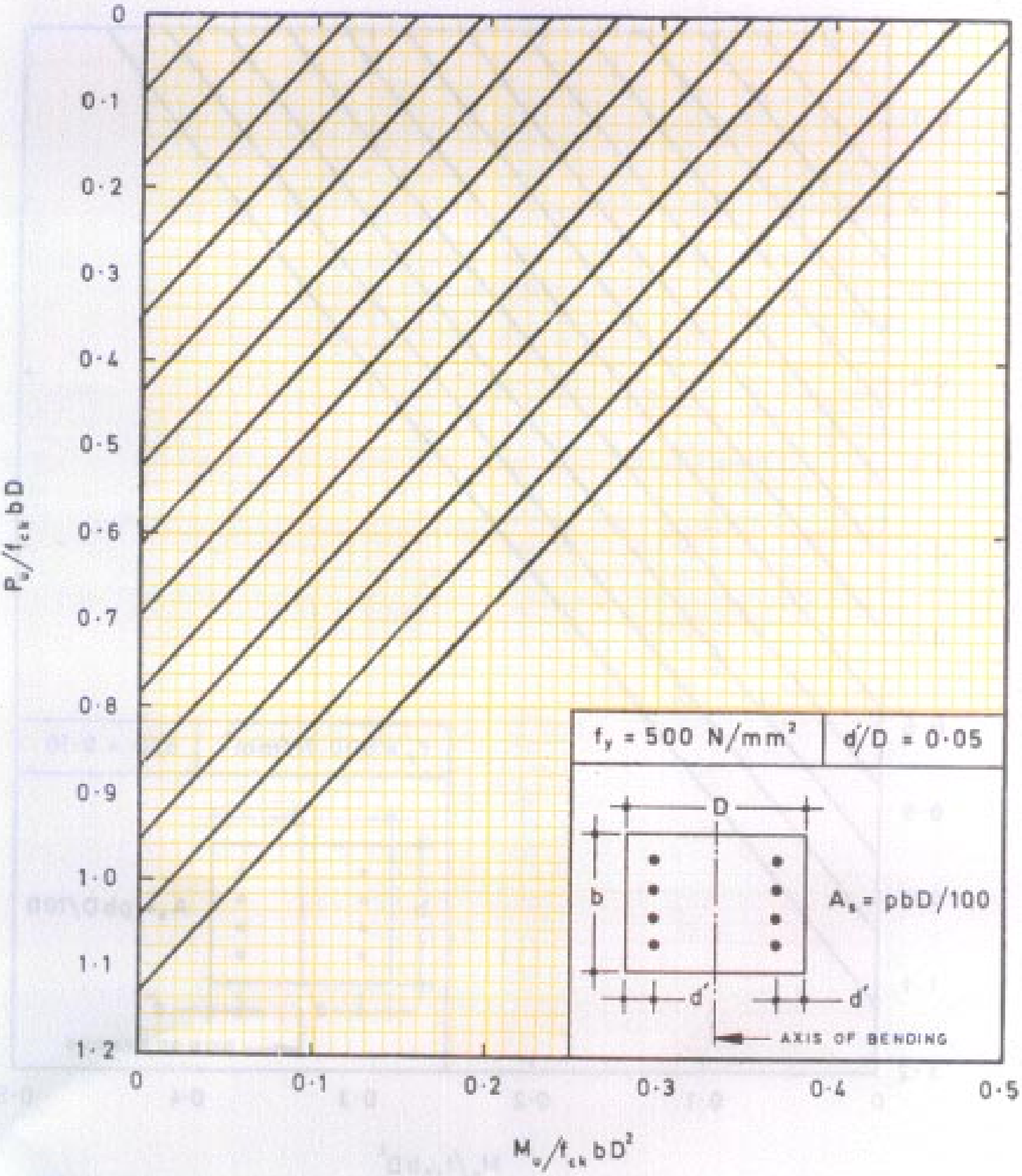


Chart 73 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

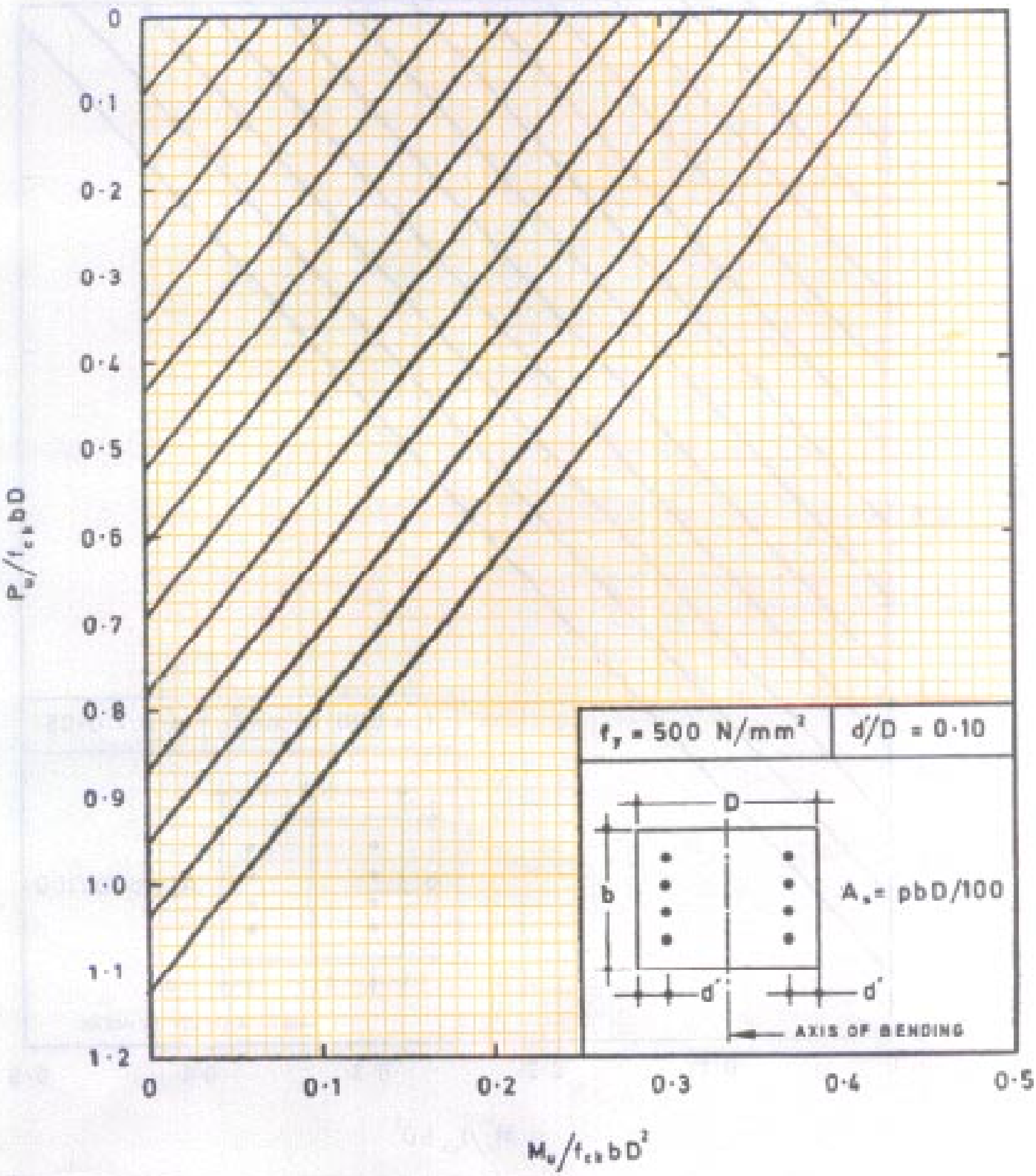


Chart 74 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

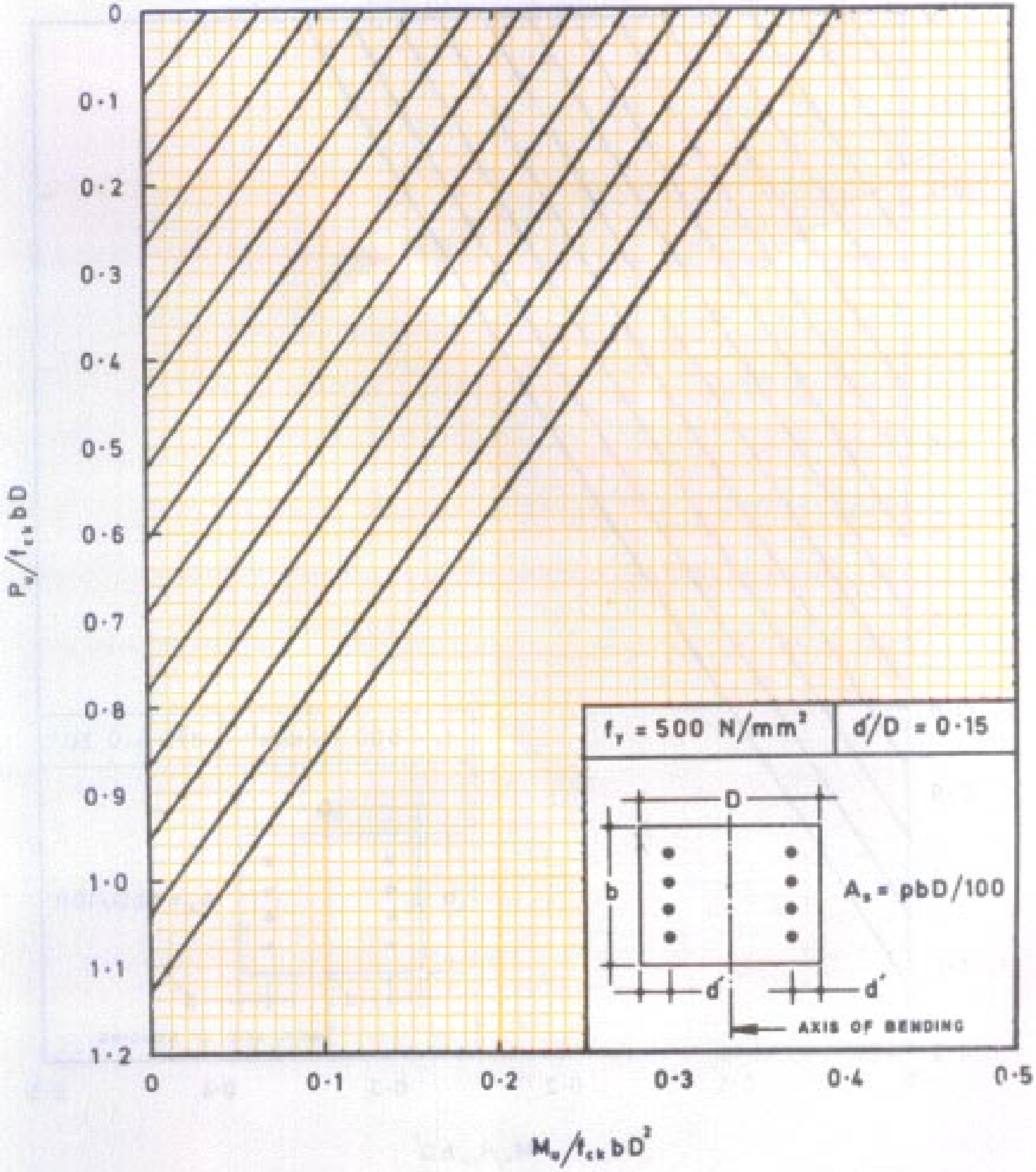


Chart 75 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides

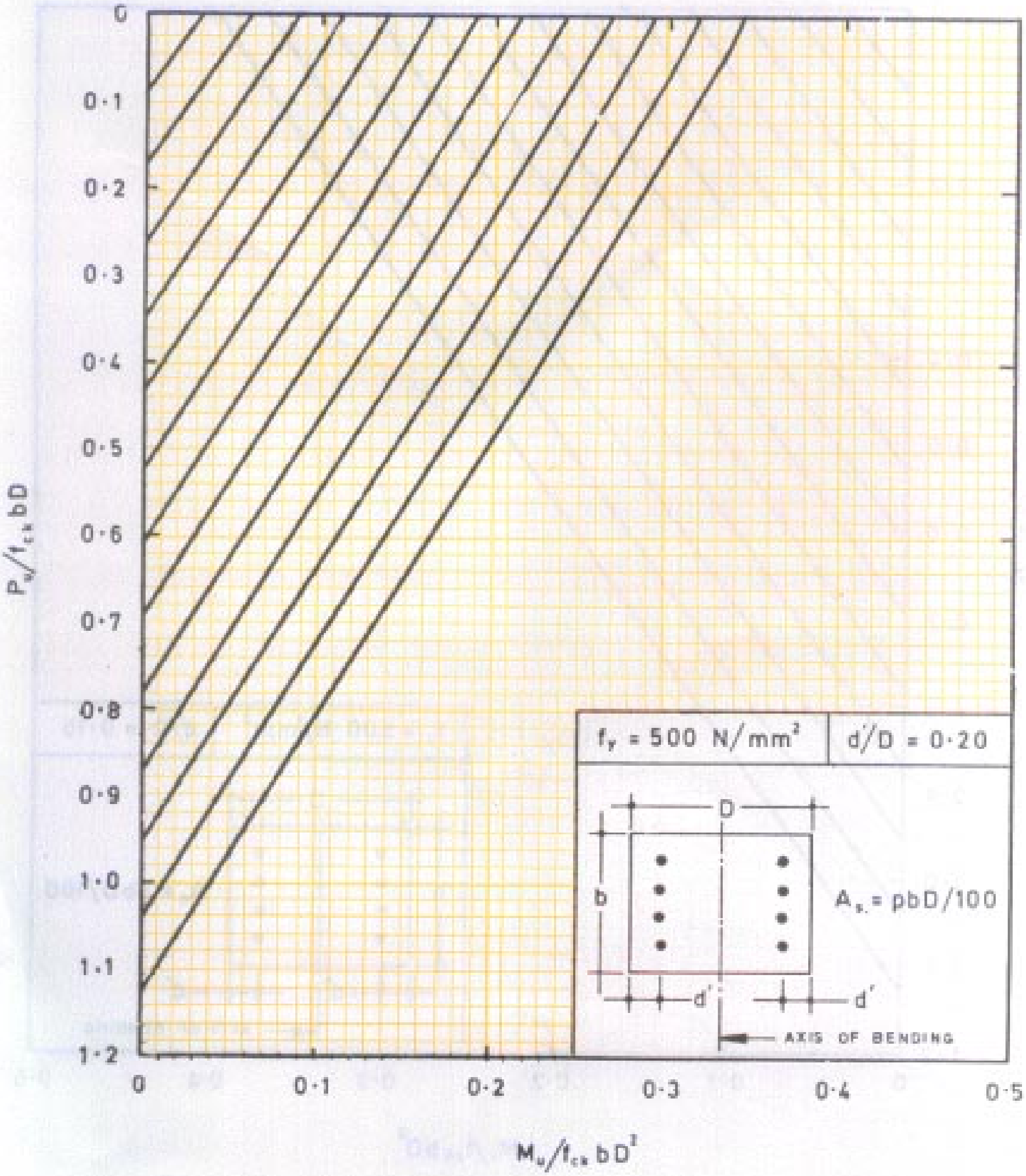
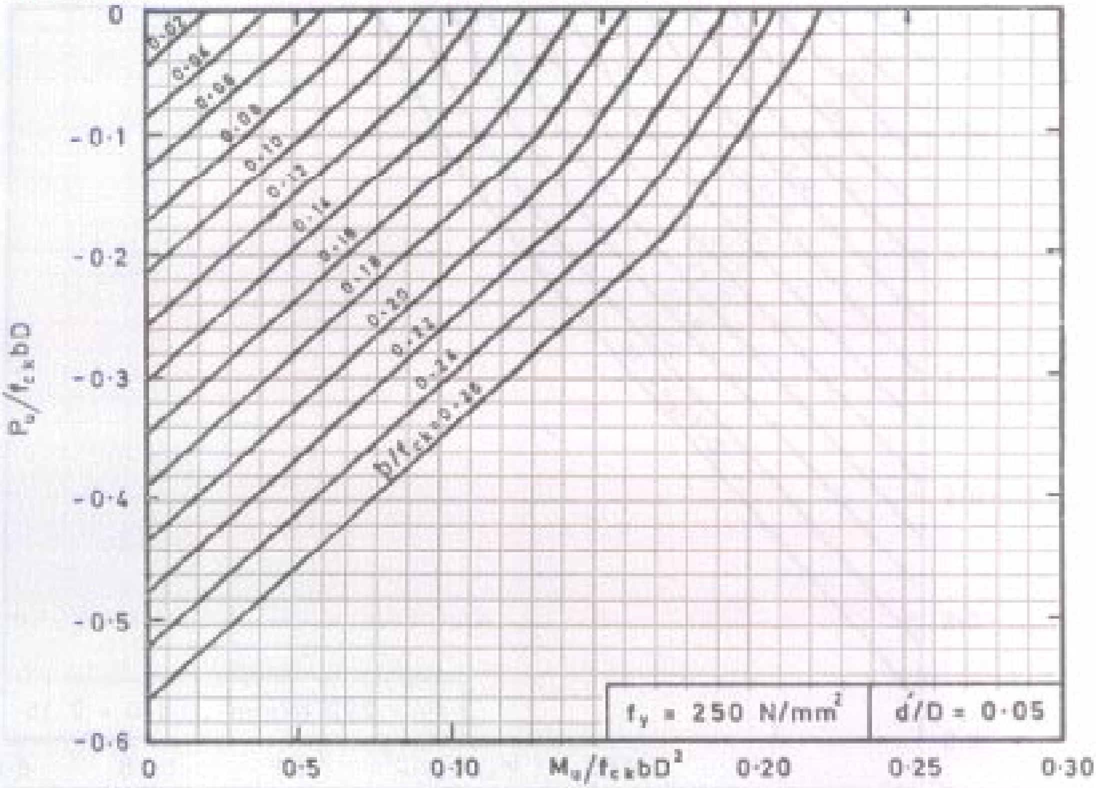


Chart 76 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides



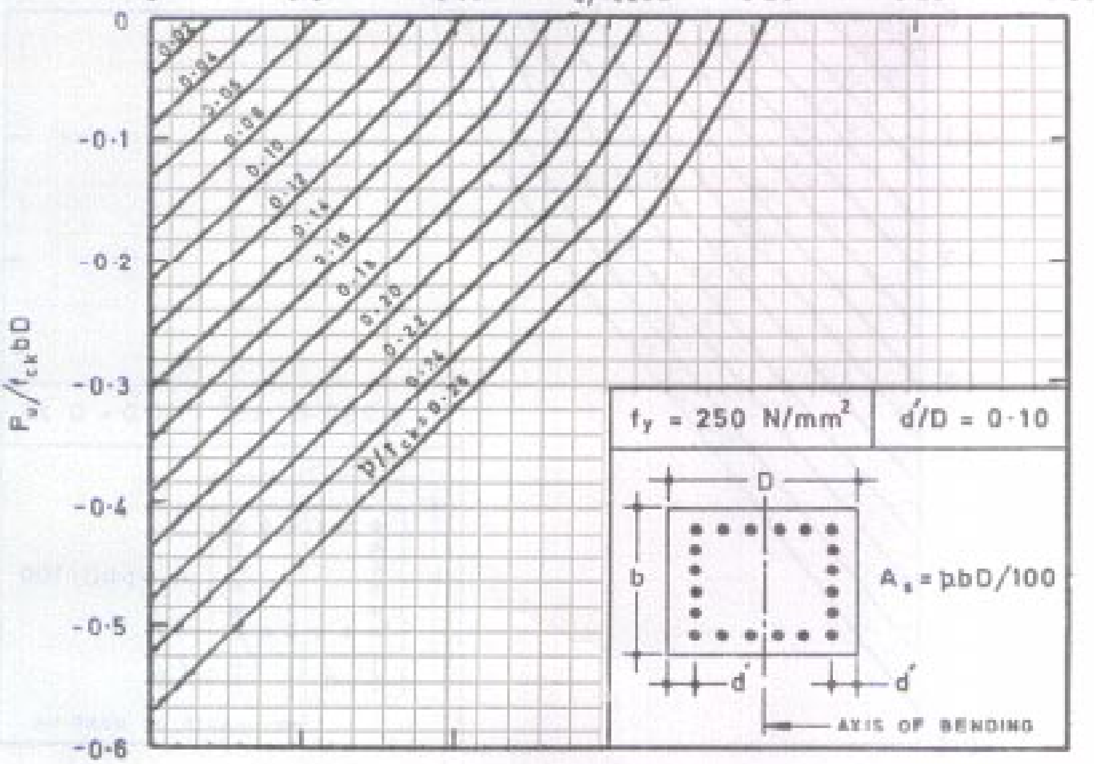
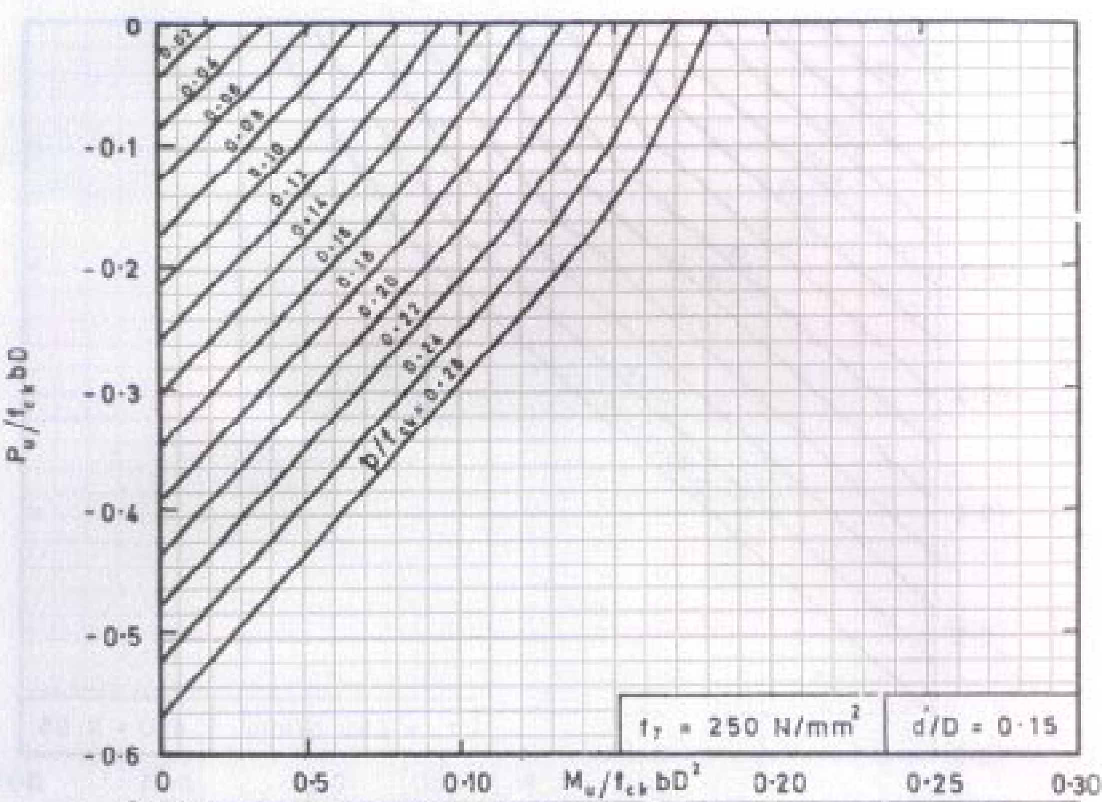


Chart 77 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides



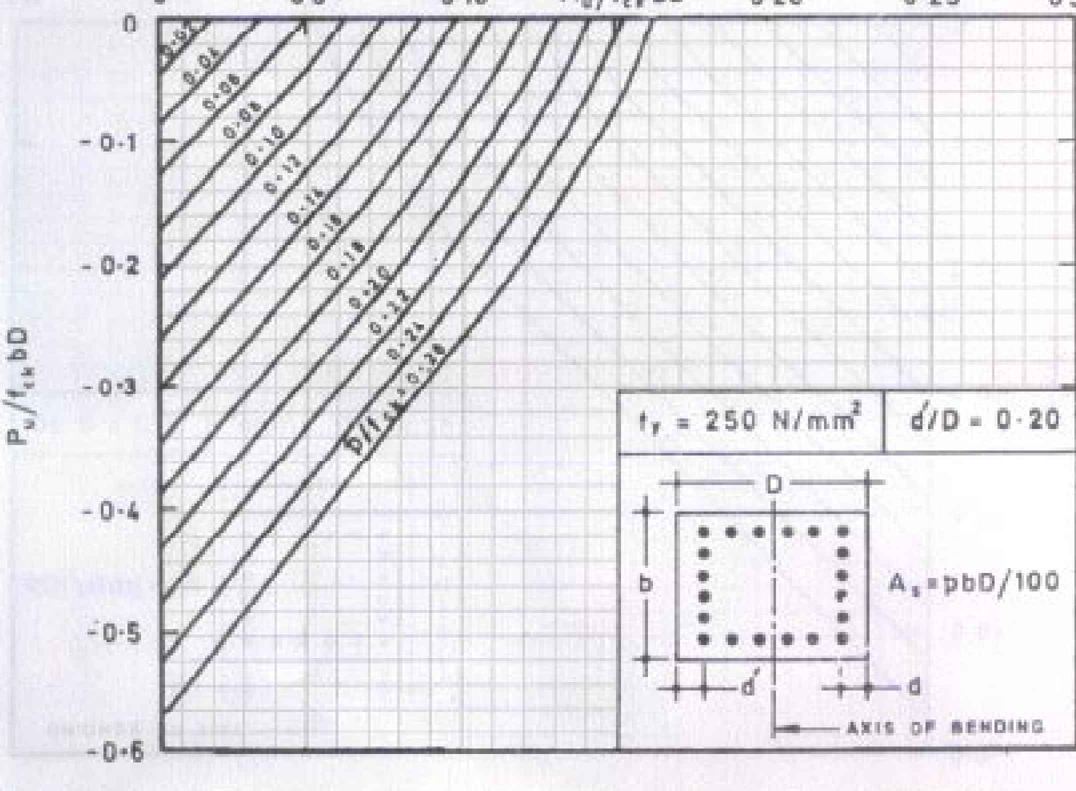


Chart 78 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

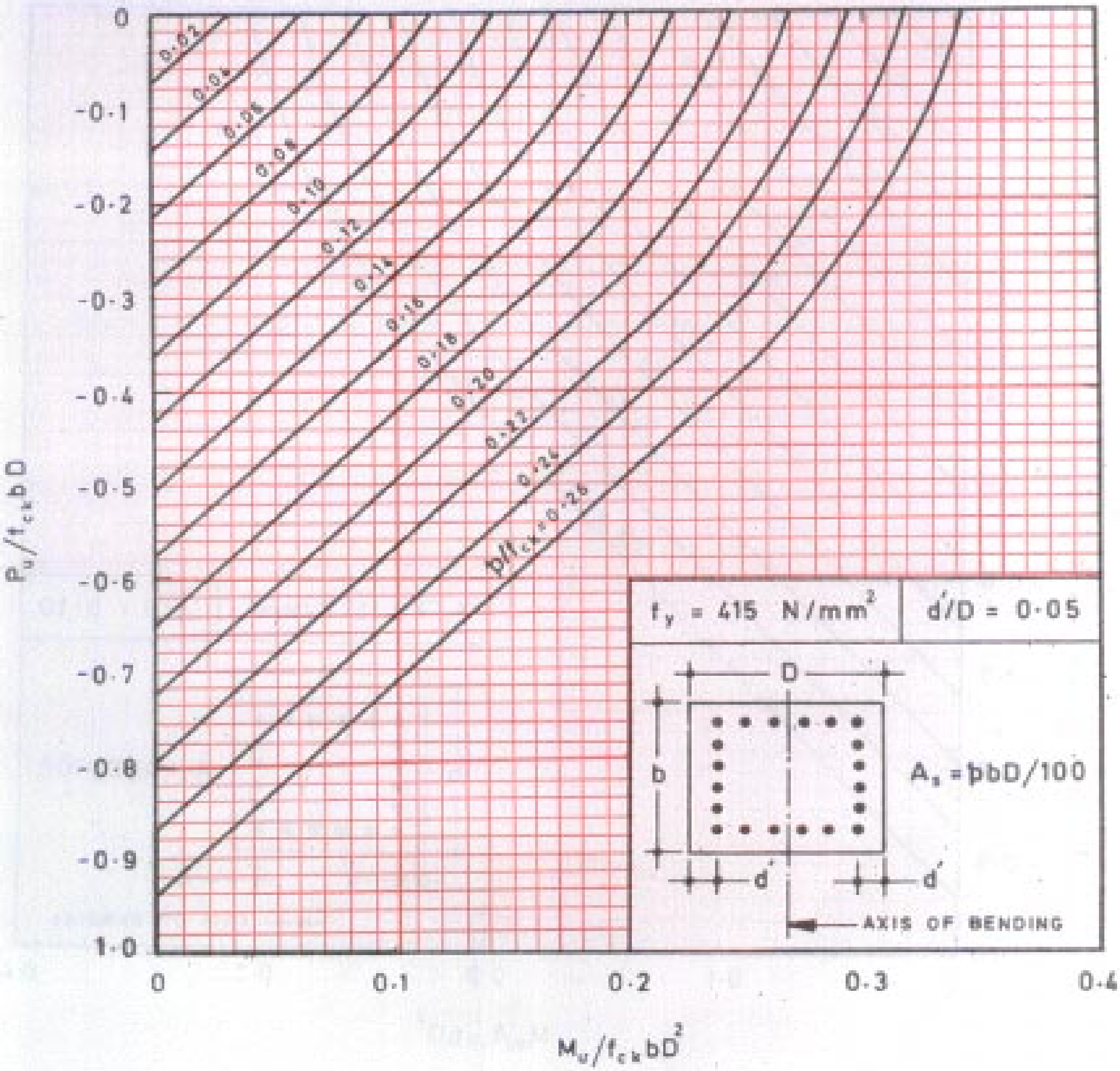


Chart 79 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

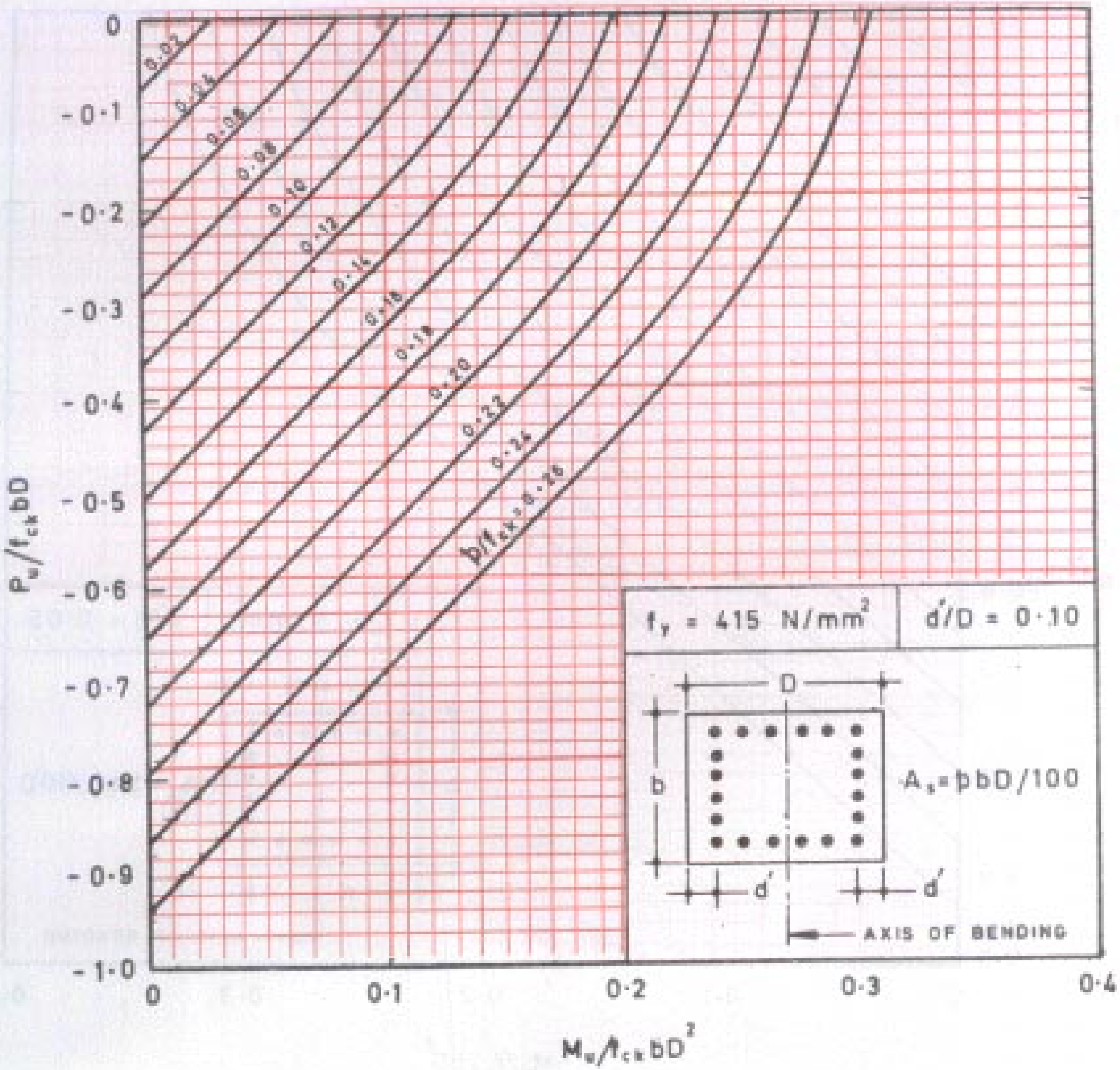


Chart 80 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

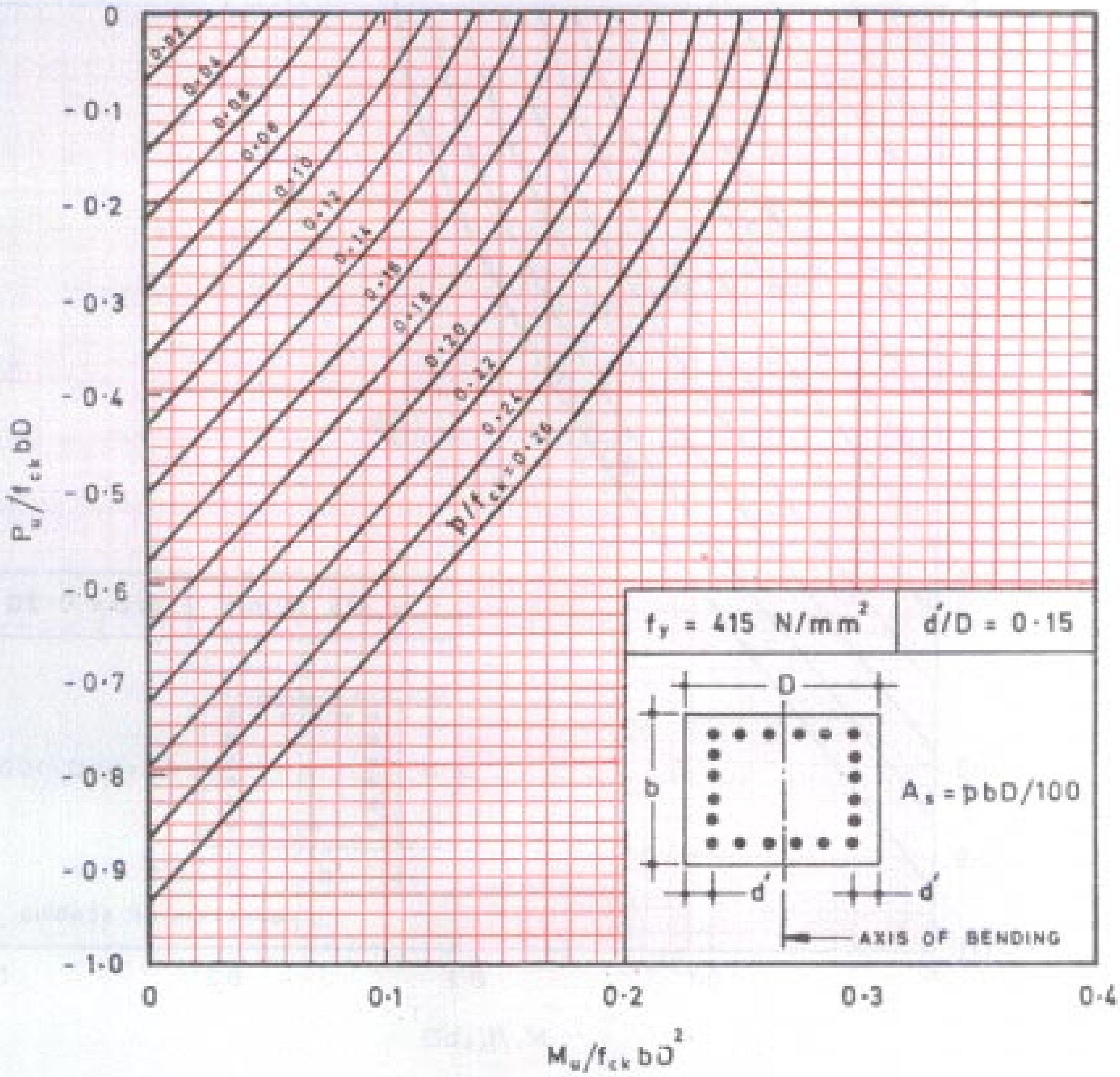


Chart 81 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

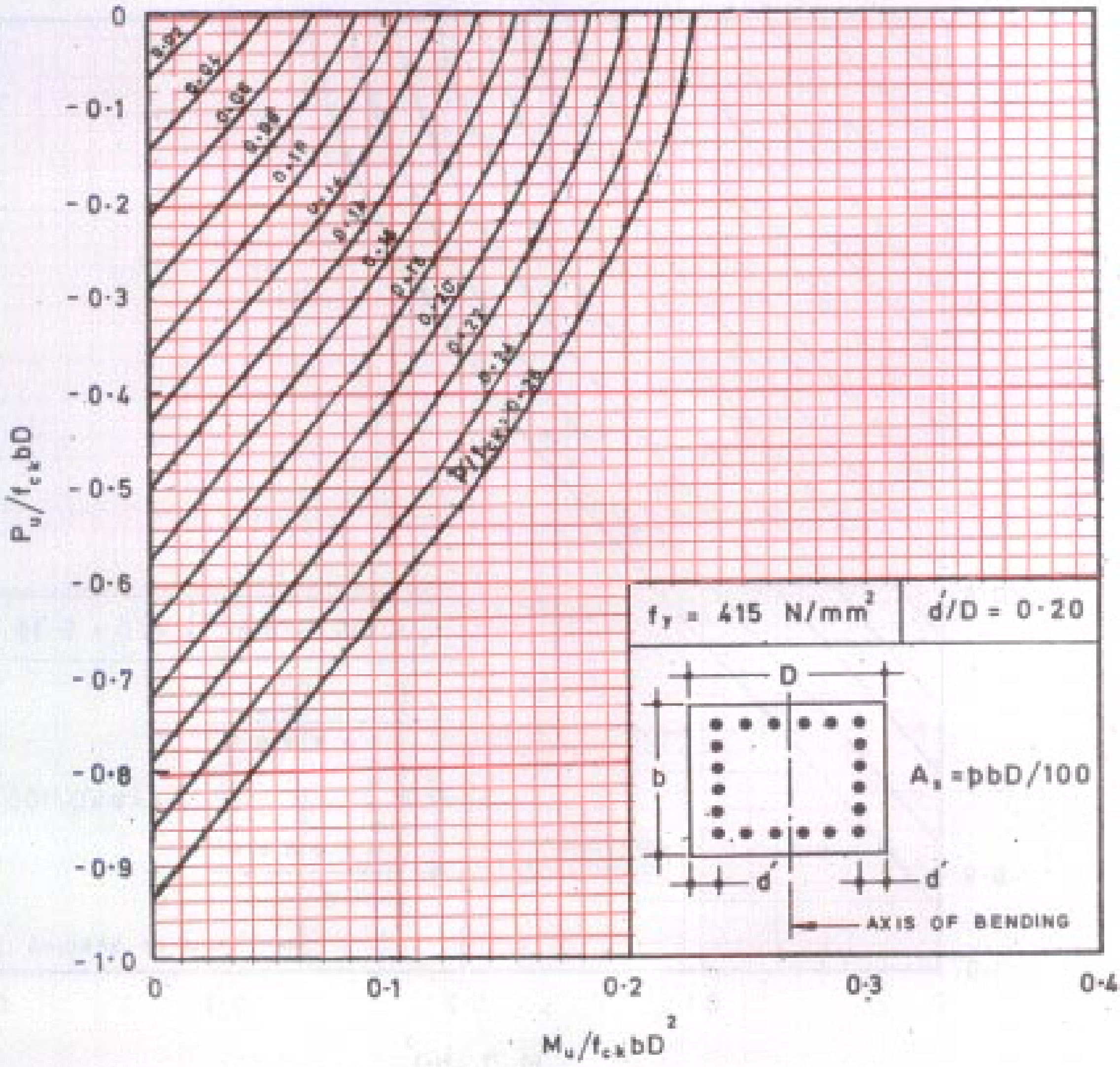


Chart 82 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

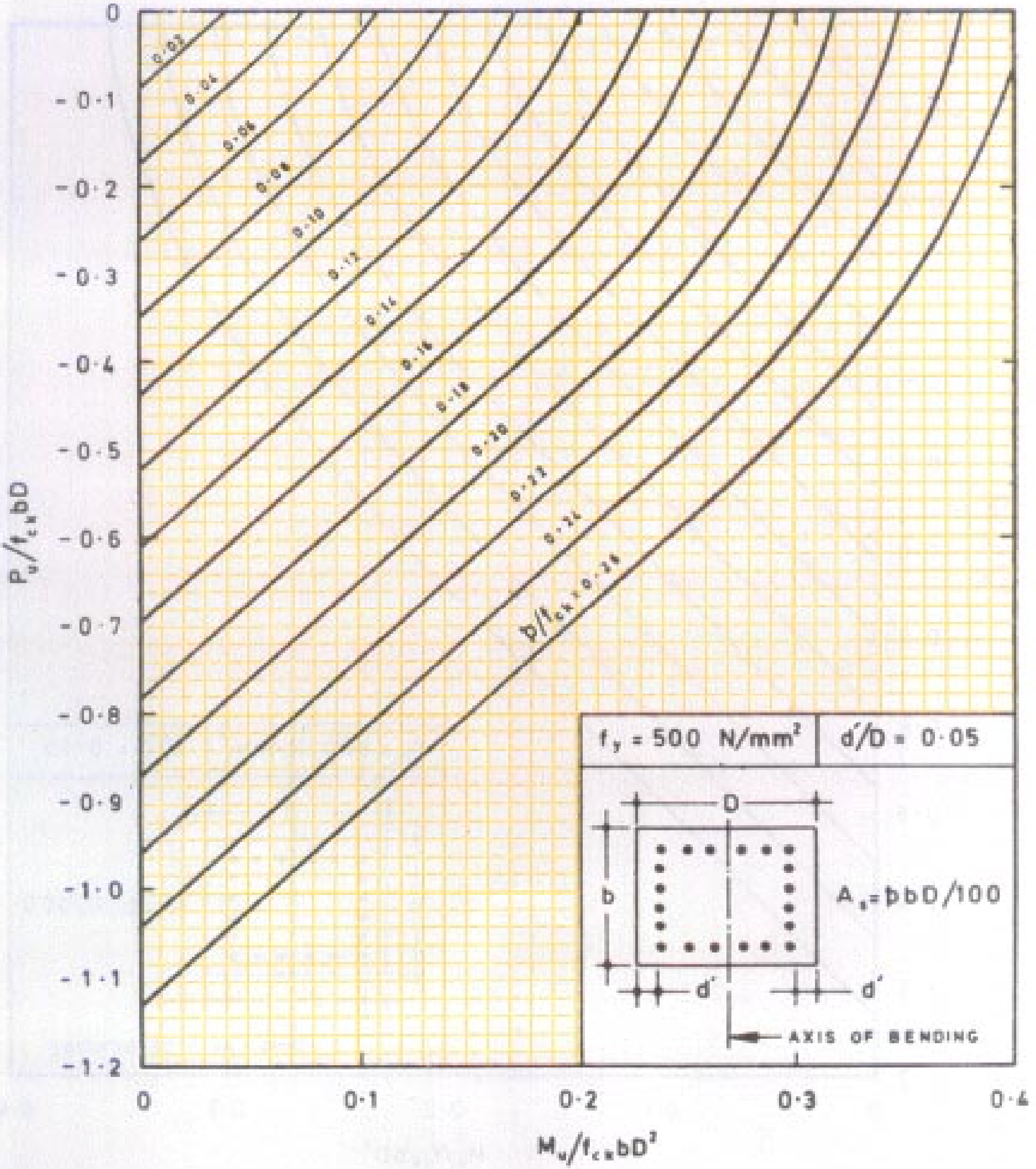


Chart 63 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

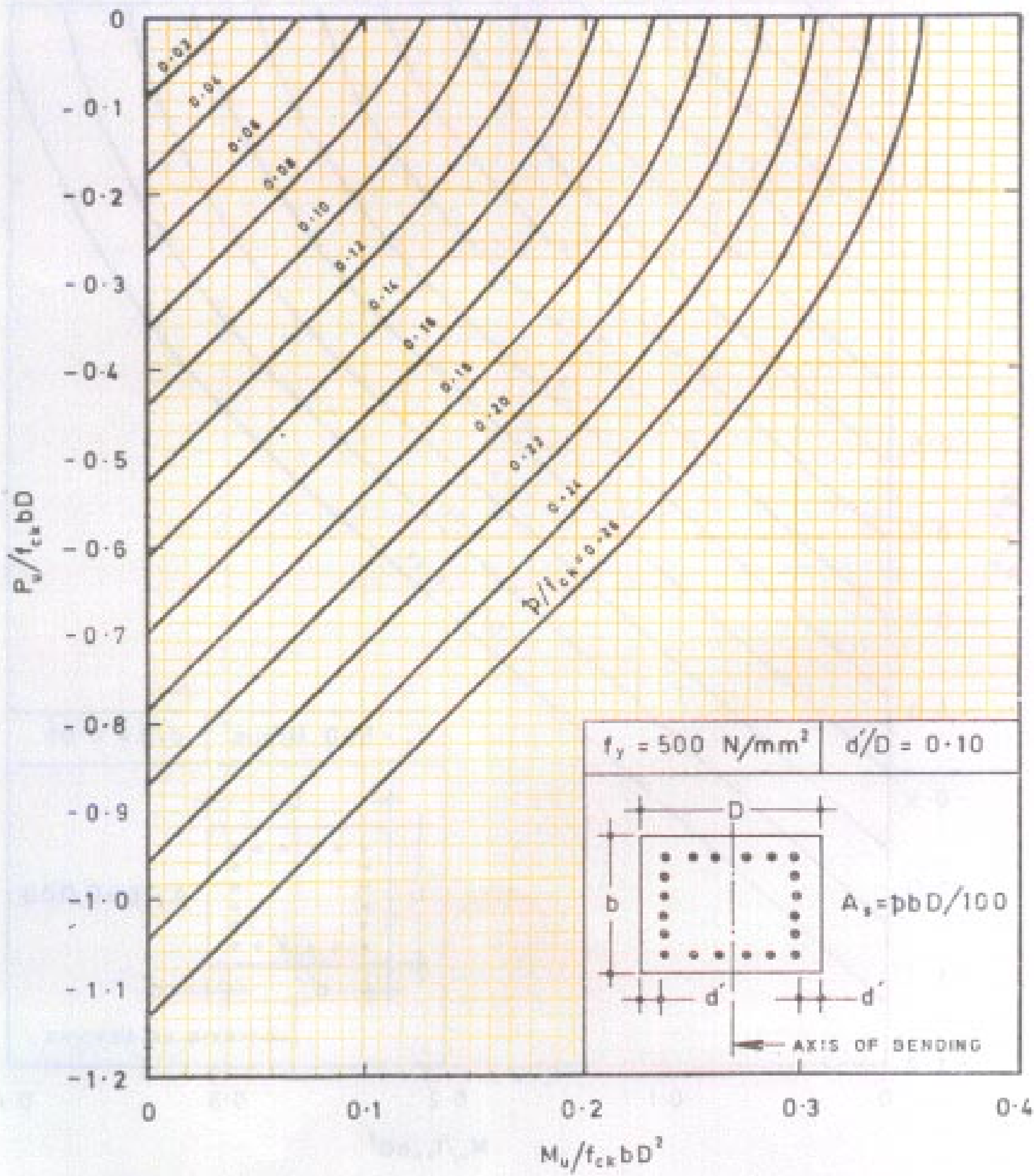


Chart 84 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

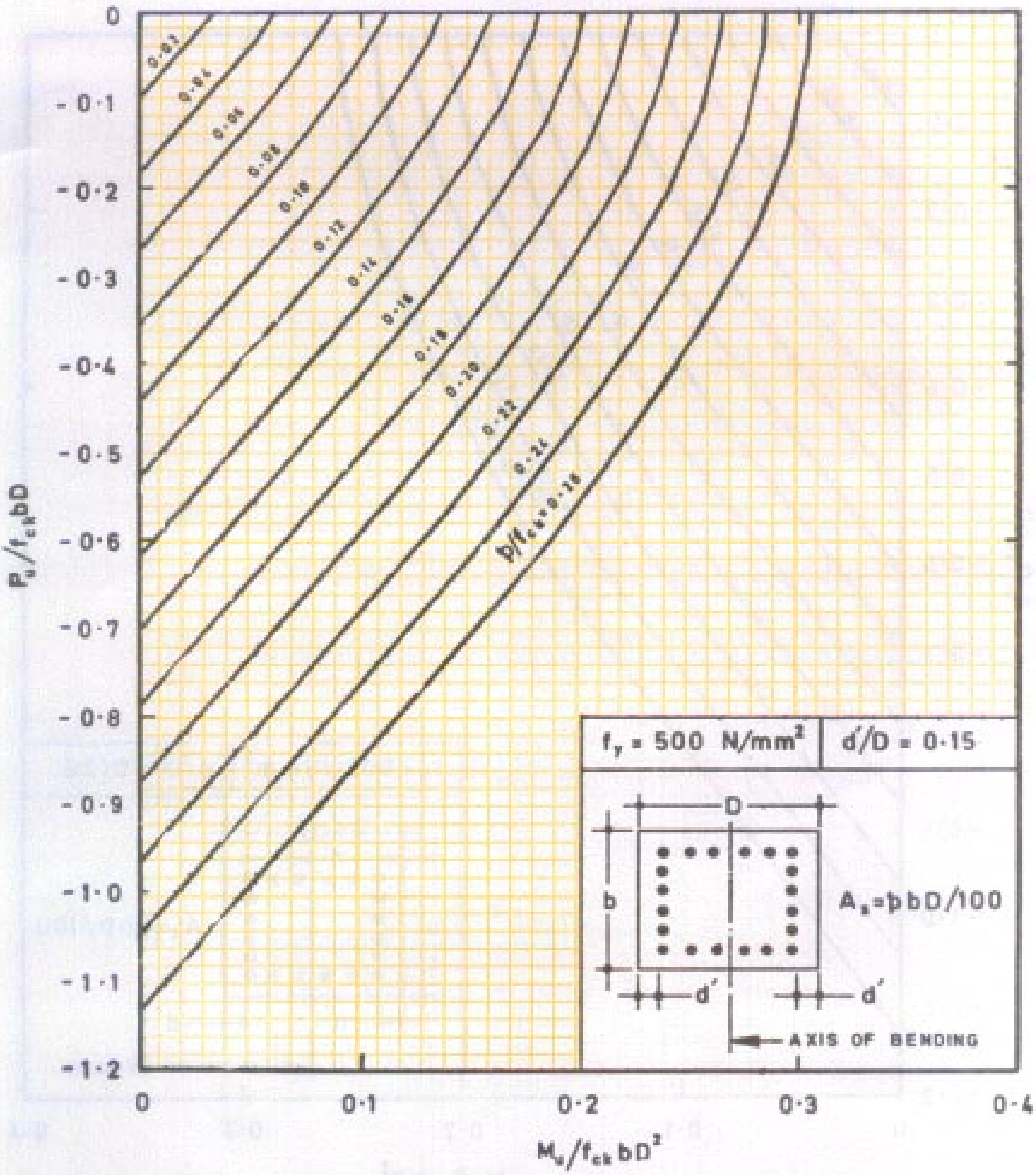


Chart 85 TENSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides

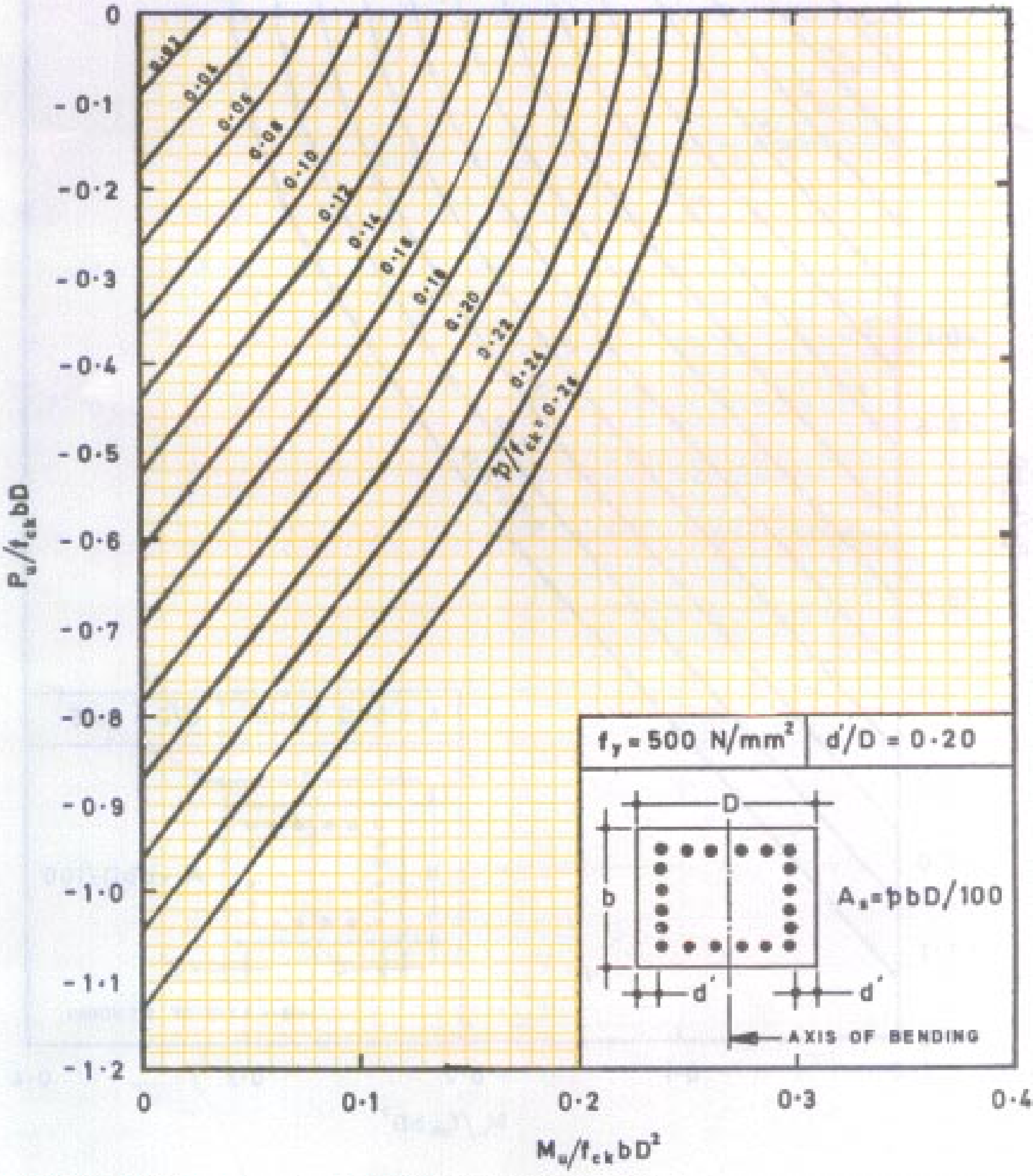


TABLE 60 SLENDER COMPRESSION MEMBERS — VALUES OF P_b

Rectangular Sections:

$$P_b/f_{ck}\,bD = k_1 + k_2\,p/f_{ck}$$

Circular Sections:

$$P_b/f_{ck}D^2 = k_1 + k_2 p/f_{ck}$$

Values of k_1

Section	d'/D			
	0·05	0·10	0·15	0·20
Rectangular	0·219	0·207	0·196	0·184
Circular	0·172	0·160	0·149	0·138

Values of k_2

Section	f_y N/mm²	d'/D			
		0·05	0·10	0·15	0·20
Rectangular; equal reinforcement on two opposite sides	250	−0·045	−0·045	−0·045	−0·045
	415	0·096	0·082	0·046	−0·022
	500	0·213	0·173	0·104	−0·001
Rectangular; equal reinforcement on four sides	250	0·215	0·146	0·061	−0·011
	415	0·424	0·328	0·203	0·028
	500	0·545	0·425	0·256	0·040
Circular	250	0·193	0·148	0·077	−0·020
	415	0·410	0·323	0·201	0·036
	500	0·543	0·443	0·291	0·056

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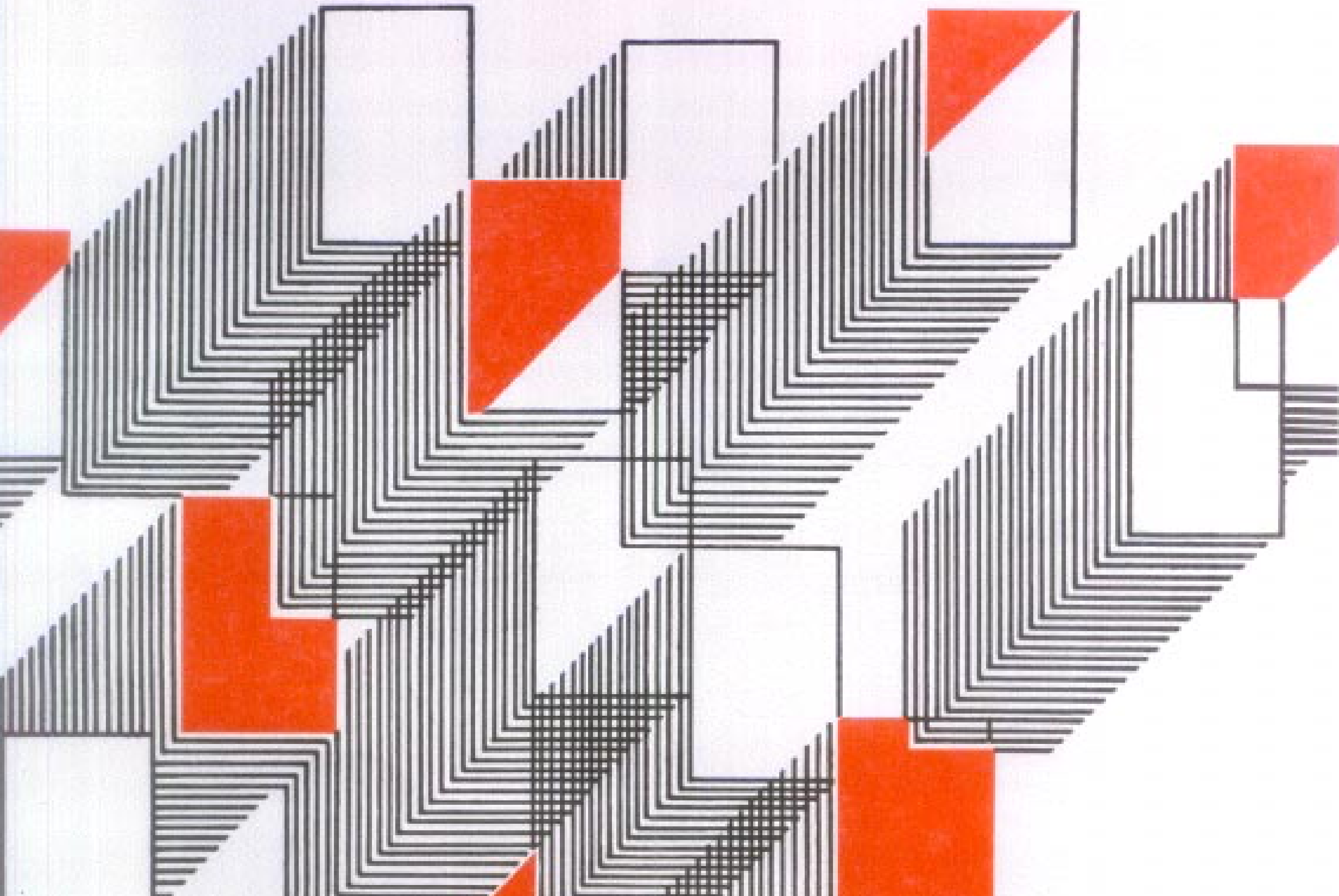
SHEAR AND TORSION



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4. SHEAR AND TORSION

4.1 DESIGN SHEAR STRENGTH OF CONCRETE

The design shear strength of concrete is given in Table 13 of the Code. The values given in the Code are based on the following equation:

$$\tau_c = \frac{0{\cdot}85\ \sqrt{0{\cdot}8 f_{ck}}\ (\sqrt{1+5\beta}-1)}{6\beta}$$

where

$\beta = 0{\cdot}8\ f_{ck}/6{\cdot}89\, p_t$, but not less than 1·0, and $p_t = 100\ A_{st}/b_w d$.

The value of τ_c corresponding to p_t varying from 0·20 to 3·00 at intervals of 0·10 are given in Table 61 for different grades of concrete.

4.2 NOMINAL SHEAR STRESS

The nominal shear stress τ_v is calculated by the following equation:

$$\tau_v = \frac{V_u}{bd}$$

where

V_u is the shear force.

When τ_v exceeds τ_c, shear reinforcement should be provided for carrying a shear equal to $V_u - \tau_c\, bd$. The shear stress τ_v should not in any case exceed the values of $\tau_{c,max}$, given in Table J. (If $\tau_v > \tau_{c,max}$, the section is to be redesigned.)

TABLE J MAXIMUM SHEAR STRESS $\tau_{c,max}$

CONCRETE GRADE	M15	M20	M25	M30	M35	M40
$\tau_{c,max}$, N/mm²	2·5	2·8	3·1	3·5	3·7	4·0

4.3 SHEAR REINFORCEMENT

The design shear strength of vertical stirrups is given by the following equation:

$$V_{us} = \frac{0{\cdot}87 f_y A_{sv} d}{s_v}$$

where

A_{sv} is the total cross sectional area of the vertical legs of the stirrups, and

s_v is the spacing (pitch) of the stirrups.

The shear strength expressed as V_{us}/d are given in Table 62 for different diameters and spacings of stirrups, for two grades of steel. For a series of inclined stirrups, the value of V_{us}/d for vertical stirrups should be multiplied by ($\sin\alpha + \cos\alpha$) where α is the angle between the inclined stirrups and the axis of the member. The multiplying factor works out to 1·41 and 1·37 for 45° and 60° angles respectively.

For a bent up bar,

$V_{us} = 0{\cdot}87\, f_y\, A_{sv} \sin\alpha$

Values of V_{us} for different sizes of bars, bent up at 45° and 60° to the axis of the member are given in Table 63 for two grades of steel.

4.4 TORSION

Separate Charts or Tables are not given for torsion. The method of design for torsion is based on the calculation of an equivalent shear force and an equivalent bending moment. After determining these, some of the Charts and Tables for shear and flexure can be used. The method of design for torsion is illustrated in Example 11.

Example 10 Shear

Determine the shear reinforcement (vertical stirrups) required for a beam section with the following data:

Beam size	30 × 60 cm
Depth of beam	60 cm
Concrete grade	M 15
Characteristic strength of stirrup reinforcement	250 N/mm²
Tensile reinforcement percentage	0·8
Factored shear force, V_u	180 kN

Assuming 25 mm dia bars with 25 mm cover,

$$d = 60 - \frac{2{\cdot}5}{2} - 2.5 = 56{\cdot}25 \text{ cm}$$

$$\text{Shear stress, } \tau_v = \frac{V_u}{bd} = \frac{180\times10^3}{30\times56{\cdot}25\times10^2} = 1{\cdot}07 \text{ N/mm}^2$$

From *Table J* for M15, $\tau_{c,max} = 2{\cdot}5$ N/mm²

τ_v is less than $\tau_{c,max}$

From *Table 61*, for $P_t = 0{\cdot}8$, $\tau_c = 0{\cdot}55$ N/mm²

Shear capacity of concrete section $= \tau_c\, bd$
$= 0{\cdot}55\times30\times56{\cdot}25\times10^2/10^3 = 92{\cdot}8$ kN

Shear to be carried by stirrups, $V_{us} = V_s - \tau_c bd$
$= 180 - 92{\cdot}8 = 87{\cdot}2$ kN

$$\frac{V_{us}}{d} = \frac{87{\cdot}2}{56{\cdot}25} = 1{\cdot}55 \text{ kN/cm}$$

Referring to *Table 62*, for steel $f_y = 250$ N/mm². Provide 8 mm diameter two legged vertical stirrups at 14 cm spacing.

Example 11 Torsion

Determine the reinforcements required for a rectangular beam section with the following data:

Size of the beam	30 × 60 cm
Concrete grade	M 15
Characteristic strength of steel	415 N/mm²
Factored shear force	95 kN
Factored torsional moment	45 kN.m
Factored bending moment	115 kN.m

Assuming 25 mm dia bars with 25 mm cover,

$$d = 60 - 2{\cdot}5 - \frac{2{\cdot}5}{2} = 56{\cdot}25 \text{ cm}$$

Equivalent shear,

$$V_e = V + 1{\cdot}6\left(\frac{T}{b}\right)$$

$$= 95 + 1{\cdot}6 \times \frac{45}{0{\cdot}3} = 95 + 240 = 335 \text{ kN}$$

Equivalent shear stress.

$$\tau_{ve} = \frac{V_e}{bd} = \frac{335 \times 10^3}{30 \times 56{\cdot}25 \times 10^2} = 1{\cdot}99 \text{ N/mm}^2$$

From *Table J*, for M15, $\tau_{c,max} = 2{\cdot}5$ N/mm²

τ_{ve} is less than $\tau_{c,max}$; hence the section does not require revision.

From *Table 61*, for an assumed value of $p_t = 0{\cdot}5$,

$\tau_c = 0{\cdot}46$ N/mm² $< \tau_{ve}$.

Hence longitudinal and transverse reinforcements are to be designed Longitudinal reinforcement (*see 40.4.2* of the Code): Equivalent bending moment,

$$M_{e1} = M_u + M_t$$

$$= M_u + \frac{T_u(1 + D/b)}{1{\cdot}7}$$

$$= 115 + 45\left(1 + \frac{60}{30}\right)/1{\cdot}7$$

$$= 115 + 79{\cdot}4$$

$$= 194{\cdot}4 \text{ kN.m}$$

$$M_{e1}/bd^2 = \frac{194{\cdot}4 \times 10^5}{30 \times (56{\cdot}25)^2 \times 10^3} = 2{\cdot}05 \text{ N/mm}^2$$

Referring to *Table 1*, corresponding to $M_u/bd^2 = 2{\cdot}05$

$$p_t = 0{\cdot}708$$

$$A_{st} = 0{\cdot}708 \times 30 \times 56{\cdot}25/100 = 11{\cdot}95 \text{ cm}^2$$

Provide 4 bars of 20 mm dia ($A_{st} = 12{\cdot}56$ cm²) on the flexural tensile face. As M_t is less than M_u, we need not consider M_{e2} according to *40.4.2.1* of the Code. Therefore, provide only two bars of 12 mm dia on the compression face, one bar being at each corner.

As the depth of the beam is more than 45 cm, side face reinforcement of 0·05 percent on each side is to be provided (*see 25.5.1.7* and *25.5.1.3* of the Code). Providing one bar at the middle of each side,

Spacing of bar $= 53{\cdot}4/2 = 26{\cdot}7$ cm

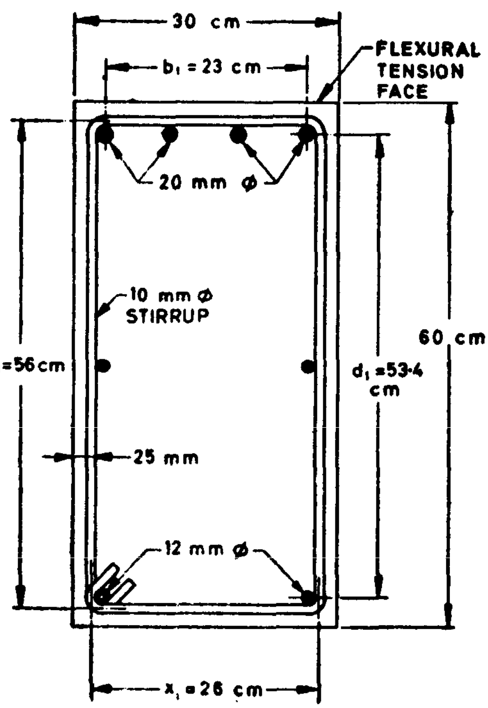
$$\text{Area required for each bar} = \frac{0{\cdot}05 \times 30 \times 26{\cdot}7}{100}$$

$$= 0{\cdot}40 \text{ cm}^2$$

Provide one bar of 12 mm dia on each side. Transverse reinforcement (*see 40.4.3* of the Code):

Area of two legs of the stirrup should satisfy the following:

$$A_{su} = \frac{T_u S_v}{b_1 d_1 (0{\cdot}87 f_y)} + \frac{V_u \, S_v}{2{\cdot}5\, d_1 (0{\cdot}87 f_y)}$$



Assuming diameter of stirrups as 10 mm

$d_1 = 60 - (2{\cdot}5 + 1{\cdot}0) - (2{\cdot}5 + 0{\cdot}6) = 53{\cdot}4$ cm

$b_1 = 30 - 2\,(2{\cdot}5 + 1{\cdot}0) = 23$ cm

$$\frac{A_{sv}\,(0{\cdot}87\,f_y)}{S_v} = \frac{45 \times 10^6}{23 \times 53{\cdot}4 \times 10^2} + \frac{95 \times 10^3}{2{\cdot}5 \times 53{\cdot}4 \times 10} = 366{\cdot}4 + 71{\cdot}2$$

$$= 437{\cdot}6 \text{ N/mm}$$

$$= 4{\cdot}38 \text{ kN/cm}$$

Area of all the legs of the stirrup should satisfy the condition that A_{sv}/S_v should not be less than $\dfrac{(\tau_{ve} - \tau_c)b}{0{\cdot}87\,f_y}$

From *Table 61*, for tensile reinforcement percentage of 0·71, the value of τ_c is 0·53 N/mm²

$$\frac{A_{sv}\,(0{\cdot}87\,f_y)}{S_v} = (\tau_{ve} - \tau_c)b$$

$$= (1{\cdot}99 - 0{\cdot}53)$$

$$30 \times 10$$

$$= 438 \text{ N/mm} = 4{\cdot}38 \text{ kN/cm}$$

NOTE—It is only a coincidence that the values of $A_{sv}\,(0{\cdot}87\,f_y)/S_v$ calculated by the two equations are the same.

Referring *Table 62* (for $f_y = 415$ N/mm²).

Provide 10 mm ϕ two legged stirrups at 12·5 cm spacing.

According to *25.5.1.7(a)* of the Code, the spacing of stirrups shall not exceed x_1, $(x_1 + y_1)/4$ and 300 mm, where x_1 and y_1 are the short and long dimensions of the stirrup.

$$x_1 = 30 - 2(2{\cdot}5 - 0{\cdot}5) = 26 \text{ cm}$$

$$y_1 = 60 - 2(2{\cdot}5 - 0{\cdot}5) = 56 \text{ cm}$$

$$(x_1 + y_1)/4 = (26 + 56)/4 = 20{\cdot}5 \text{ cm}$$

10 mm ϕ two legged stirrups at 12·5 cm spacing will satisfy all the codal requirements.

TABLE 61 SHEAR — DESIGN SHEAR STRENGTH OF CONCRETE, τ_c' N/mm²

p_t	f_{ck}, N/mm²					
	15	20	25	30	35	40
0·20	0·32	0·33	0·33	0·33	0·34	0·34
0·30	0·38	0·39	0·39	0·40	0·40	0·41
0·40	0·43	0·44	0·45	0·45	0·46	0·46
0·50	0·46	0·48	0·49	0·50	0·50	0·51
0·60	0·50	0·51	0·53	0·54	0·54	0·55
0·70	0·53	0·55	0·56	0·57	0·58	0·59
0·80	0·55	0·57	0·59	0·60	0·61	0·62
0·90	0·57	0·60	0·62	0·63	0·64	0·65
1·00	0·60	0·62	0·64	0·66	0·67	0·68
1·10	0·62	0·64	0·66	0·68	0·69	0·70
1·20	0·63	0·66	0·69	0·70	0·72	0·73
1·30	0·65	0·68	0·71	0·72	0·74	0·75
1·40	0·67	0·70	0·72	0·74	0·76	0·77
1·50	0·68	0·72	0·74	0·76	0·78	0·79
1·60	0·69	0·73	0·76	0·78	0·80	0·81
1·70	0·71	0·75	0·77	0·80	0·81	0·83
1·80	0·71	0·76	0·79	0·81	0·83	0·85
1·90	0·71	0·77	0·80	0·83	0·85	0·86
2·00	0·71	0·79	0·82	0·84	0·86	0·88
2·10	0·71	0·80	0·83	0·86	0·88	0·90
2·20	0·71	0·81	0·84	0·87	0·89	0·91
2·30	0·71	0·82	0·86	0·88	0·91	0·93
2·40	0·71	0·82	0·87	0·90	0·92	0·94
2·50	0·71	0·82	0·88	0·91	0·93	0·95
2·60	0·71	0·82	0·89	0·92	0·94	0·97
2·70	0·71	0·82	0·90	0·93	0·96	0·98
2·80	0·71	0·82	0·91	0·94	0·97	0·99
2·90	0·71	0·82	0·92	0·95	0·98	1·00
3·00	0·71	0·82	0·92	0·96	0·99	1·01

TABLE 62 SHEAR — VERTICAL STIRRUPS

Values of V_{us}/d for two legged stirrups, kN/cm.

Stirrup Spacing, cm	f_y = 250 N/mm² Diameter, mm				f_y = 415 N/mm² Diameter, mm			
	6	8	10	12	6	8	10	12
5	2·460	4·373	6·833	9·839	4·083	7·259	11·342	16·334
6	2·050	3·644	5·694	8·200	3·403	6·049	9·452	13·611
7	1·757	3·124	4·881	7·028	2·917	5·185	8·102	11·667
8	1·537	2·733	4·271	6·150	2·552	4·537	7·089	10·208
9	1·367	2·429	3·796	5·466	2·269	4·033	6·302	9·074
10	1·230	2·186	3·416	4·920	2·042	3·630	5·671	8·167
11	1·118	1·988	3·106	4·472	1·856	3·299	5·156	7·424
12	1·025	1·822	2·847	4·100	1·701	3·025	4·726	6·806
13	0·946	1·682	2·628	3·784	1·571	2·792	4·363	6·286
14	0·879	1·562	2·440	3·514	1·458	2·593	4·051	5·833
15	0·820	1·458	2·278	3·280	1·361	2·420	3·781	5·445
16	0·769	1·366	2·135	3·075	1·276	2·269	3·545	5·104
17	0·723	1·286	2·010	2·894	1·201	2·135	3·336	4·804
18	0·683	1·215	1·898	2·733	1·134	2·016	3·151	4·537
19	0·647	1·151	1·798	2·589	1·075	1·910	2·985	4·298
20	0·615	1·093	1·708	2·460	1·020	1·815	2·836	4·083
25	0·492	0·875	1·367	1·968	0·817	1·452	2·269	3·267
30	0·410	0·729	1·139	1·640	0·681	1·210	1·890	2·722
35	0·351	0·625	0·976	1·406	0·583	1·037	1·620	2·333
40	0·307	0·547	0·854	1·230	0·510	0·907	1·418	2·042
45	0·273	0·486	0·759	1·093	0·454	0·807	1·260	1·815

TABLE 63 SHEAR — BENT-UP BARS

Values of V_{us} for singal bar, kN

Bar Diameter, mm	f_y = 250 N/mm²		f_y = 415 N/mm²	
	$\alpha = 45°$	$\alpha = 60°$	$\alpha = 45°$	$\alpha = 60°$
10	12·08	14·79	20·05	24·56
12	17·39	21·30	28·87	35·36
16	30·92	37·87	51·33	62·87
18	39·14	47·93	64·97	79·57
20	48·32	59·18	80·21	98·23
22	58·46	71·60	97·05	118.86
25	75·49	92·46	125·32	153.48
28	94·70	115·98	157·20	192·53
32	123·69	151·49	205·32	251·47
36	156·54	191·73	259·86	318·27

NOTE — α is the angle between the bent-up bar and the axis of the member.

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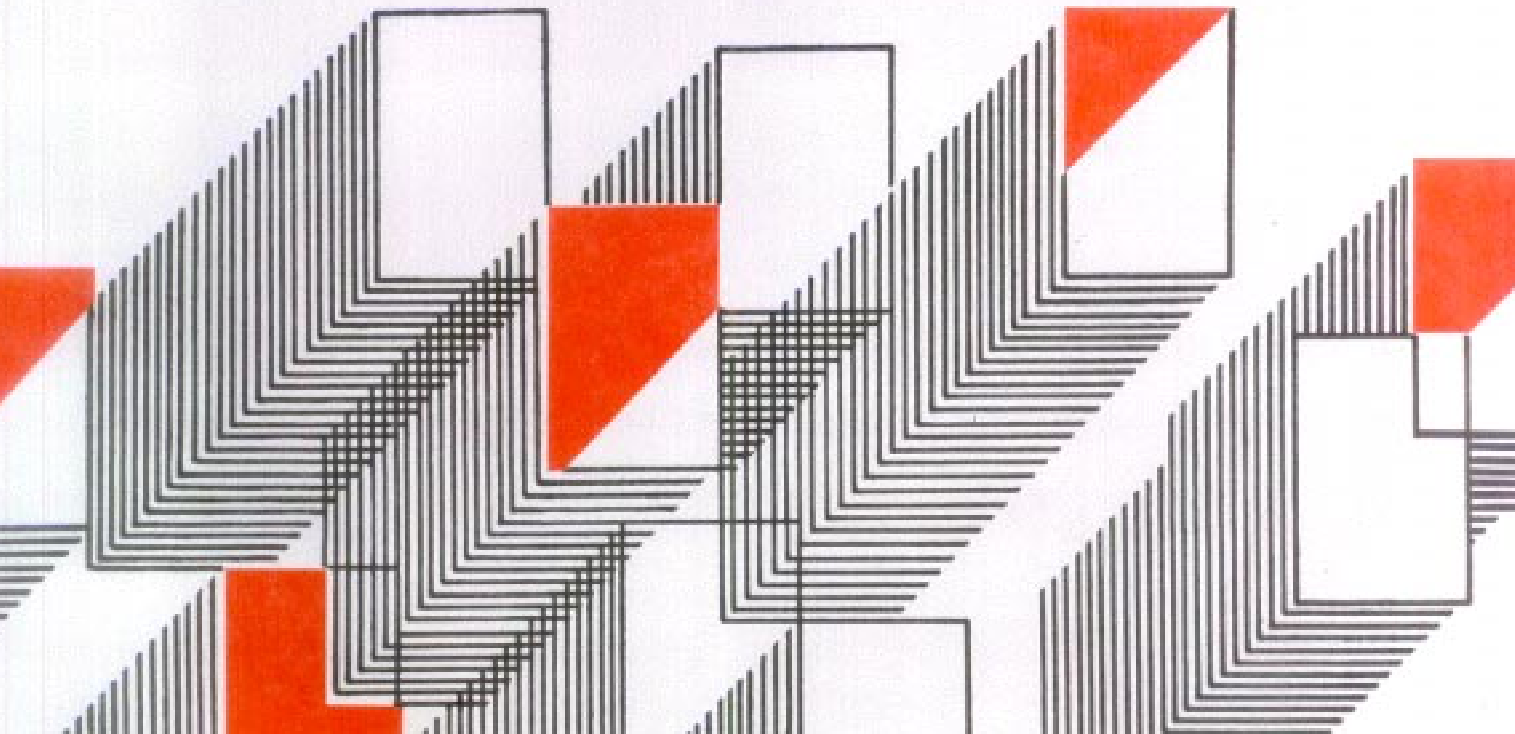
DEVELOPMENT LENGTH AND ANCHORAGE



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5. DEVELOPMENT LENGTH AND ANCHORAGE

5.1 DEVELOPMENT LENGTH OF BARS

The development length L_d, is given by

$$L_d = \frac{\phi \, \sigma_s}{4 \, \tau_{bd}}$$

where

ϕ is the diameter of the bar,

σ_s is the stress in the bar, and

τ_{bd} is the design bond stress given in *25.2.1.1* of the Code.

The value of the development length corresponding to a stress of 0·87 f_y in the reinforcement, is required for determining the maximum permissible bar diameter for positive moment reinforcement [*see 25.2.3.3*(*c*) of the Code] and for determining the length of lap splices (*see 25.2.5.1* of the Code). Values of this development length for different grades of steel and concrete are given in Tables 64 to 66. The tables contain the development length values for bars in tension as well as compression.

5.2 ANCHORAGE VALUE OF HOOKS AND BENDS

In the case of bars in tension, a standard hook has an anchorage value equivalent to a straight length of 16ϕ and a 90° bend has an anchorage value of 8ϕ. The anchorage values of standard hooks and bends for different bar diameters are given in *Table 67.*

f_y
250
415
f_{ck}
15
20
25
30

TABLE 64 DEVELOPMENT LENGTH FOR FULLY STRESSED PLAIN BARS

f_y = 250 N/mm² for bars up to 20 mm diameter.
= 240 N/mm² for bars over 20 mm diameter.

Tabulated values are in centimetres.

Bar Diameter, mm	Tension Bars Grade of Concrete				Compression Bars Grade of Concrete			
	M15	M20	M25	M30	M15	M20	M25	M30
6	32·6	27·2	23·3	21·8	26·1	21·8	18·6	17·4
8	43·5	36·3	31·1	29·0	34·8	29·0	24·9	23·2
10	54·4	45·3	38·8	36·3	43·5	36·3	31·1	29·0
12	65·3	54·4	46·6	43·5	52·2	43·5	37·3	34·8
16	87·0	72·5	62·1	58·0	69·6	58·0	49·7	46·4
18	97·9	81·6	69·9	65·3	78·3	65·3	55·9	52·2
20	108·8	90·6	77·7	72·5	87·0	72·5	62·1	58·0
22	114·8	95·7	82·0	76·6	91·9	76·6	65·6	61·2
25	130·5	108·8	93·2	87·0	104·4	87·0	74·6	69·6
28	146·2	121·8	104·4	97·4	116·9	97·4	83·5	78·0
32	167·0	139·2	119·3	111·4	133·6	111·4	95·5	89·1
36	187·9	156·6	134·2	125·3	150·3	125·3	107·4	100·2

NOTE — The development lengths given above are for a stress of 0·87 f_y in the bar.

TABLE 65 DEVELOPMENT LENGTH FOR FULLY STRESSED DEFORMED BARS

f_y = 415 N/mm²

Tabulated values are in centimetres.

Bar Diameter, mm	Tension Bars Grade of Concrete				Compression Bars Grade of Concrete			
	M15	M20	M25	M30	M15	M20	M25	M30
6	33·8	28·2	24·2	22·6	27·1	22·6	19·3	18·1
8	45·1	37·6	32·2	30·1	36·1	30·1	25·8	24·1
10	56·4	47·0	40·3	37·6	45·1	37·6	32·2	30·1
12	67·7	56·4	48·4	45·1	54·2	45·1	38·7	36·1
16	90·3	75·2	64·5	60·2	72·2	60·2	51·6	48·1
18	101·5	84·6	72·5	67·7	81·2	67·7	58·0	54·2
20	112·8	94·0	80·6	75·2	90·3	75·2	64·5	60·2
22	124·1	103·4	88·7	82·7	99·3	82·7	70·9	66·2
25	141·0	117·5	100·7	94·0	112·8	94·0	80·6	75·2
28	158·0	131·6	112·8	105·3	126·4	105·3	90·3	84·2
32	180·5	150·4	128·9	120·3	144·4	120·3	103·2	96·3
36	203·1	169·3	145·0	135·4	162·5	135·4	116·1	108·3

NOTE—The development lengths given above are for a stress of 0·87 f_y in the bars.

TABLE 66 DEVELOPMENT LENGTH FOR FULLY STRESSED DEFORMED BARS

f_y = 500 N/mm²

Tabulated values are in centimetres.

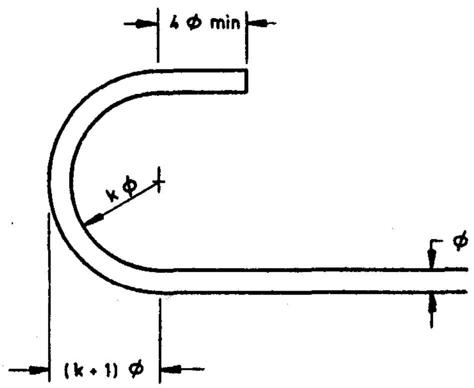
Bar Diameter, mm	Tension Bars — Grade of Concrete				Compression Bars — Grade of Concrete			
	M15	M20	M25	M30	M15	M20	M25	M30
6	40·8	34·0	29·1	27·2	32·6	27·2	23·3	21·8
8	54·4	45·3	38·8	36·3	43·5	36·3	31·1	29·0
10	68·0	56·6	48·5	45·3	54·4	45·3	38·8	36·3
12	81·6	68·0	58·3	54·4	65·3	54·4	46·6	43·5
16	108·8	90·6	77·7	72·5	87·0	72·5	62·1	58·0
18	122·3	102·0	87·4	81·6	97·9	81·6	69·9	65·3
20	135·9	113·3	97·1	90·6	108·8	90·6	77·7	72·5
22	149·5	124·6	106·8	99·7	119·6	99·7	85·4	79·8
25	169·9	141·6	121·4	113·3	135·9	113·3	97·1	90·6
28	190·3	158·6	135·9	126·9	152·3	126·9	108·8	101·5
32	217·5	181·3	155·4	145·0	174·0	145·0	124·3	116·0
36	244·7	203·9	174·8	163·1	195·8	163·1	139·8	130·5

Note — The development lengths given above are for a stress of 0·87 f_y in the bar.

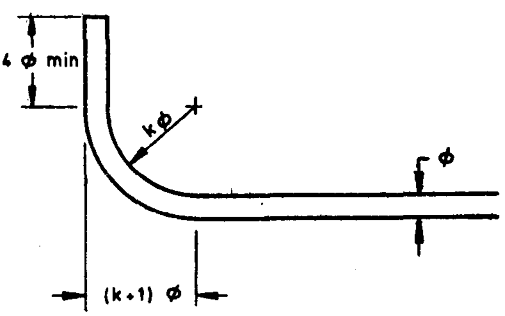
TABLE 67 ANCHORAGE VALUE OF HOOKS AND BENDS

Tabulated values are in centimetres.

BAR DIAMETER, mm	6	8	10	12	16	18	20	22	25	28	32	36
Anchorage Value of hook	9·6	12·8	16·0	19·2	25·6	28·8	32·0	35·2	40·0	44·8	51·2	57·6
Anchorage Value of 90° bend	4·8	6·4	8·0	9·6	12·8	14·4	16·0	17·6	20·0	22·4	25·6	28·8



STANDARD HOOK



STANDARD 90° BEND

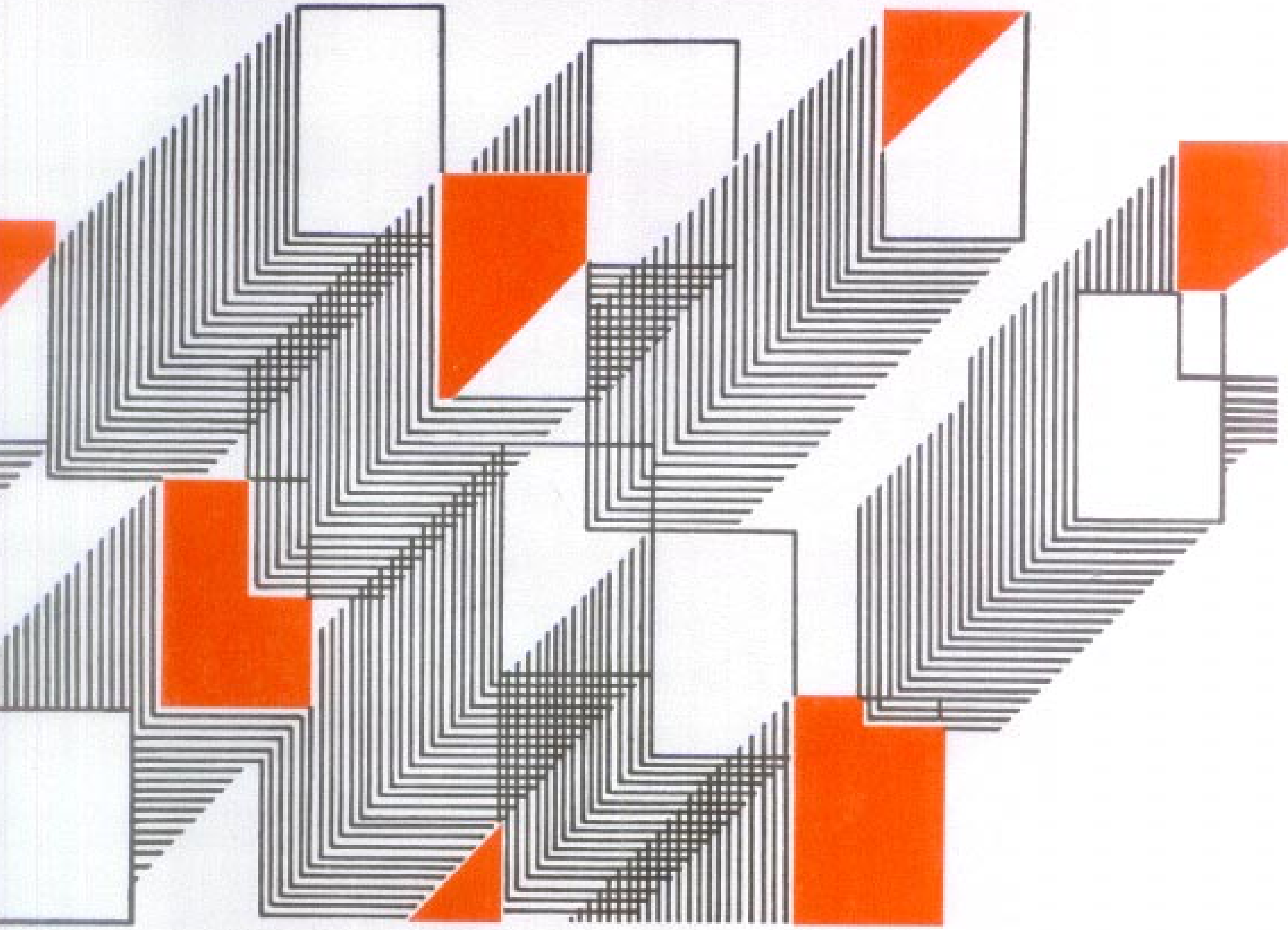
STANDARD HOOK AND BEND

Type of Steel	*Min Value of k*
Mild steel	2
Cold worked steel	4

NOTE 1 — Table is applicable to all grades of reinforcement bars.

NOTE 2 — Hooks and bends shall conform to the details given above.

WORKING STRESS DESIGN



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6. WORKING STRESS DESIGN

6.1 FLEXURAL MEMBERS

Design of flexural members by working stress method is based on the well known assumptions given in *43.3* of the Code. The value of the modular ratio, m is given by

$$m = \frac{280}{3\ \sigma_{cbc}} = \frac{93\cdot33}{\sigma_{cbc}}$$

Therefore, for all values of σ_{cbc} we have

$$m\ \sigma_{cbc} = 93\cdot33$$

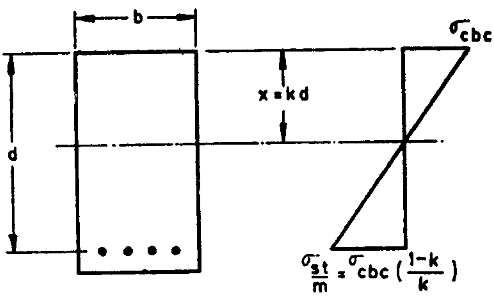

FIG. 9 BALANCED SECTION (WORKING STRESS DESIGN)

6.1.1 Balanced Section (see Fig. 9)

Stress in steel $= \sigma_{st} = m\sigma_{cbc}\left(\frac{1}{k} - 1\right)$

$$\left(\frac{1}{k} - 1\right) = \frac{\sigma_{st}}{m\sigma_{cbc}} = \frac{\sigma_{st}}{93\cdot33}$$

$$\frac{1}{k} = \frac{\sigma_{st}}{93\cdot33} + 1 = \frac{\sigma_{st} + 93\cdot33}{93\cdot33}$$

$$k = \frac{93\cdot33}{\sigma_{st} + 93\cdot33}$$

The value of k for balanced section depends only on σ_{st}. It is independent of σ_{cbc}. Moment of resistance of a balanced section is given by $M_{bal} = \frac{bd^2}{2}\sigma_{cbc}\,k\left(1 - \frac{k}{3}\right)$. The values of M_{bal}/bd^2 for different values of σ_{cbc} and σ_{st} are given in Table K.

TABLE K MOMENT OF RESISTANCE FACTOR M/bd^2, N/mm² FOR BALANCED RECTANGULAR SECTION

σ_{cbc} N/mm²	σ_{st}, N/mm² 140	230	275
5·0	0·87	0·65	0·58
7·0	1·21	0·91	0·81
8·5	1·47	1·11	0·99
10·0	1·73	1·30	1·16

Reinforcement percentage $p_{t,bal}$ for balanced section is determined by equating the compressive force and tensile force.

$$\frac{\sigma_{cbc}\,kdb}{2} = \frac{p_{t,bal}\,bd\ \sigma_{st}}{100}$$

$$p_{t,bal} = \frac{50\,k\cdot\sigma_{cbc}}{\sigma_{st}}$$

The value of $p_{t,bal}$ for different values of σ_{cbc} and σ_{st} are given in Table L.

TABLE L PERCENTAGE OF TENSILE REINFORCEMENT $p_{t,bal}$ FOR SINGLY REINFORCED BALANCED SECTION
(*Clause 6.1.1*)

σ_{cbc} N/mm²	σ_{st} N/mm² 140	230	275
5·0	0·71	0·31	0·23
7·0	1·00	0·44	0·32
8·5	1·21	0·53	0·39
10·0	1·43	0·63	0·46

6.1.2 Under Reinforced Section

The position of the neutral axis is found by equating the moments of the equivalent areas.

$$bkd\frac{kd}{2} = \frac{p_t\,bd}{100}\,m\,(d - kd)$$

$$bd^2\frac{k^2}{2} = bd^2\ \frac{p_t m}{100}(1 - k)$$

$$k^2 = \frac{p_t m}{50}(1 - k)$$

$$k^2 + \frac{p_t mk}{50} - \frac{p_t m}{50} = 0.$$

The positive root of this equation is given by

$$k = -\frac{p_t\,m}{100} + \sqrt{\frac{p^2_t\,m^2}{(100)^2} + \frac{p_t\,m}{50}}$$

This is the general expression for the depth of neutral axis of a singly reinforced section. Moment of resistance of an under-reinforced section is given by

$$M = bd^2\,\frac{p_t\,\sigma_{st}}{100}\left(1 - \frac{k}{3}\right)$$

Values of the moment of resistance factor M/bd^2 have been tabulated against p_t in

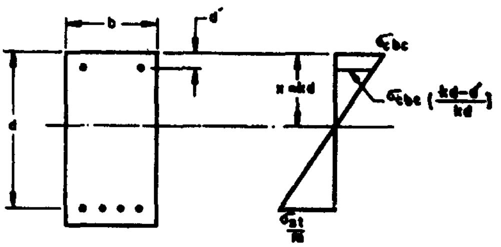

FIG. 10 DOUBLY REINFORCED SECTION (WORKING STRESS DESIGN)

Tables 68 to 71. The Tables cover four grades of concrete and five values of σ_{st}.

6.1.3 Doubly Reinforced Section — Doubly reinforced sections are adopted when the bending moment exceeds the moment of resistance of a balanced section.

$$M = M_{bal} + M'$$

The additional moment M' is resisted by providing compression reinforcement and additional tensile reinforcement. The stress in the compression reinforcement is taken as 1·5 m times the stress in the surrounding concrete.

Taking moment about the centroid of tensile reinforcement,

$$M' = \frac{p_c\, bd}{100}(1{\cdot}5\,m - 1)\,\sigma_{cbc} \times \left(\frac{kd - d'}{kd}\right)(d - d')$$

$$= \frac{p_c}{100}(1{\cdot}5\,m - 1)\,\sigma_{cbc} \times \left(1 - \frac{d'}{kd}\right)\left(1 - \frac{d'}{d}\right) bd^2$$

Equating the additional tensile force and additional compressive force,

$$bd\,\frac{(p_t - p_{t,bal})}{100}\,\sigma_{st} = \frac{p_c\, bd}{100}(1{\cdot}5\,m - 1)\sigma_{cbc}\left(1 - \frac{d'}{kd}\right)$$

or $(p_t - p_{t,bal})\,\sigma_{st}$

$$= p_c\,(1{\cdot}5\,m - 1)\,\sigma_{cbc}\left(1 - \frac{d'}{kd}\right)$$

$$\therefore\quad M = M_{bal} + \frac{(p_t - p_{t,bal})}{100}\,\sigma_{st} \times \left(1 - \frac{d'}{d}\right) bd^2$$

Total tensile reinforcement A_{st} is given by

$$A_{st} = A_{st1} + A_{st2}$$

where $A_{ts1} = p_{t,bal}\,\dfrac{bd}{100}$

and $A_{st2} = \dfrac{M'}{\sigma_{st}\,(d - d')}$

The compression reinforcement can be expressed as a ratio of the additional tensile reinforcement area A_{st2}.

$$\frac{A_{sc}}{A_{st2}} = \frac{p_c}{(p_t - p_{t,bal})}$$

$$= \frac{\sigma_{st}}{\sigma_{cbc}}\,\frac{1}{(1{\cdot}5\,m - 1)\,(1 - d'/kd)}$$

Values of this ratio have been tabulated for different values of d'/d and σ_{cbc} in Table M. The table includes two values of σ_{st}. The values of p_t and p_c for four values of d'/d have been tabulated against M/bd^2 in *Tables 72 to 79*. Tables are given for four grades of concrete and two grades of steel.

TABLE M VALUES OF THE RATIO A_{sc}/A_{st2}
(*Clause 6.1.3*)

σ_{st} N/mm²	σ_{cbc} N/mm²	d'/d 0·05	0·10	0·15	0·20
140	5·0	1·19	1·38	1·66	2·07
	7·0	1·20	1·40	1·68	2·11
	8·5	1·22	1·42	1·70	2·13
	10·0	1·23	1·44	1·72	2·15
230	5·0	2·06	2·61	3·55	5·54
	7·0	2·09	2·65	3·60	5·63
	8·5	2·12	2·68	3·64	5·69
	10·0	2·14	2·71	3·68	5·76

6.2 COMPRESSION MEMBERS

Charts 86 and 87 are given for determining the permissible axial load on a pedestal or short column reinforced with longitudinal bars and lateral ties. Charts are given for two values of σ_{sc}. These charts have been made in accordance with *45.1* of the Code.

According to *46.3* of the Code, members subject to combined axial load and bending designed by methods based on elastic theory should be further checked for their strength under ultimate load conditions. Therefore it would be advisable to design such members directly by the limit state method. Hence, no design aids are given for designing such members by elastic theory.

6.3 SHEAR AND TORSION

The method of design for shear and torsion by working stress method are similar to the limit state method. The values of permissible shear stress in concrete are given in *Table 80*. *Tables 81 and 82* are given for design of shear reinforcement.

6.4 DEVELOPMENT LENGTH AND ANCHORAGE

The method of calculating development length is the same as given under limit state design. The difference is only in the values of bond stresses. Development lengths for plain bars and two grades of deformed bars are given in *Tables 83 to 85*.

Anchorage value of standard hooks and bends as given in *Table 67* are applicable to working stress method also.

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Chart 86 AXIAL COMPRESSION (Working Stress Design)

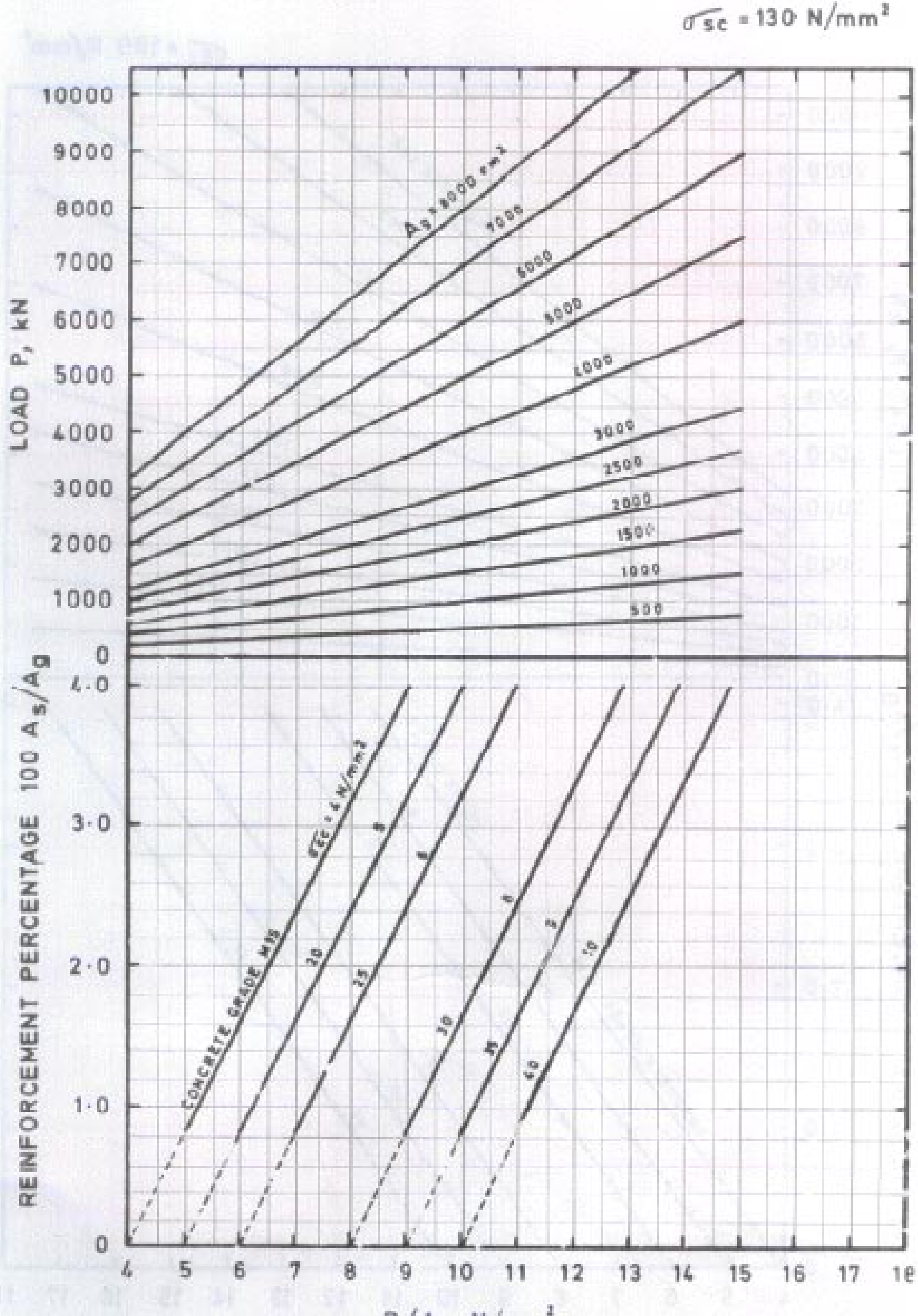

Chart 87 AXIAL COMPRESSION (Working Stress Design)

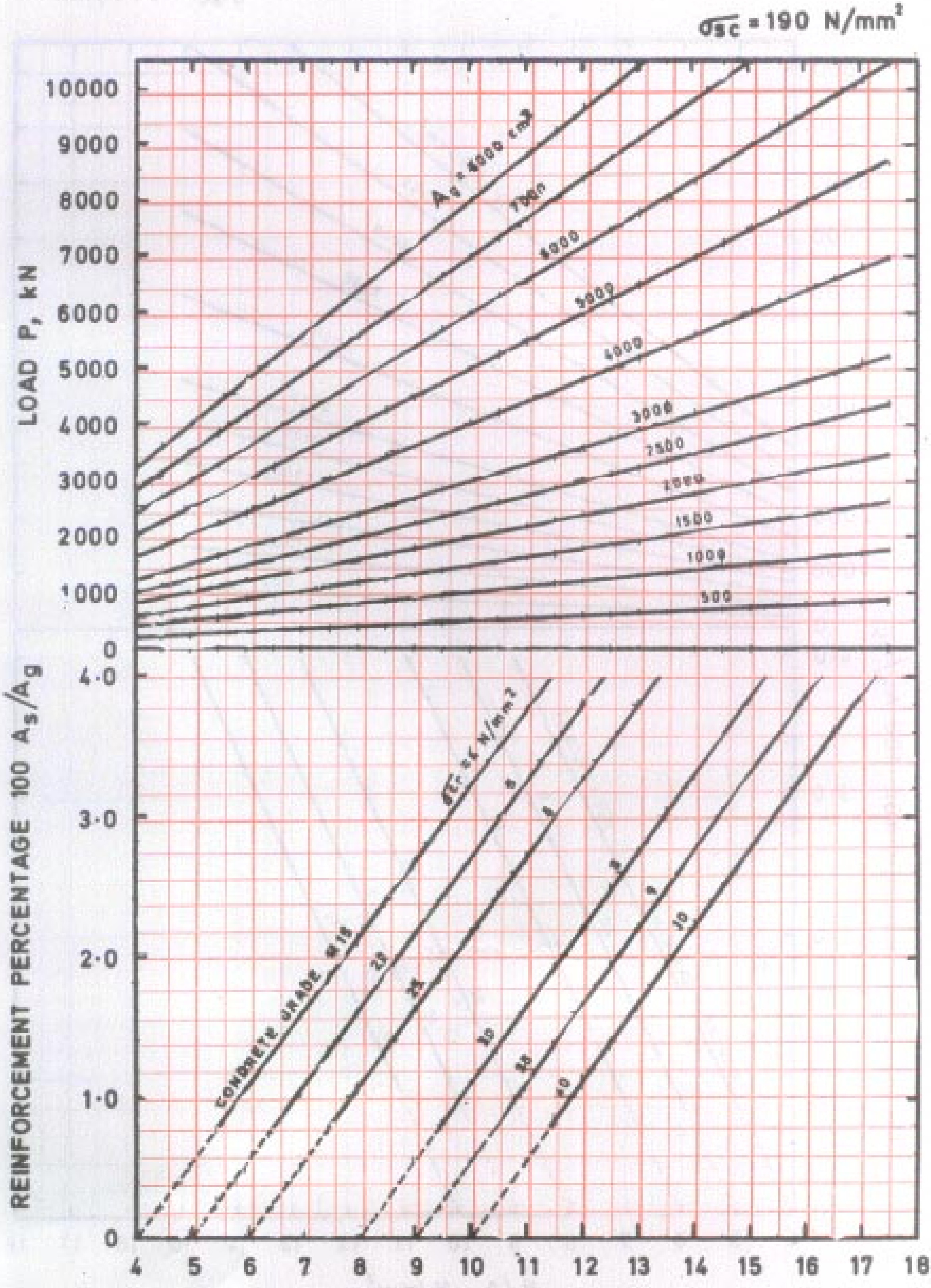

TABLE 68 FLEXURE — MOMENT OF RESISTANCE FACTOR, M/bd^2, N/mm^2 FOR SINGLY REINFORCED SECTIONS

σ_{cbc}=5·0 N/mm^2

p_t	σ_{st}, N/mm^2					p_t	σ_{st}, N/mm^2				
	130	140	190	230	275		130	140	190	230	275
0·12	0·146	0·157	0·214	0·258	0·309	0·47	0·542	0·583			
0·13	0·158	0·170	0·231	0·279	0·334	0·48	0·553	0·595			
0·14	0·170	0·183	0·248	0·300	0·359	0·49	0·564	0·607			
0·15	0·181	0·195	0·265	0·321	0·384	0·50	0·574	0·619			
0·16	0·193	0·208	0·282	0·341	0·408	0·51	0·585	0·630			
0·17	0·205	0·220	0·299	0·362	0·433	0·52	0·596	0·642			
0·18	0·216	0·233	0·316	0·383	0·457	0·53	0·607	0·654			
0·19	0·228	0·245	0·333	0·403	0·482	0·54	0·618	0·665			
0·20	0·239	0·258	0·350	0·423	0·506	0·55	0·629	0·677			
0·21	0·251	0·270	0·367	0·444	0·531	0·56	0·640	0·689			
0·22	0·262	0·282	0·383	0·464	0·555	0·57	0·650	0·700			
0·23	0·274	0·295	0·400	0·484	0·579	0·58	0·661	0·712			
0·24	0·285	0·307	0·417	0·505		0·59	0·672	0·724			
0·25	0·297	0·319	0·433	0·525		0·60	0·683	0·735			
0·26	0·308	0·332	0·450	0·545		0·61	0·693	0·747			
0·27	0·319	0·344	0·467	0·565		0·62	0·704	0·758			
0·28	0·331	0·356	0·483	0·585		0·63	0·715	0·770			
0·29	0·342	0·368	0·500	0·605		0·64	0·726	0·781			
0·30	0·353	0·380	0·516	0·625		0·65	0·736	0·793			
0·31	0·364	0·392	0·533	0·645		0·66	0·747	0·804			
0·32	0·376	0·405	0·549			0·67	0·758	0·816			
0·33	0·387	0·417	0·565			0·68	0·768	0·827			
0·34	0·398	0·429	0·582			0·69	0·779	0·839			
0·35	0·409	0·441	0·598			0·70	0·790	0·850			
0·36	0·420	0·453	0·614			0·71	0·800	0·862			
0·37	0·431	0·465	0·631			0·72	0·811				
0·38	0·443	0·477	0·647			0·73	0·821				
0·39	0·454	0·489	0·663			0·74	0·832				
0·40	0·465	0·500	0·679			0·75	0·843				
0·41	0·476	0·512	0·695			0·76	0·853				
0·42	0·487	0·524	0·711			0·77	0·864				
0·43	0·498	0·536	0·728			0·78	0·874				
0·44	0·509	0·548				0·79	0·885				
0·45	0·520	0·560				0·80	0·895				
0·46	0·531	0·572									

WORKING STRESS METHOD

TABLE 69 FLEXURE — MOMENT OF RESISTANCE FACTOR, M/bd^2, N/mm² FOR SINGLY REINFORCED SECTIONS

$\sigma_{cbc} = 7{\cdot}0$ N/mm²

P_t	σ_{st}, N/mm² 130	140	190	230	275	P_t	σ_{st}, N/mm² 130	140	190	230	275
0·20	0·242	0·261	0·354	0·428	0·512	0·76	0·869	0·936			
0·22	0·266	0·286	0·388	0·470	0·562	0·77	0·880	0·948			
0·24	0·289	0·311	0·422	0·511	0·611	0·78	0·891	0·960			
0·26	0·312	0·336	0·456	0·552	0·660	0·79	0·902	0·971			
0·28	0·335	0·361	0·490	0·593	0·709	0·80	0·913	0·983			
0·30	0·358	0·386	0·523	0·633	0·757	0·81	0·923	0·994			
0·32	0·381	0·410	0·557	0·674	0·806	0·82	0·934	1·006			
0·34	0·404	0·435	0·590	0·714		0·83	0·945	1·018			
0·36	0·427	0·459	0·623	0·755		0·84	0·956	1·029			
0·38	0·449	0·484	0·657	0·795		0·85	0·966	1·041			
0·40	0·472	0·508	0·690	0·835		0·86	0·977	1·052			
0·42	0·494	0·532	0·723	0·875		0·87	0·988	1·064			
0·43	0·506	0·545	0·739	0·895		0·88	0·999	1·075			
0·44	0·517	0·557	0·756			0·89	1·009	1·087			
0·45	0·528	0·569	0·772			0·90	1·020	1·099			
0·46	0·539	0·581	0·788			0·91	1·031	1·110			
0·47	0·551	0·593	0·805			0·92	1·041	1·122			
0·48	0·562	0·605	0·821			0·93	1·052	1·133			
0·49	0·573	0·617	0·837			0·94	1·063	1·145			
0·50	0·584	0·629	0·854			0·95	1·073	1·156			
0·51	0·595	0·641	0·870			0·96	1·084	1·168			
0·52	0·606	0·653	0·886			0·97	1·095	1·179			
0·53	0·617	0·665	0·902			0·98	1·105	1·190			
0·54	0·628	0·677	0·919			0·99	1·116	1·202			
0·55	0·640	0·689	0·935			1·00	1·127				
0·56	0·651	0·701	0·951			1·01	1·137				
0·57	0·662	0·713	0·967			1·02	1·148				
0·58	0·673	0·724	0·983			1·03	1·158				
0·59	0·684	0·736	0·999			1·04	1·169				
0·60	0·695	0·748	1·015			1·05	1·180				
0·61	0·706	0·760				1·06	1·190				
0·62	0·717	0·772				1·07	1·201				
0·63	0·728	0·784				1·08	1·211				
0·64	0·739	0·795				1·09	1·222				
0·65	0·750	0·807				1·10	1·232				
0·66	0·761	0·819				1·11	1·243				
0·67	0·772	0·831				1·12	1·254				
0·68	0·782	0·843				1·13	1·264				
0·69	0·793	0·854									
0·70	0·804	0·866									
0·71	0·815	0·878									
0·72	0·826	0·890									
0·73	0·837	0·901									
0·74	0·848	0·913									
0·75	0·859	0·925									

TABLE 70 FLEXURE — MOMENT OF RESISTANCE FACTOR, M/bd^2, N/mm² FOR SINGLY REINFORCED SECTIONS

$\sigma_{cbc} = 8·5$ N/mm²

p_t	σ_{st}, N/mm² 130	140	190	230	275
0·20	0·244	0·262	0·356	0·431	0·515
0·22	0·267	0·288	0·391	0·473	0·565
0·24	0·291	0·313	0·425	0·514	0·615
0·26	0·314	0·338	0·459	0·556	0·664
0·28	0·337	0·363	0·493	0·597	0·714
0·30	0·361	0·388	0·527	0·638	0·763
0·32	0·394	0·413	0·561	0·679	0·812
0·34	0·407	0·438	0·595	0·720	0·861
0·36	0·430	0·463	0·628	0·761	0·909
0·38	0·453	0·488	0·662	0·801	0·958
0·40	0·476	0·512	0·695	0·842	
0·42	0·498	0·537	0·729	0·882	
0·44	0·521	0·561	0·762	0·922	
0·46	0·544	0·586	0·795	0·962	
0·48	0·567	0·610	0·828	1·002	
0·50	0·589	0·634	0·861	1·042	
0·52	0·612	0·659	0·894	1·082	
0·54	0·634	0·683	0·927		
0·56	0·657	0·707	0·960		
0·58	0·679	0·731	0·992		
0·60	0·701	0·755	1·025		
0·62	0·723	0·779	1·057		
0·64	0·746	0·803	1·090		
0·66	0·768	0·827	1·122		
0·68	0·790	0·851	1·155		
0·70	0·812	0·875	1·187		
0·72	0·834	0·898	1·219		
0·74	0·856	0·922			
0·76	0·878	0·946			
0·78	0·900	0·969			
0·80	0·922	0·993			
0·82	0·944	1·016			
0·83	0·955	1·028			
0·84	0·966	1·040			
0·85	0·977	1·052			
0·86	0·987	1·063			
0·87	0·998	1·075			
0·88	1·009	1·087			
0·89	1·020	1·099			
0·90	1·031	1·110			
0·91	1·042	1·122			
0·92	1·053	1·134			
0·93	1·063	1·145			
0·94	1·074	1·157			
0·95	1·085	1·169			

p_t	σ_{st}, N/mm² 130	140	190	230	275
0·96	1·096	1·180			
0·97	1·107	1·192			
0·98	1·117	1·203			
0·99	1·128	1·215			
1·00	1·139	1·227			
1·01	1·150	1·238			
1·02	1·161	1·250			
1·03	1·171	1·261			
1·04	1·182	1·273			
1·05	1·193	1·285			
1·06	1·203	1·296			
1·07	1·214	1·308			
1·08	1·225	1·319			
1·09	1·236	1·331			
1·10	1·246	1·342			
1·11	1·257	1·354			
1·12	1·268	1·365			
1·13	1·278	1·377			
1·14	1·289	1·388			
1·15	1·300	1·400			
1·16	1·310	1·411			
1·17	1·321	1·423			
1·18	1·332	1·434			
1·19	1·342	1·446			
1·20	1·353	1·457			
1·21	1·364	1·468			
1·22	1·374				
1·23	1·385				
1·24	1·395				
1·25	1·406				
1·26	1·417				
1·27	1·427				
1·28	1·438				
1·29	1·448				
1·30	1·459				
1·31	1·469				
1·32	1·480				
1·33	1·491				
1·34	1·501				
1·35	1·512				
1·36	1·522				
1·37	1·533				

TABLE 71 FLEXURE — MOMENT OF RESISTANCE FACTOR, M/bd^2, N/mm² FOR SINGLY REINFORCED SECTIONS

$\sigma_{cbc} = 10\cdot0$ N/mm²

p_t	σ_{st}, N/mm²				
	130	140	190	230	275
0·20	0·245	0·264	0·358	0·433	0·518
0·22	0·269	0·289	0·392	0·475	0·568
0·24	0·292	0·315	0·427	0·517	0·618
0·26	0·316	0·340	0·461	0·559	0·668
0·28	0·339	0·365	0·496	0·600	0·718
0·30	0·363	0·391	0·530	0·642	0·767
0·32	0·386	0·416	0·564	0·683	0·817
0·34	0·409	0·441	0·598	0·724	0·866
0·36	0·432	0·466	0·632	0·765	0·915
0·38	0·456	0·491	0·666	0·806	0·964
0·40	0·479	0·515	0·700	0·847	1·013
0·42	0·502	0·540	0·733	0·888	1·061
0·44	0·525	0·565	0·767	0·928	1·110
0·46	0·548	0·590	0·800	0·969	1·158
0·48	0·570	0·614	0·834	1·009	
0·50	0·593	0·639	0·867	1·049	
0·52	0·616	0·663	0·900	1·090	
0·54	0·639	0·688	0·933	1·130	
0·56	0·661	0·712	0·966	1·170	
0·58	0·684	0·736	0·999	1·210	
0·60	0·706	0·761	1·032	1·250	
0·62	0·729	0·785	1·065	1·289	
0·64	0·751	0·809	1·098		
0·66	0·774	0·833	1·131		
0·68	0·796	0·857	1·163		
0·70	0·818	0·881	1·196		
0·72	0·841	0·905	1·229		
0·74	0·863	0·929	1·261		
0·76	0·885	0·953	1·294		
0·78	0·907	0·977	1·326		
0·80	0·929	1·001	1·358		
0·82	0·952	1·025	1·391		
0·84	0·974	1·048	1·423		
0·86	0·996	1·072	1·455		
0·88	1·018	1·096			
0·90	1·040	1·120			
0·92	1·062	1·143			
0·94	1·083	1·167			
0·96	1·105	1·190			
0·98	1·127	1·214			
1·00	1·149	1·237			
1·02	1·171	1·261			
1·04	1·192	1·284			
1·06	1·214	1·308			
1·08	1·236	1·331			

p_t	σ_{st}, N/mm²				
	130	140	190	230	275
1·10	1·257	1·354			
1·12	1·279	1·377			
1·14	1·301	1·401			
1·16	1·322	1·424			
1·18	1·344	1·447			
1·20	1·365	1·470			
1·22	1·387	1·494			
1·24	1·408	1·517			
1·26	1·430	1·540			
1·28	1·451	1·563			
1·30	1·473	1·586			
1·31	1·483	1·597			
1·32	1·494	1·609			
1·33	1·505	1·620			
1·34	1·515	1·632			
1·35	1·526	1·643			
1·36	1·537	1·655			
1·37	1·547	1·666			
1·38	1·558	1·678			
1·39	1·569	1·689			
1·40	1·579	1·701			
1·41	1·590	1·712			
1·42	1·600	1·724			
1·43	1·611				
1·44	1·622				
1·45	1·632				
1·46	1·643				
1·47	1·653				
1·48	1·664				
1·49	1·675				
1·50	1·685				
1·51	1·696				
1·52	1·706				
1·53	1·717				
1·54	1·727				
1·55	1·738				
1·56	1·749				
1·57	1·759				
1·58	1·770				
1·59	1·780				
1·60	1·791				

TABLE 72 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

σ_{cbc} = 5·0 N/mm²
σ_{st} = 140 N/mm²

M/bd^2, N/mm²	d'/d = 0·05		d'/d = 0·10		d'/d = 0·15		d'/d = 0·20	
	P_t	P_c	P_t	P_c	P_t	P_c	P_t	P_c
0·87	0·717	0·003	0·717	0·004	0·717	0·005	0·717	0·006
0·90	0·739	0·030	0·741	0·037	0·742	0·046	0·744	0·062
0·95	0·777	0·070	0·780	0·091	0·784	0·116	0·789	0·154
1·00	0·815	0·119	0·820	0·146	0·826	0·186	0·833	0·247
1·05	0·852	0·163	0·860	0·201	0·868	0·256	0·878	0·340
1·10	0·890	0·208	0·899	0·256	0·910	0·325	0·923	0·432
1·15	0·927	0·252	0·939	0·311	0·952	0·395	0·967	0·525
1·20	0·965	0·297	0·979	0·366	0·994	0·465	1·012	0·617
1·25	1·003	0·342	1·019	0·421	1·036	0·534	1·057	0·710
1·30	1·040	0·386	1·058	0·476	1·078	0·604	1·101	0·802
1·35	1·078	0·431	1·098	0·530	1·120	0·674	1·146	0·895
1·40	1·115	0·475	1·138	0·585	1·162	0·744	1·190	0·988
1·45	1·153	0·520	1·177	0·640	1·204	0·813	1·235	1·080
1·50	1·190	0·564	1·217	0·695	1·247	0·883	1·280	1·173
1·55	1·228	0·609	1·257	0·750	1·289	0·953	1·324	1·265
1·60	1·266	0·653	1·296	0·805	1·331	1·023	1·369	1·358
1·65	1·303	0·698	1·336	0·860	1·373	1·092	1·414	1·451
1·70	1·341	0·743	1·376	0·914	1·415	1·162	1·458	1·543
1·75	1·378	0·787	1·415	0·969	1·457	1·232	1·503	1·636
1·80	1·416	0·832	1·455	1·024	1·499	1·301	1·548	1·728
1·85	1·454	0·876	1·495	1·079	1·541	1·371	1·592	1·821
1·90	1·491	0·921	1·534	1·134	1·583	1·441	1·637	1·914
1·95	1·529	0·965	1·574	1·189	1·625	1·511	1·682	2·006
2·00	1·566	1·010	1·614	1·244	1·667	1·580	1·726	2·099
2·05	1·604	1·054	1·653	1·299	1·709	1·650	1·771	2·191
2·10	1·642	1·099	1·693	1·353	1·751	1·720	1·815	2·284
2·15	1·679	1·144	1·733	1·408	1·793	1·789	1·860	2·377
2·20	1·717	1·188	1·772	1·463	1·835	1·859	1·905	2·469
2·25	1·754	1·233	1·812	1·518	1·877	1·929	1·949	2·562
2·30	1·792	1·277	1·852	1·573	1·919	1·999	1·994	2·654
2·35	1·830	1·322	1·892	1·628	1·961	2·068	2·039	2·747
2·40	1·867	1·366	1·931	1·683	2·003	2·138	2·083	2·840
2·45	1·905	1·411	1·971	1·738	2·045	2·208	2·128	2·932
2·50	1·942	1·455	2·011	1·792	2·087	2·277	2·173	3·025
2·55	1·980	1·500	2·050	1·847	2·129	2·347	2·217	3·117
2·60	2·018	1·545	2·090	1·902	2·171	2·417	2·262	3·210
2·65	2·055	1·589	2·130	1·957	2·213	2·487	2·307	3·302
2·70	2·093	1·634	2·169	2·012	2·255	2·556	2·351	3·395
2·75	2·130	1·678	2·209	2·067	2·297	2·626	2·396	3·488
2·80	2·168	1·723	2·249	2·122	2·339	2·696	2·440	3·580
2·85	2·206	1·767	2·288	2·177	2·381	2·765	2·485	3·673
2·90	2·243	1·812	2·328	2·231	2·423	2·835	2·530	3·765
2·95	2·281	1·857	2·368	2·286	2·465	2·905	2·574	3·858
3·00	2·318	1·901	2·407	2·341	2·507	2·975	2·619	3·951
3·05	2·356	1·946	2·447	2·396	2·549	3·044	2·664	4·043

TABLE 73 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

σ_{cbc} = 7·0 N/mm²
σ_{st} = 140 N/mm²

M_u/bd^2 N/mm²	d'/d = 0·05		d'/d = 0·10		d'/d = 0·15		d'/d = 0·20	
	p_t	p_c	p_t	p_c	p_t	p_c	p_t	p_c
1·22	1·005	0·006	1·005	0·007	1·006	0·009	1·006	0·013
1·25	1·028	0·033	1·029	0·041	1·031	0·052	1·033	0·069
1·30	1·065	0·078	1·069	0·097	1·073	0·123	1·077	0·163
1·35	1·103	0·124	1·108	0·152	1·115	0·193	1·122	0·257
1·40	1·140	0·169	1·148	0·208	1·157	0·264	1·167	0·351
1·45	1·178	0·214	1·188	0·264	1·199	0·335	1·211	0·445
1·50	1·216	0·259	1·228	0·319	1·241	0·406	1·255	0·539
1·55	1·253	0·305	1·267	0·375	1·283	0·476	1·301	0·633
1·60	1·291	0·350	1·307	0·431	1·325	0·547	1·345	0·727
1·65	1·328	0·395	1·347	0·486	1·367	0·618	1·390	0·821
1·70	1·366	0·440	1·386	0·542	1·409	0·689	1·435	0·915
1·75	1·404	0·485	1·426	0·598	1·451	0·760	1·479	1·009
1·80	1·441	0·531	1·466	0·653	1·493	0·830	1·524	1·103
1·85	1·479	0·576	1·505	0·709	1·535	0·901	1·568	1·197
1·90	1·516	0·621	1·545	0·765	1·577	0·972	1·613	1·291
1·95	1·554	0·666	1·585	0·821	1·619	1·043	1·658	1·385
2·00	1·591	0·712	1·624	0·876	1·661	1·113	1·702	1·479
2·05	1·629	0·757	1·664	0·932	1·703	1·184	1·747	1·573
2·10	1·667	0·802	1·704	0·988	1·745	1·255	1·792	1·667
2·15	1·704	0·847	1·743	1·043	1·787	1·326	1·836	1·761
2·20	1·742	0·892	1·783	1·099	1·829	1·396	1·881	1·855
2·25	1·779	0·938	1·823	1·155	1·871	1·467	1·926	1·949
2·30	1·817	0·983	1·862	1·210	1·913	1·538	1·970	2·043
2·35	1·855	1·028	1·902	1·266	1·955	1·609	2·015	2·137
2·40	1·892	1·073	1·942	1·322	1·997	1·680	2·060	2·231
2·45	1·930	1·119	1·981	1·378	2·039	1·750	2·104	2·325
2·50	1·967	1·164	2·021	1·433	2·081	1·821	2·149	2·419
2·55	2·005	1·209	2·061	1·489	2·123	1·892	2·193	2·513
2·60	2·043	1·254	2·101	1·545	2·165	1·963	2·238	2·607
2·65	2·080	1·299	2·140	1·600	2·207	2·033	2·283	2·701
2·70	2·118	1·345	2·180	1·656	2·249	2·104	2·327	2·795
2·75	2·155	1·390	2·220	1·712	2·291	2·175	2·372	2·888
2·80	2·193	1·435	2·259	1·767	2·333	2·246	2·417	2·982
2·85	2·231	1·480	2·299	1·823	2·375	2·316	2·461	3·076
2·90	2·268	1·526	2·339	1·879	2·417	2·387	2·506	3·170
2·95	2·306	1·571	2·378	1·934	2·459	2·458	2·551	3·264
3·00	2·343	1·616	2·418	1·990	2·501	2·529	2·595	3·358
3·05	2·381	1·661	2·458	2·046	2·543	2·599	2·640	3·452
3·10	2·419	1·707	2·497	2·102	2·585	2·670	2·685	3·546
3·15	2·456	1·752	2·537	2·157	2·627	2·741	2·729	3·640
3·20	2·494	1·797	2·577	2·213	2·669	2·812	2·774	3·734
3·25	2·531	1·842	2·616	2·269	2·711	2·883	2·818	3·828
3·30	2·569	1·887	2·656	2·324	2·754	2·953	2·863	3·922
3·35	2·607	1·933	2·696	2·380	2·796	3·024	2·908	4·016
3·40	2·644	1·978	2·735	2·436	2·838	3·095	2·952	4·110

TABLE 74 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

σ_{cbc} = 8·5 N/mm²
σ_{st} = 140 N/mm²

M/bd^2, N/mm²	d'/d = 0·05		d'/d = 0·10		d'/d = 0·15		d'/d = 0·20	
	p_t	p_c	p_t	p_c	p_t	p_c	p_t	p_c
1·48	1·219	0·006	1·220	0·008	1·220	0·010	1·220	0·013
1·50	1·234	0·024	1·235	0·030	1·237	0·038	1·238	0·051
1·55	1·272	0·070	1·275	0·086	1·279	0·110	1·283	0·146
1·60	1·310	0·116	1·315	0·143	1·321	0·181	1·327	0·241
1·65	1·347	0·162	1·354	0·199	1·363	0·253	1·372	0·336
1·70	1·385	0·207	1·394	0·255	1·405	0·324	1·417	0·431
1·75	1·422	0·253	1·434	0·312	1·447	0·396	1·461	0·526
1·80	1·460	0·299	1·474	0·368	1·489	0·468	1·506	0·621
1·85	1·497	0·345	1·513	0·424	1·531	0·539	1·551	0·716
1·90	1·535	0·390	1·553	0·481	1·573	0·611	1·595	0·811
1·95	1·573	0·436	1·593	0·537	1·615	0·682	1·640	0·906
2·00	1·610	0·482	1·632	0·593	1·657	0·754	1·685	1·001
2·05	1·648	0·528	1·672	0·650	1·699	0·825	1·729	1·096
2·10	1·685	0·573	1·712	0·706	1·741	0·897	1·774	1·191
2·15	1·723	0·619	1·751	0·762	1·783	0·969	1·818	1·286
2·20	1·761	0·665	1·791	0·819	1·825	1·040	1·863	1·382
2·25	1·798	0·711	1·831	0·875	1·867	1·112	1·908	1·477
2·30	1·836	0·756	1·870	0·931	1·909	1·183	1·952	1·572
2·35	1·873	0·802	1·910	0·988	1·951	1·255	1·997	1·667
2·40	1·911	0·848	1·950	1·044	1·993	1·326	2·042	1·762
2·45	1·949	0·893	1·989	1·100	2·035	1·398	2·086	1·857
2·50	1·986	0·939	2·029	1·157	2·077	1·470	2·131	1·952
2·55	2·024	0·985	2·069	1·213	2·119	1·541	2·176	2·047
2·60	2·061	1·031	2·108	1·269	2·161	1·613	2·220	2·142
2·65	2·099	1·076	2·148	1·326	2·203	1·684	2·265	2·237
2·70	2·137	1·122	2·188	1·382	2·245	1·756	2·310	2·332
2·75	2·174	1·168	2·228	1·438	2·287	1·827	2·354	2·427
2·80	2·212	1·214	2·267	1·495	2·329	1·899	2·399	2·522
2·85	2·249	1·259	2·307	1·551	2·371	1·971	2·443	2·617
2·90	2·287	1·305	2·347	1·607	2·413	2·042	2·488	2·712
2·95	2·325	1·351	2·386	1·664	2·455	2·114	2·533	2·807
3·00	2·362	1·397	2·426	1·720	2·497	2·185	2·577	2·902
3·05	2·400	1·442	2·466	1·776	2·539	2·257	2·622	2·997
3·10	2·437	1·488	2·505	1·833	2·581	2·329	2·667	3·093
3·15	2·475	1·534	2·545	1·889	2·623	2·400	2·711	3·188
3·20	2·513	1·580	2·585	1·945	2·665	2·472	2·756	3·283
3·25	2·550	1·625	2·624	2·002	2·707	2·543	2·801	3·378
3·30	2·588	1·671	2·664	2·058	2·749	2·615	2·845	3·473
3·35	2·625	1·717	2·704	2·114	2·791	2·686	2·890	3·568
3·40	2·663	1·763	2·743	2·171	2·833	2·758	2·935	3·663
3·45	2·701	1·808	2·783	2·227	2·875	2·830	2·979	3·758
3·50	2·738	1·854	2·823	2·283	2·917	2·901	3·024	3·853
3·55	2·776	1·900	2·862	2·340	2·959	2·973	3·068	3·948
3·60	2·813	1·946	2·902	2·396	3·001	3·044	3·113	4·043
3·65	2·851	1·991	2·942	2·452	3·043	3·116	3·158	4·138

TABLE 75 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

σ_{cbc} = 10·0 N/mm²
σ_{st} = 140 N/mm²

M/bd^2, N/mm²	d'/d=0·05		d'/d=0·10		d'/d=0·15		d'/d=0·20	
	p_t	p_c	p_t	p_c	p_t	p_c	p_t	p_c
1·74	1·434	0·006	1·434	0·008	1·434	0·010	1·435	0·013
1·75	1·441	0·015	1·442	0·019	1·443	0·024	1·443	0·032
1·80	1·479	0·062	1·481	0·076	1·485	0·097	1·488	0·128
1·85	1·516	0·108	1·521	0·133	1·527	0·169	1·533	0·224
1·90	1·554	0·154	1·561	0·190	1·569	0·241	1·577	0·321
1·95	1·591	0·201	1·601	0·247	1·611	0·314	1·622	0·417
2·00	1·629	0·247	1·640	0·304	1·653	0·386	1·667	0·513
2·05	1·667	0·293	1·680	0·361	1·695	0·459	1·711	0·609
2·10	1·704	0·339	1·720	0·418	1·737	0·531	1·756	0·705
2·15	1·742	0·386	1·759	0·475	1·779	0·603	1·801	0·801
2·20	1·779	0·432	1·799	0·532	1·821	0·676	1·845	0·897
2·25	1·817	0·478	1·839	0·589	1·863	0·748	1·890	0·994
2·30	1·855	0·524	1·878	0·646	1·905	0·821	1·935	1·090
2·35	1·892	0·571	1·918	0·703	1·947	0·893	1·979	1·186
2·40	1·930	0·617	1·958	0·760	1·989	0·965	2·024	1·282
2·45	1·967	0·663	1·997	0·817	2·031	1·038	2·068	1·378
2·50	2·005	0·709	2·037	0·874	2·073	1·110	2·113	1·474
2·55	2·043	0·756	2·077	0·931	2·115	1·183	2·158	1·571
2·60	2·080	0·802	2·116	0·988	2·157	1·255	2·202	1·667
2·65	2·118	0·848	2·156	1·045	2·199	1·327	2·247	1·763
2·70	2·155	0·895	2·196	1·102	2·241	1·400	2·292	1·859
2·75	2·193	0·941	2·235	1·159	2·283	1·472	2·336	1·955
2·80	2·231	0·987	2·275	1·216	2·325	1·545	2·381	2·051
2·85	2·268	1·033	2·315	1·273	2·367	1·617	2·426	2·147
2·90	2·306	1·080	2·354	1·330	2·409	1·689	2·470	2·244
2·95	2·343	1·126	2·394	1·387	2·451	1·762	2·515	2·340
3·00	2·381	1·172	2·434	1·444	2·493	1·834	2·560	2·436
3·05	2·419	1·218	2·474	1·500	2·535	1·906	2·604	2·532
3·10	2·456	1·265	2·513	1·557	2·577	1·979	2·649	2·628
3·15	2·494	1·311	2·553	1·614	2·619	2·051	2·693	2·724
3·20	2·531	1·357	2·593	1·671	2·661	2·124	2·738	2·821
3·25	2·569	1·404	2·632	1·728	2·703	2·196	2·783	2·917
3·30	2·607	1·450	2·672	1·785	2·745	2·268	2·827	3·013
3·35	2·644	1·496	2·712	1·842	2·787	2·341	2·872	3·109
3·40	2·682	1·542	2·751	1·899	2·829	2·413	2·917	3·205
3·45	2·719	1·589	2·791	1·956	2·871	2·486	2·961	3·301
3·50	2·757	1·635	2·831	2·013	2·913	2·558	3·006	3·397
3·55	2·794	1·681	2·870	2·070	2·955	2·630	3·051	3·494
3·60	2·832	1·727	2·910	2·127	2·997	2·703	3·095	3·590
3·65	2·870	1·774	2·950	2·184	3·039	2·775	3·140	3·686
3·70	2·907	1·820	2·989	2·241	3·081	2·848	3·185	3·782
3·75	2·945	1·866	3·029	2·298	3·123	2·920	3·229	3·878
3·80	2·982	1·912	3·069	2·355	3·165	2·992	3·274	3·974
3·85	3·020	1·959	3·108	2·412	3·207	3·065		
3·90	3·058	2·005	3·148	2·469	3·249	3·137		

TABLE 76 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

σ_{cbc} = 5·0 N/mm²
σ_{st} = 230 N/mm²

M/bd^2, N/mm	d'/d = 0·05		d'/d = 0·10		d'/d =		d'/d = 0·20	
	p_t	p_c	p_t	p_c	p_t	p_c	p_t	p_c
0·66	0·317	0·007	0·318	0·010	0·318	0·014	0·318	0·023
0·70	0·336	0·045	0·337	0·060	0·338	0·087	0·340	0·144
0·75	0·359	0·092	0·361	0·123	0·364	0·177	0·367	0·295
0·80	0·381	0·139	0·385	0·186	0·389	0·268	0·394	0·446
0·85	0·404	0·187	0·409	0·249	0·415	0·359	0·421	0·596
0·90	0·427	0·234	0·433	0·312	0·441	0·450	0·448	0·747
0·95	0·450	0·281	0·458	0·375	0·466	0·540	0·476	0·898
1·00	0·473	0·328	0·482	0·438	0·492	0·631	0·503	1·048
1·05	0·496	0·375	0·506	0·501	0·517	0·722	0·530	1·199
1·10	0·519	0·422	0·530	0·564	0·543	0·812	0·557	1·350
1·15	0·542	0·469	0·554	0·627	0·568	0·903	0·584	1·501
1·20	0·564	0·517	0·578	0·690	0·594	0·994	0·611	1·651
1·25	0·587	0·564	0·603	0·753	0·620	1·085	0·639	1·802
1·30	0·610	0·611	0·627	0·816	0·645	1·175	0·666	1·953
1·35	0·633	0·658	0·651	0·879	0·671	1·266	0·693	2·104
1·40	0·656	0·705	0·675	0·942	0·696	1·357	0·720	2·254
1·45	0·679	0·752	0·699	1·005	0·722	1·447	0·747	2·405
1·50	0·702	0·800	0·723	1·068	0·747	1·538	0·775	2·556
1·55	0·725	0·847	0·747	1·131	0·773	1·629	0·802	2·707
1·60	0·748	0·894	0·772	1·194	0·799	1·719	0·829	2·857
1·65	0·770	0·941	0·796	1·257	0·824	1·810	0·856	3·008
1·70	0·793	0·988	0·820	1·319	0·850	1·901	0·883	3·159
1·75	0·816	1·035	0·844	1·382	0·875	1·992	0·910	3·309
1·80	0·839	1·082	0·868	1·445	0·901	2·082	0·938	3·460
1·85	0·862	1·130	0·892	1·508	0·926	2·173	0·965	3·611
1·90	0·885	1·177	0·917	1·571	0·952	2·264	0·992	3·762
1·95	0·908	1·224	0·941	1·634	0·978	2·354	1·019	3·912
2·00	0·931	1·271	0·965	1·697	1·003	2·445		
2·05	0·953	1·318	0·989	1·760	1·029	2·536		
2·10	0·976	1·365	1·013	1·823	1·054	2·627		
2·15	0·999	1·413	1·037	1·886	1·080	2·717		
2·20	1·022	1·460	1·061	1·949	1·105	2·808		
2·25	1·045	1·507	1·086	2·012	1·131	2·899		
2·30	1·068	1·554	1·110	2·075	1·157	2·989		
2·35	1·091	1·601	1·134	2·138	1·182	3·080		
2·40	1·114	1·648	1·158	2·201	1·208	3·171		
2·45	1·137	1·695	1·182	2·264	1·233	3·262		
2·50	1·159	1·743	1·206	2·327	1·259	3·352		
2·55	1·182	1·790	1·231	2·390	1·284	3·443		
2·60	1·205	1·837	1·255	2·453	1·310	3·534		
2·65	1·228	1·884	1·279	2·516	1·336	3·624		
2·70	1·251	1·931	1·303	2·579	1·361	3·715		
2·75	1·274	1·978	1·327	2·642	1·387	3·806		
2·80	1·297	2·026	1·351	2·705	1·412	3·897		
2·85	1·320	2·073	1·375	2·768	1·438	3·987		

WORKING STRESS METHOD

TABLE 77 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

σ_{cbc} = 7·0 N/mm²
σ_{st} = 230 N/mm²

M/bd^2, N/mm²	d'/d = 0·05 P_t	d'/d = 0·05 P_c	d'/d = 0·10 P_t	d'/d = 0·10 P_c	d'/d = 0·15 P_t	d'/d = 0·15 P_c	d'/d = 0·20 P_t	d'/d = 0·20 P_c
0·92	0·442	0·007	0·443	0·009	0·443	0·013	0·443	0·021
0·95	0·456	0·035	0·457	0·047	0·458	0·068	0·459	0·113
1·00	0·479	0·083	0·481	0·111	0·484	0·160	0·486	0·266
1·05	0·502	0·131	0·505	0·175	0·509	0·252	0·514	0·419
1·10	0·525	0·179	0·530	0·239	0·535	0·344	0·541	0·572
1·15	0·548	0·227	0·554	0·303	0·560	0·436	0·568	0·725
1·20	0·571	0·275	0·578	0·367	0·586	0·528	0·595	0·878
1·25	0·593	0·323	0·602	0·431	0·612	0·620	0·622	1·031
1·30	0·616	0·370	0·626	0·495	0·637	0·712	0·650	1·184
1·35	0·639	0·418	0·650	0·558	0·663	0·805	0·677	1·337
1·40	0·662	0·466	0·674	0·622	0·688	0·897	0·704	1·490
1·45	0·685	0·514	0·699	0·686	0·714	0·989	0·731	1·643
1·50	0·708	0·562	0·723	0·750	0·739	1·081	0·758	1·796
1·55	0·731	0·610	0·747	0·814	0·765	1·173	0·785	1·949
1·60	0·754	0·658	0·771	0·878	0·791	1·265	0·813	2·102
1·65	0·777	0·705	0·795	0·942	0·816	1·357	0·840	2·255
1·70	0·799	0·753	0·819	1·006	0·842	1·449	0·867	2·408
1·75	0·822	0·801	0·844	1·070	0·867	1·541	0·894	2·561
1·80	0·845	0·849	0·868	1·134	0·893	1·633	0·921	2·714
1·85	0·868	0·897	0·892	1·198	0·919	1·725	0·948	2·867
1·90	0·891	0·945	0·916	1·262	0·944	1·817	0·976	3·020
1·95	0·914	0·993	0·940	1·325	0·970	1·909	1·003	3·173
2·00	0·937	1·040	0·964	1·389	0·995	2·002	1·030	3·326
2·05	0·960	1·088	0·988	1·453	1·021	2·094	1·057	3·479
2·10	0·982	1·136	1·013	1·517	1·046	2·186	1·084	3·632
2·15	1·005	1·184	1·037	1·581	1·072	2·278	1·111	3·785
2·20	1·028	1·232	1·061	1·645	1·098	2·370	1·139	3·938
2·25	1·051	1·280	1·085	1·709	1·123	2·462		
2·30	1·074	1·328	1·109	1·773	1·149	2·554		
2·35	1·097	1·376	1·133	1·837	1·174	2·646		
2·40	1·120	1·423	1·158	1·901	1·200	2·738		
2·45	1·143	1·471	1·182	1·965	1·225	2·830		
2·50	1·166	1·519	1·206	2·028	1·251	2·922		
2·55	1·188	1·567	1·230	2·092	1·277	3·014		
2·60	1·211	1·615	1·254	2·156	1·302	3·106		
2·65	1·234	1·663	1·278	2·220	1·328	3·198		
2·70	1·257	1·711	1·303	2·284	1·353	3·291		
2·75	1·280	1·758	1·327	2·348	1·379	3·383		
2·80	1·303	1·806	1·351	2·412	1·404	3·475		
2·85	1·326	1·854	1·375	2·476	1·430	3·567		
2·90	1·349	1·902	1·399	2·540	1·456	3·659		
2·95	1·371	1·950	1·423	2·604	1·481	3·751		
3·00	1·394	1·998	1·447	2·668	1·507	3·843		
3·05	1·417	2·046	1·472	2·731	1·532	3·935		
3·10	1·440	2·093	1·496	2·795	1·558	4·027		

TABLE 78 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

σ_{cbc} = 8·5 N/mm²
σ_{st} = 230 N/mm²

M/bd^2, N/mm²	d'/d = 0·05 p_t	d'/d = 0·05 p_c	d'/d = 0·10 p_t	d'/d = 0·10 p_c	d'/d = 0·15 p_t	d'/d = 0·15 p_c	d'/d = 0·20 p_t	d'/d = 0·20 p_c
1·11	0·534	0·001	0·534	0·002	0·534	0·002	0·534	0·004
1·15	0·552	0·040	0·553	0·053	0·554	0·077	0·556	0·128
1·20	0·575	0·088	0·577	0·118	0·580	0·170	0·583	0·282
1·25	0·598	0·137	0·602	0·183	0·606	0·263	0·610	0·437
1·30	0·621	0·185	0·626	0·247	0·631	0·356	0·637	0·592
1·35	0·644	0·234	0·650	0·312	0·657	0·449	0·665	0·747
1·40	0·667	0·282	0·674	0·377	0·682	0·542	0·692	0·901
1·45	0·690	0·330	0·698	0·441	0·708	0·636	0·719	1·056
1·50	0·712	0·379	0·722	0·506	0·734	0·729	0·746	1·211
1·55	0·735	0·427	0·747	0·570	0·759	0·822	0·773	1·366
1·60	0·758	0·476	0·771	0·635	0·785	0·915	0·800	1·520
1·65	0·781	0·524	0·795	0·700	0·810	1·008	0·828	1·675
1·70	0·804	0·572	0·819	0·764	0·836	1·101	0·855	1·830
1·75	0·827	0·621	0·843	0·829	0·861	1·194	0·882	1·985
1·80	0·850	0·669	0·867	0·894	0·887	1·287	0·909	2·139
1·85	0·873	0·718	0·891	0·958	0·913	1·381	0·936	2·294
1·90	0·896	0·766	0·916	1·023	0·938	1·474	0·963	2·449
1·95	0·918	0·814	0·940	1·088	0·964	1·567	0·991	2·604
2·00	0·941	0·863	0·964	1·152	0·989	1·660	1·018	2·758
2·05	0·964	0·911	0·988	1·217	1·015	1·753	1·045	2·913
2·10	0·987	0·960	1·012	1·282	1·040	1·846	1·072	3·068
2·15	1·010	1·008	1·036	1·346	1·066	1·939	1·099	3·223
2·20	1·033	1·057	1·061	1·411	1·092	2·032	1·126	3·377
2·25	1·056	1·105	1·085	1·475	1·117	2·126	1·154	3·532
2·30	1·079	1·153	1·109	1·540	1·143	2·219	1·181	3·687
2·35	1·101	1·202	1·133	1·605	1·168	2·312	1·208	3·842
2·40	1·124	1·250	1·157	1·669	1·194	2·405	1.235	3·996
2·45	1·147	1·299	1·181	1·734	1·219	2·498		
2·50	1·170	1·347	1·205	1·799	1·245	2·591		
2·55	1·193	1·395	1·230	1·863	1·271	2·684		
2·60	1·216	1·444	1·254	1·928	1·296	2·777		
2·65	1·239	1·492	1·278	1·993	1·322	2·871		
2·70	1·262	1·541	1·302	2·057	1·347	2·964		
2·75	1·285	1·589	1·326	2·122	1·373	3·057		
2·80	1·307	1·637	1·350	2·186	1·398	3·150		
2·85	1·330	1·686	1·375	2·251	1·424	3·243		
2·90	1·353	1·734	1·399	2·316	1·450	3·336		
2·95	1·376	1·783	1·423	2·380	1·475	3·429		
3·00	1·399	1·831	1·447	2·445	1·501	3·522		
3·05	1·422	1·879	1·471	2·510	1·526	3·616		
3·10	1·445	1·928	1·495	2·574	1·552	3·709		
3·15	1·468	1·976	1·520	2·639	1·578	3·802		
3·20	1·490	2·025	1·544	2·704	1·603	3·895		
3·25	1·513	2·073	1·568	2·768	1·629	3·988		
3·30	1·536	2·122	1·592	2·833	1·654	4·081		

TABLE 79 FLEXURE — REINFORCEMENT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

σ_{cbc} = 10·0 N/mm²
σ_{st} = 230 N/mm²

M/bd^2, N/mm²	d'/d=0·05		d'/d=0·10		d'/d=0·15		d'/d=0·20	
	p_t	p_c	p_t	p_c	p_t	p_c	p_t	p_c
1·31	0·630	0·005	0·630	0·007	0·630	0·011	0·631	0·017
1·35	0·648	0·045	0·650	0·060	0·651	0·086	0·652	0·143
1·40	0·671	0·094	0·674	0·125	0·676	0·180	0·679	0·299
1·45	0·694	0·143	0·698	0·190	0·702	0·274	0·707	0·456
1·50	0·717	0·192	0·722	0·256	0·728	0·368	0·734	0·612
1·55	0·740	0·241	0·746	0·321	0·753	0·463	0·761	0·769
1·60	0·763	0·289	0·770	0·387	0·779	0·557	0·788	0·925
1·65	0·786	0·338	0·794	0·452	0·804	0·651	0·815	1·082
1·70	0·809	0·387	0·819	0·517	0·830	0·745	0·843	1·238
1·75	0·831	0·436	0·443	0·583	0·855	0·839	0·870	1·395
1·80	0·854	0·485	0·867	0·648	0·881	0·934	0·897	1·551
1·85	0·877	0·534	0·891	0·713	0·907	1·028	0·924	1·708
1·90	0·900	0·583	0·915	0·779	0·932	1·122	0·951	1·865
1·95	0·923	0·632	0·939	0·844	0·958	1·216	0·978	2·021
2·00	0·946	0·681	0·964	0·910	0·983	1·310	1·006	2·178
2·05	0·969	0·730	0·988	0·975	1·009	1·405	1·033	2·334
2·10	0·992	0·779	1·012	1·040	1·034	1·499	1·060	2·491
2·15	1·015	0·828	1·036	1·106	1·060	1·593	1·087	2·647
2·20	1·037	0·877	1·060	1·171	1·086	1·687	1·114	2·804
2·25	1·060	0·926	1·084	1·237	1·111	1·781	1·141	2·960
2·30	1·083	0·975	1·108	1·302	1·137	1·876	1·169	3·117
2·35	1·106	1·024	1·133	1·367	1·162	1·970	1·196	3·273
2·40	1·129	1·073	1·157	1·433	1·188	2·064	1·223	3·430
2·45	1·152	1·122	1·181	1·498	1·213	2·158	1·250	3·586
2·50	1·175	1·171	1·205	1·564	1·239	2·252	1·277	3·743
2·55	1·198	1·220	1·229	1·629	1·265	2·347	1·304	3·899
2·60	1·220	1·269	1·253	1·694	1·290	2·441		
2·65	1·243	1·318	1·278	1·760	1·316	2·535		
2·70	1·266	1·367	1·302	1·825	1·341	2·629		
2·75	1·289	1·416	1·326	1·890	1·367	2·723		
2·80	1·312	1·465	1·350	1·956	1·393	2·818		
2·85	1·335	1·514	1·374	2·021	1·418	2·912		
2·90	1·358	1·563	1·398	2·087	1·444	3·006		
2·95	1·381	1·612	1·422	2·152	1·469	3·100		
3·00	1·404	1·661	1·447	2·217	1·495	3·194		
3·05	1·426	1·710	1·471	2·283	1·520	3·289		
3·10	1·449	1·759	1·495	2·348	1·546	3·383		
3·15	1·472	1·807	1·519	2·414	1·572	3·477		
3·20	1·495	1·856	1·543	2·479	1·597	3·571		
3·25	1·518	1·905	1·567	2·544	1·623	3·665		
3·30	1·541	1·954	1·592	2·610	1·648	3·760		
3·35	1·564	2·003	1·616	2·675	1·674	3·854		
3·40	1·587	2·052	1·640	2·740	1·699	3·948		
3·45	1·609	2·101	1·664	2·806				
3·50	1·632	2·150	1·688	2·871				

TABLE 80 SHEAR — PERMISSIBLE SHEAR STRESS IN CONCRETE, τ_c, N/mm²

$\frac{100 A_{st}}{bd}$	Grade of Concrete					
	M15	M20	M25	M30	M35	M40
0·20	0·20	0·20	0·21	0·21	0·21	0·21
0·30	0·24	0·24	0·25	0·25	0·25	0·25
0·40	0·27	0·27	0·28	0·28	0·29	0·29
0·50	0·29	0·30	0·31	0·31	0·31	0·32
0·60	0·31	0·32	0·33	0·33	0·34	0·34
0·70	0·33	0·34	0·35	0·36	0·36	0·37
0·80	0·34	0·36	0·37	0·38	0·38	0·39
0·90	0·36	0·37	0·39	0·39	0·40	0·41
1·00	0·37	0·39	0·40	0·41	0·42	0·42
1·10	0·38	0·40	0·42	0·43	0·43	0·44
1·20	0·40	0·41	0·43	0·44	0·45	0·45
1·30	0·41	0·43	0·44	0·45	0·46	0·47
1·40	0·42	0·44	0·45	0·46	0·47	0·48
1·50	0·42	0·45	0·46	0·48	0·49	0·49
1·60	0·43	0·46	0·47	0·49	0·50	0·51
1·70	0·44	0·47	0·48	0·50	0·51	0·52
1·80	0·44	0·47	0·49	0·51	0·52	0·53
1·90	0·44	0·48	0·50	0·52	0·53	0·54
2·00	0·44	0·49	0·51	0·53	0·54	0·55
2·10	0·44	0·50	0·52	0·54	0·55	0·56
2·20	0·44	0·51	0·53	0·54	0·56	0·57
2·30	0·44	0·51	0·53	0·55	0·57	0·58
2·40	0·44	0·51	0·54	0·56	0·57	0·59
2·50	0·44	0·51	0·55	0·57	0·58	0·60
2·60	0·44	0·51	0·56	0·57	0·59	0·60
2·70	0·44	0·51	0·56	0·58	0·60	0·61
2·80	0·44	0·51	0·57	0·59	0·60	0·62
2·90	0·44	0·51	0·57	0·59	0·61	0·63

TABLE 81 SHEAR — VERTICAL STIRRUPS

Values of $\frac{V_s}{d}$ for two legged stirrups, kN/cm

Stirrup Spacing, cm	σ_{sv} = 140 N/mm², Diameter, mm				σ_{sv} = 230 N/mm², Diameter, mm			
	6	8	10	12	6	8	10	12
5	1·583	2·815	4·398	6·333	2·601	4.624	7·226	10·405
6	1·314	2·346	3·665	5·278	2·168	3.854	6·021	8·671
7	1·131	2·011	3·142	4·524	1·858	3.303	5·161	7·432
8	0·990	1·759	2·749	3·958	1·626	2.890	4·516	6·503
9	0·880	1·564	2·443	3·519	1·445	2·569	4·014	5·781
10	0·792	1·407	2·199	3·167	1·301	2·312	3·613	5·202
11	0·720	1·279	1·999	2·879	1·182	2·102	3·284	4·730
12	0·660	1·173	1·833	2·639	1·084	1·927	3·012	4·335
13	0·609	1·083	1·692	2·436	1·000	1·779	2·779	4·002
14	0·565	1·005	1·571	2·262	0·920	1·652	2·580	3·716
15	0·528	0·938	1·466	2·111	0·867	1·541	2·409	3·468
16	0·495	0·880	1·374	1·979	0·813	1·445	2·258	3·252
17	0·466	0·828	1·294	1·863	0·765	1·360	2·125	3·060
18	0·440	0·782	1·222	1·759	0·723	1·285	2·007	2·890
19	0·417	0·741	1·157	1·667	0·605	1·217	1·901	2·738
20	0·396	0·704	1·100	1·583	0·650	1·156	1·806	2·601
25	0·317	0·563	0·880	1·267	0·520	0·925	1·445	2·081
30	0·264	0·469	0·733	1·056	0·432	0·771	1·204	1·734
35	0·226	0·402	0·628	0·905	0·372	0·661	1·032	1·486
40	0·198	0·352	0·550	0·792	0·325	0·578	0·903	1·301
45	0·176	0·313	0·489	0·704	0·289	0·514	0·803	1·156

TABLE 82 SHEAR — BENT UP BARS

Values of V_s for single bar, kN

Bar Diameter, mm	σ_{sv} = 140 N/mm² up to 20 mm diameter = 130 N/mm² over 20 mm diameter		σ_{sv} = 230 N/mm²	
	α=45°	α=60°	α=45°	α=60°
10	7·78	9·52	12·77	15·64
12	11·20	13·71	18·39	22 53
16	19·90	24·38	32·70	40·05
18	25·19	30·86	41·39	50·69
20	31·10	38·09	51·09	62·58
22	34·94	42·80	61·82	75·72
25	45·12	55·26	79·83	97·77
28	56·60	69·32	100·14	122.65
32	73·93	90·54	130·80	160.19
36	93·57	114·60	165·54	202·75

NOTE — α is the angle between the bent up bar and the axis of the member.

TABLE 83 DEVELOPMENT LENGTH FOR PLAIN BARS

σ_{st} = 140 N/mm² for bars up to 20 mm diameter
= 130 N/mm² for bars over 20 mm diameter
σ_{sc} = 130 N/mm² for all diameter

Tabulated values are in centimetres.

Bar Diameter, mm	Tension Bars Grade of Concrete				Compression Bars Grade of Concrete			
	M15	M20	M25	M30	M15	M20	M25	M30
6	35·0	26·3	23·3	21·0	26·0	19·5	17·3	15·6
8	46·7	35·0	31·1	28·0	34·7	26·0	23·1	20·8
10	58·3	43·8	38·9	35·0	43·3	32·5	28·9	26·0
12	70·0	52·5	46·7	42·0	52·0	39·0	34·7	31·2
16	93·3	70·0	62·2	56·0	69·3	52·0	46·2	41·6
18	105·0	78·8	70·0	63·0	78·0	58·5	52·0	46·8
20	116·7	87·5	77·8	70·0	86·7	65·0	57·8	52·0
22	119·2	89·4	79·4	71·5	95·3	71·5	63·6	57·2
25	135·4	101·6	90·3	81·3	108·3	81·3	72·2	65·0
28	151·7	113·8	101·1	91·0	121·3	91·0	80·9	72·8
32	173·3	130·0	115·6	104·0	138·7	104·0	92·4	83·2
36	195·0	146·3	130·0	117·0	156·0	117·0	104·0	93·6

TABLE 84 DEVELOPMENT LENGTH FOR DEFORMED BARS

Tabulated values are in centimetres.

$\sigma_{st} = 230$ N/mm²
$\sigma_{sc} = 190$ N/mm²

Bar Diameter, mm	Tension Bars, Grade of Concrete				Compression Bars, Grade of Concrete			
	M15	M20	M25	M30	M15	M20	M25	M30
6	41·1	30·8	27·4	24·6	27·1	20·4	18·1	16·3
8	54·8	41·1	36·5	32·9	36·2	27·1	24·1	21·7
10	68·5	51·3	45·6	41·1	45·2	33·9	30·2	27·1
12	82·1	61·6	54·8	49·3	54·3	40·7	36·2	32·6
16	109·5	82·1	73·0	65·7	72·4	54·3	48·3	43·4
18	123·2	92·4	82·1	73·9	81·4	61·1	54·3	48·9
20	136·9	102·7	91·3	82·1	90·5	67·9	60·3	54·3
22	150·6	112·9	100·4	90·4	99·5	74·6	66·3	59·7
25	171·1	128·3	114·1	102·7	113·1	84·8	75·4	67·9
28	191·7	143·8	127·8	115·0	126·7	95·0	84·4	76·0
32	219·0	164·3	146·0	131·4	144·8	108·6	96·5	86·9
36	246·4	184·8	164·3	147·9	162·9	122·1	108·6	97·7

TABLE 85 DEVELOPMENT LENGTH FOR DEFORMED BARS

Tabulated values are in centimetres.

$\sigma_{st} = 275$ N/mm²
$\sigma_{sc} = 190$ N/mm²

Bar Diameter, mm	Tension Bars, Grade of Concrete				Compression Bars, Grade of Concrete			
	M15	M20	M25	M30	M15	M20	M25	M30
6	49·1	36·8	32·7	29·5	27·1	20·4	18·1	16·3
8	65·5	49·1	43·7	39·3	36·2	27·1	24·1	21·7
10	81·8	61·4	54·6	49·1	45·2	33·9	30·2	27·1
12	98·2	73·7	65·5	58·9	54·3	40·7	36·2	32·6
16	131·0	98.2	87·3	78·6	72·4	54·3	48·3	43·4
18	147·3	110·5	98·2	88·4	81·4	61·1	54·3	48·9
20	163·7	122·8	109·1	98·2	90·5	67·9	60·3	54·3
22	180·1	135·0	120·0	108·0	99·5	74·6	66·3	59·7
25	204·6	153·5	136·4	122·8	113·1	84·8	75·4	67·9
28	229·2	171·9	152·8	137·5	126·7	95·0	84·4	76·0
32	261·9	196·4	174·6	157·1	144·8	108·6	96·5	86·9
36	294·6	221·0	196·4	176·8	162·9	122·1	108·6	97·7

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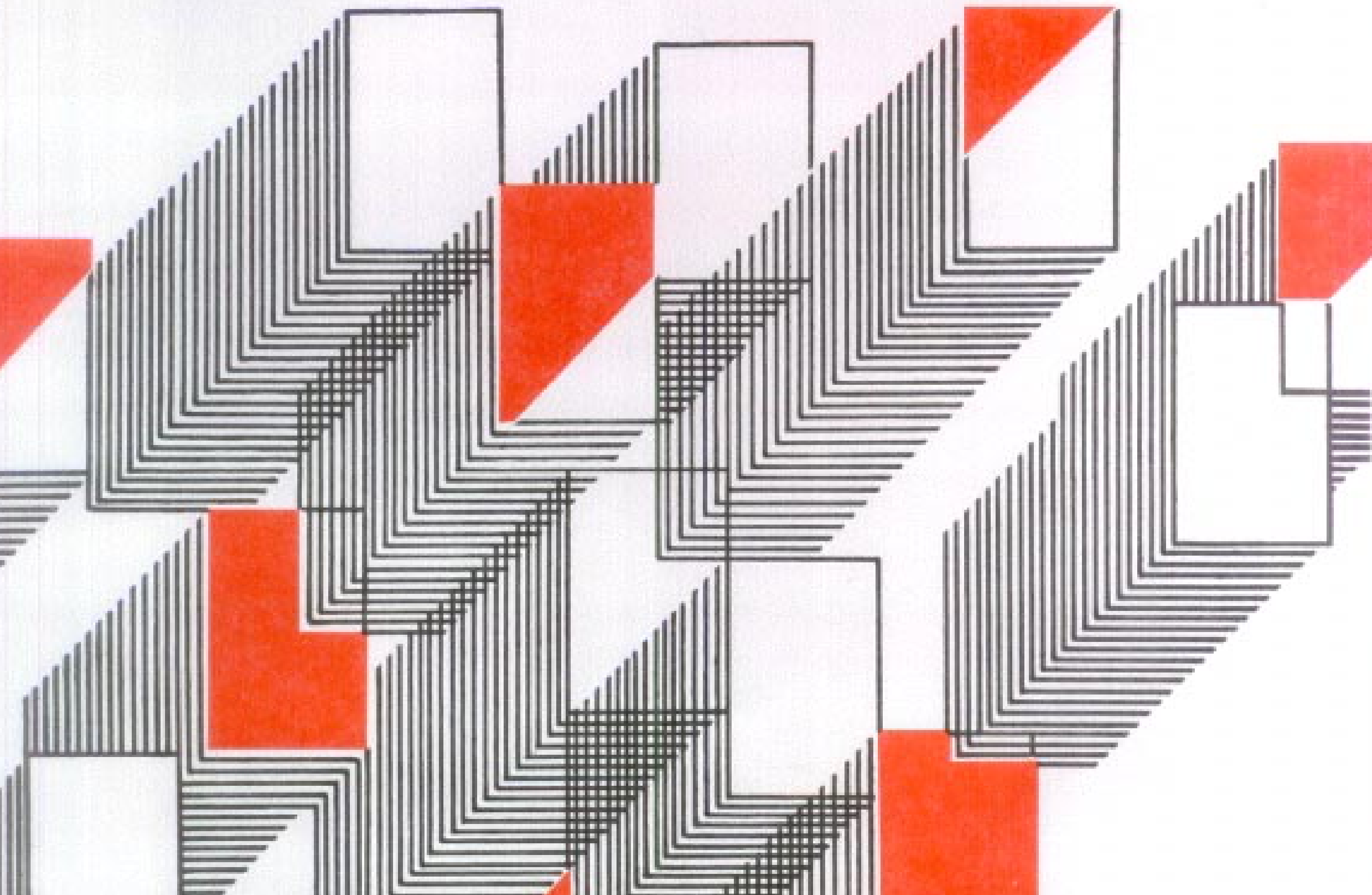
DEFLECTION CALCULATION



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7. DEFLECTION CALCULATION

7.1 EFFECTIVE MOMENT OF INERTIA

A method of calculating the deflections is given in Appendix E of the Code. This method requires the use of an effective moment of inertia I_{eff} given by the following equation

$$I_{eff} = \frac{I_r}{1{\cdot}2 - \frac{M_r}{M}\frac{z}{d}\left(1 - \frac{x}{d}\right)\frac{b_w}{b}}$$

but, $I_r < I_{eff} < I_{gr}$

where

I_r is the moment of inertia of the cracked section;

M_r is the cracking moment, equal to $\frac{f_{cr}I_{gr}}{y_t}$

where

f_{cr} is the modulus of rupture of concrete, I_{gr} is the moment of inertia of the gross section neglecting the reinforcement and y_t is the distance from the centroidal axis of the gross section to the extreme fibre in tension;

M is the maximum moment under service loads;

z is the lever arm;

d is the effective depth;

x is the depth of neutral axis;

b_w is the breadth of the web; and

b is the breadth of the compression face.

The values of x and z are those obtained by elastic theory. Hence $z = d - x/3$ for rectangular sections; also $b = b_w$ for rectangular sections. For flanged sections where the flange is in compression, b will be equal to the flange width b_f. The value of z for flanged beams will depend on the flange dimensions, but in order to simplify the calculations it is conservatively assumed the value of z for flanged beam is also $d - x/3$. With this assumption, the expression effective moment of inertia may be written as follows:

$$\frac{I_{eff}}{I_r} = \frac{1}{1{\cdot}2 - \frac{M_r}{M}\left(1 - \frac{x}{3d}\right)\left(1 - \frac{x}{d}\right)\frac{b_w}{b_f}}$$

but, $\frac{I_{eff}}{I_r} > 1$

and $I_{eff} < I_{gr}$

Chart 89 can be used for finding the value of $\frac{I_{eff}}{I_r}$ in accordance with the above equation. The chart takes into account the condition $\frac{I_{eff}}{I_r} > 1$. After finding the value of I_{eff} it has to be compared with I_{gr} and the lower of the two values should be used for calculating the deflection.

For continuous beams, a weighted average value of I_{eff} should be used, as given in *B-2.1* of the Code.

7.2 SHRINKAGE AND CREEP DEFLECTIONS

Deflections due to shrinkage and creep can be calculated in accordance with clauses *B-3* and *B-4* of the Code. This is illustrated in Example 12.

Example 12 Check for deflection

Calculate the deflection of a cantilever beam of the section designed in Example 3, with further data as given below:

Span of cantilever	4·0 m
Bending moment at service loads	210 kN.m

Sixty percent of the above moment is due to permanent loads, the loading being distributed uniformly on the span.

$$I_{gr} = \frac{bD^3}{12} = \frac{300 \times (600)^3}{12} = 5{\cdot}4 \times 10^9 \text{ mm}^4$$

From clause *5.2.2* of the Code,

Flexural tensile strength,

$$f_{cr} = 0{\cdot}7\sqrt{f_{ck}} \text{ N/mm}^2$$

$$f_{cr} = 0{\cdot}7\sqrt{15} = 2{\cdot}71 \text{ N/mm}^2$$

$$y_t = D/2 = \frac{600}{2} = 300 \text{ mm}$$

$$M_r = \frac{f_{cr}I_{gr}}{y_t} = \frac{2{\cdot}71 \times 5{\cdot}4 \times 10^9}{300}$$

$$= 4{\cdot}88 \times 10^7 \text{ N.mm}$$

$$d'/d = \left(\frac{3{\cdot}75}{56{\cdot}25}\right) = 0{\cdot}067$$

$d'/d = 0{\cdot}05$ will be used in referring to Tables.

From *5.2.3.1* of the Code,

$$E_c = 5\,700\sqrt{f_{ck}} \text{ N/mm}^2$$

$$= 5\,700\sqrt{15} = 22{\cdot}1 \times 10^3 \text{ N/mm}^2$$

$$E_s = 200 \text{ kN/mm}^2 = 2 \times 10^5 \text{ N/mm}^2$$

$$m = E_s/E_c = \frac{2 \times 10^5}{22{\cdot}1 \times 10^3} = 9{\cdot}05$$

From Example 3,

$p_t = 1.117$, $p_c = 0.418$

$p_c(m-1)/(p_t m) = (0.418 \times 8.05)/(1.117 \times 9.05) = 0.333$

$p_t m = 1.117 \times 9.05 = 10.11$

Referring to *Table 87*,

$I_r/(bd^3/12) = 0.720$

$\therefore I_r = 0.720 \times 300 \times (562.5)^3/12 = 3.204 \times 10^9 \text{ mm}^4$

Referring to *Table 91*,

$$\frac{x}{d} = 0.338$$

Moment at service load, $M = 210$ kN.m $= 21.0 \times 10^7$ N.mm

$$M_r/M = \frac{4.88 \times 10^7}{21.0 \times 10^7} = 0.232$$

Referring to *Chart 89*.

$I_{eff}/I_r = 1.0$

$\therefore I_{eff} = I_r = 3.204 \times 10^9 \text{ mm}^4$

For a cantilever with uniformly distributed load,

$$\text{Elastic deflection} = \frac{1}{4}\frac{Ml^2}{EI_{eff}}$$

$$= \frac{21.0 \times 10^7 \times (4000)^2}{4 \times 22.1 \times 10^3 \times 3.204 \times 10^9}$$

$= 11.86$ mm ...(1)

Deflection due to shrinkage (*see* clause *B-3* of the Code):

$a_{cs} = k_3 \, \Psi_{cs} \, l^2$

$k_3 = 0.5$ for cantilevers

$p_t = 1.117$, $p_c = 0.418$

$p_t - p_c = 1.117 - 0.418 = 0.699 < 1.0$

$$\therefore k_4 = 0.72 \times \frac{p_t - p_c}{\sqrt{p_t}}$$

$$= 0.72 \times \frac{(1.117 - 0.418)}{\sqrt{1.117}}$$

$= 0.476$

In the absence of data, the value of the ultimate shrinkage strain ξ_{cs} is taken as 0.000 3 as given in *5.2.4.1* of the Code.

$D = 600$ mm

$$\therefore \text{Shrinkage curvature } \Psi_{cs} = k_4 \frac{\xi_{cs}}{D}$$

$$= \frac{0.476 \times 0.000\,3}{600} = 2.38 \times 10^{-7}$$

$a_{cs} = 0.5 \times 2.38 \times 10^{-7} \times (4\,000)^2$
$= 1.90$ mm ...(2)

Deflection due to creep,

$a_{cc\,(perm)} = a_{icc\,(perm)} - a_{i\,(perm)}$

In the absence of data, the age at loading is assumed to be 28 days and the value of creep coefficient, θ is taken as 1.6 from *5.2.5.1* of the Code.

$$E_{ce} = \frac{E_c}{1+\theta}$$

$$= \frac{22.1 \times 10^3}{1 + 1.6} = 8.5 \times 10^3 \text{ N/mm}^2$$

$$m = \frac{E_s}{E_{ce}} = \frac{2 \times 10^5}{8.5 \times 10^3} = 23.53$$

$p_t = 1.117$, $p_c = 0.418$

$p_c(m-1)/(p_t m) = 0.418(23.53 - 1)/(1.117 \times 23.53) = 0.358$

Referring to *Table 87*,

$I_r/(bd^3/12) = 1.497$

$I_r = 1.497 \times 300\,(562.5)^3/12 = 6.66 \times 10^9 \text{ mm}^4$

$I_r \leqslant I_{eff} \leqslant I_{gr}$

$6.66 \times 10^9 \leqslant I_{eff} \leqslant 5.4 \times 10^9$

$\therefore I_{eff} = 5.4 \times 10^9 \text{ mm}^4$

$a_{icc\,(perm)}$ = Initial plus creep deflection due to permanent loads obtained using the above modulus of elasticity

$$= \frac{1}{4}\frac{Ml^2}{E_{cc}I_{eff}}$$

$$= \frac{1}{4} \times \frac{(0.6 \times 21 \times 10^7)\,(4\,000)^2}{8.5 \times 10^3 \times 5.4 \times 10^9}$$

$= 10.98$ mm

$a_{i\,(perm)}$ = Short term deflection due to permanent load obtained using E_c

$$= \frac{1}{4} \times \frac{(0.6 \times 21 \times 10^7)\,(4\,000)^2}{22.1 \times 10^3 \times 3.204 \times 10^9}$$

$= 7.12$ mm

$\therefore a_{cc(perm)} = 10.98 - 7.12 = 3.86$ ...(3)

$\therefore$ Total deflection (long term) due to initial load, shrinkage and creep

$= 11.86 + 1.90 + 3.86 = 17.62$ mm.

According to *22.2(a)* of the Code the final deflection should not exceed span/250.

$$\text{Permissible deflection} = \frac{4\,000}{250} = 16 \text{ mm.}$$

The calculated deflection is only slightly greater than the permissible value and hence the section may not be revised.

Chart 88 MOMENT OF INERTIA OF T - BEAMS

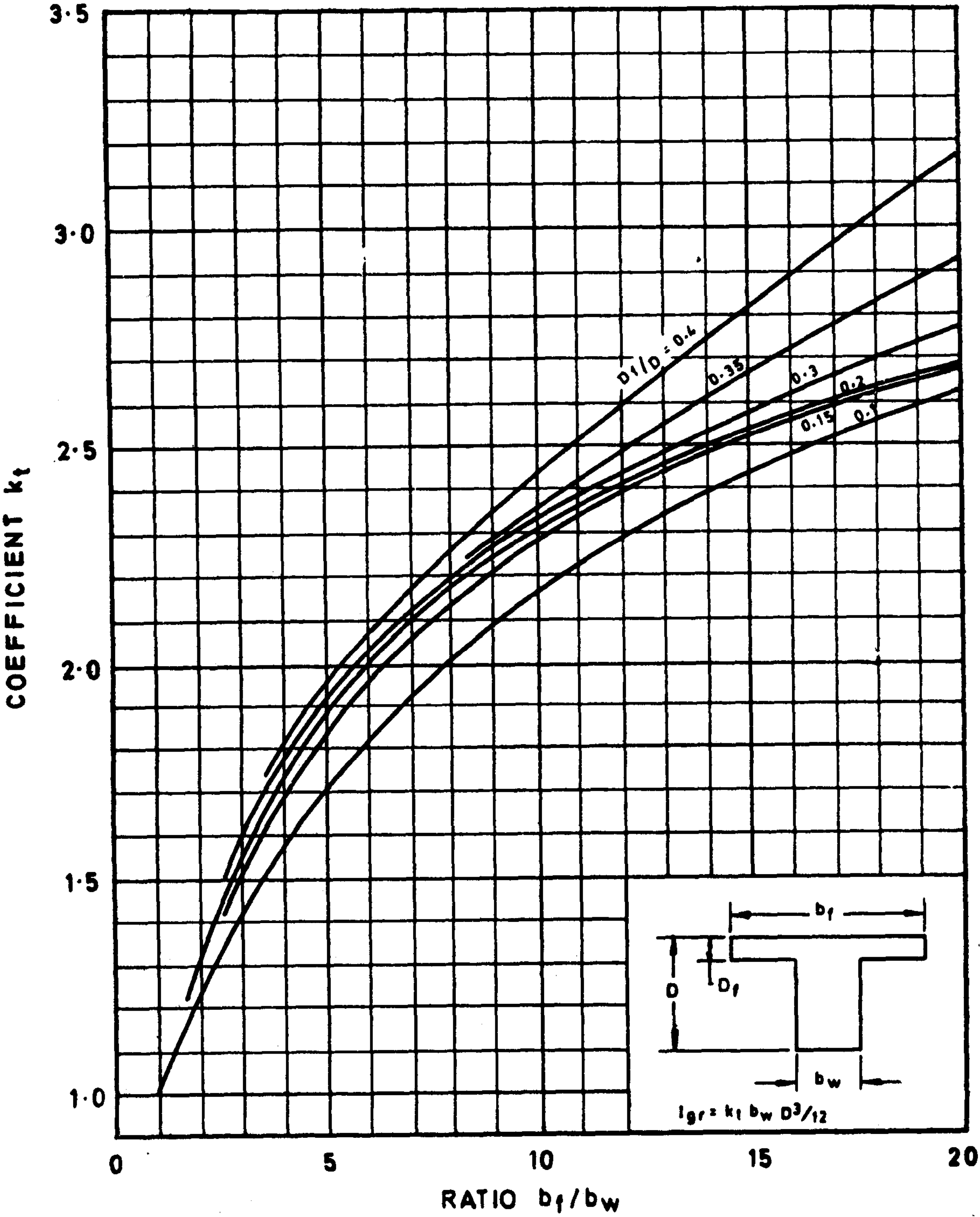


Chart 89 EFFECTIVE MOMENT OF INERTIA FOR CALCULATING DEFLECTION

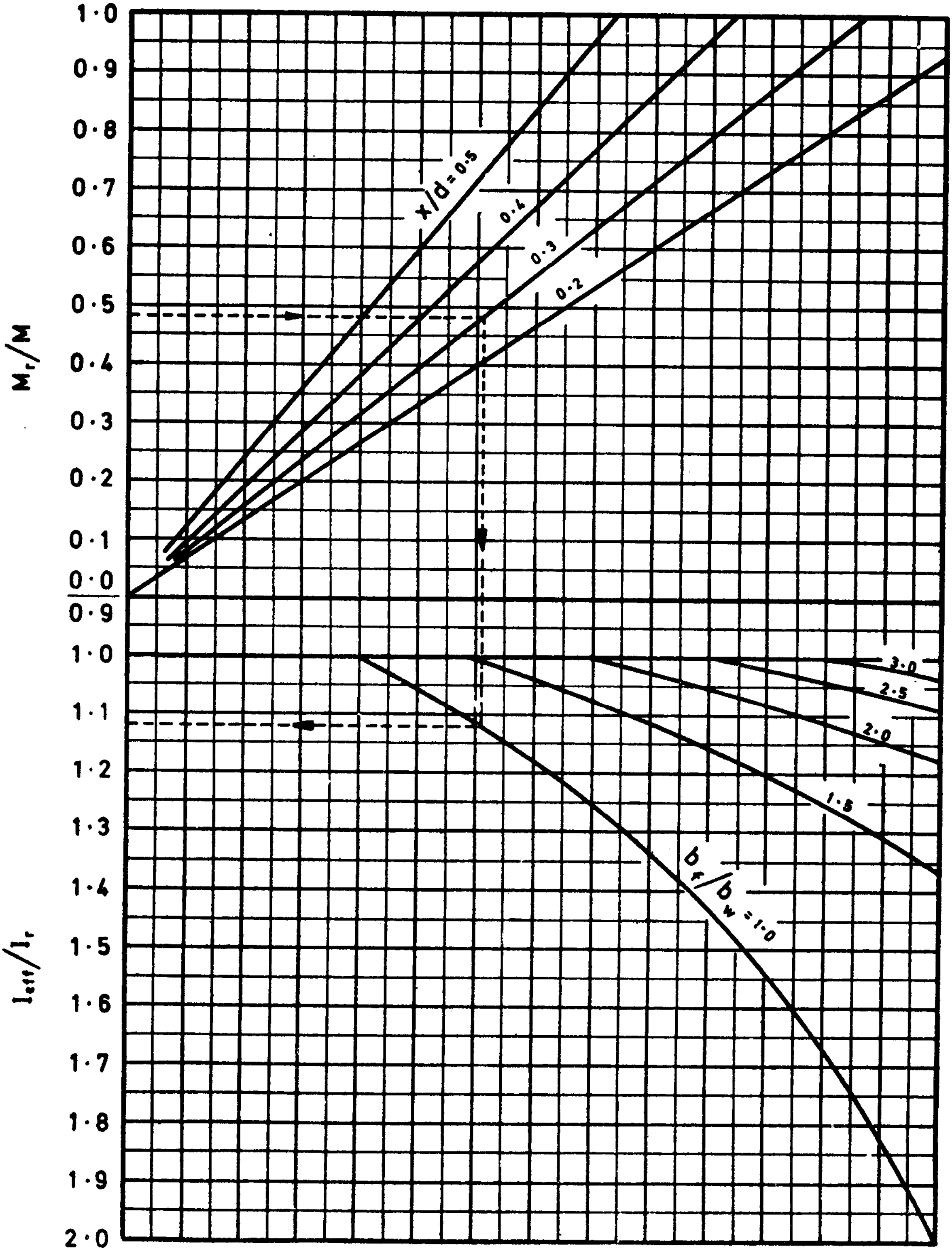
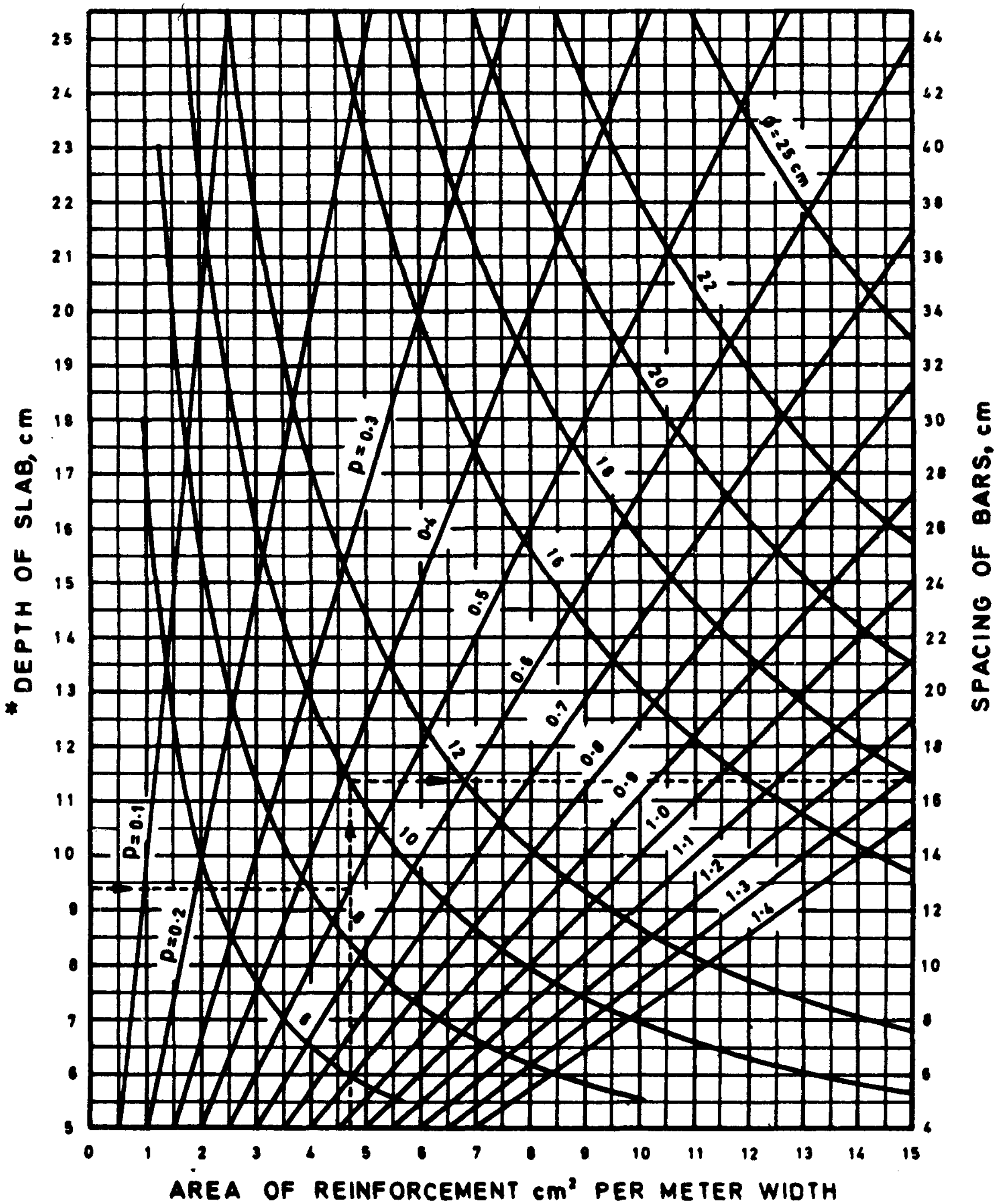
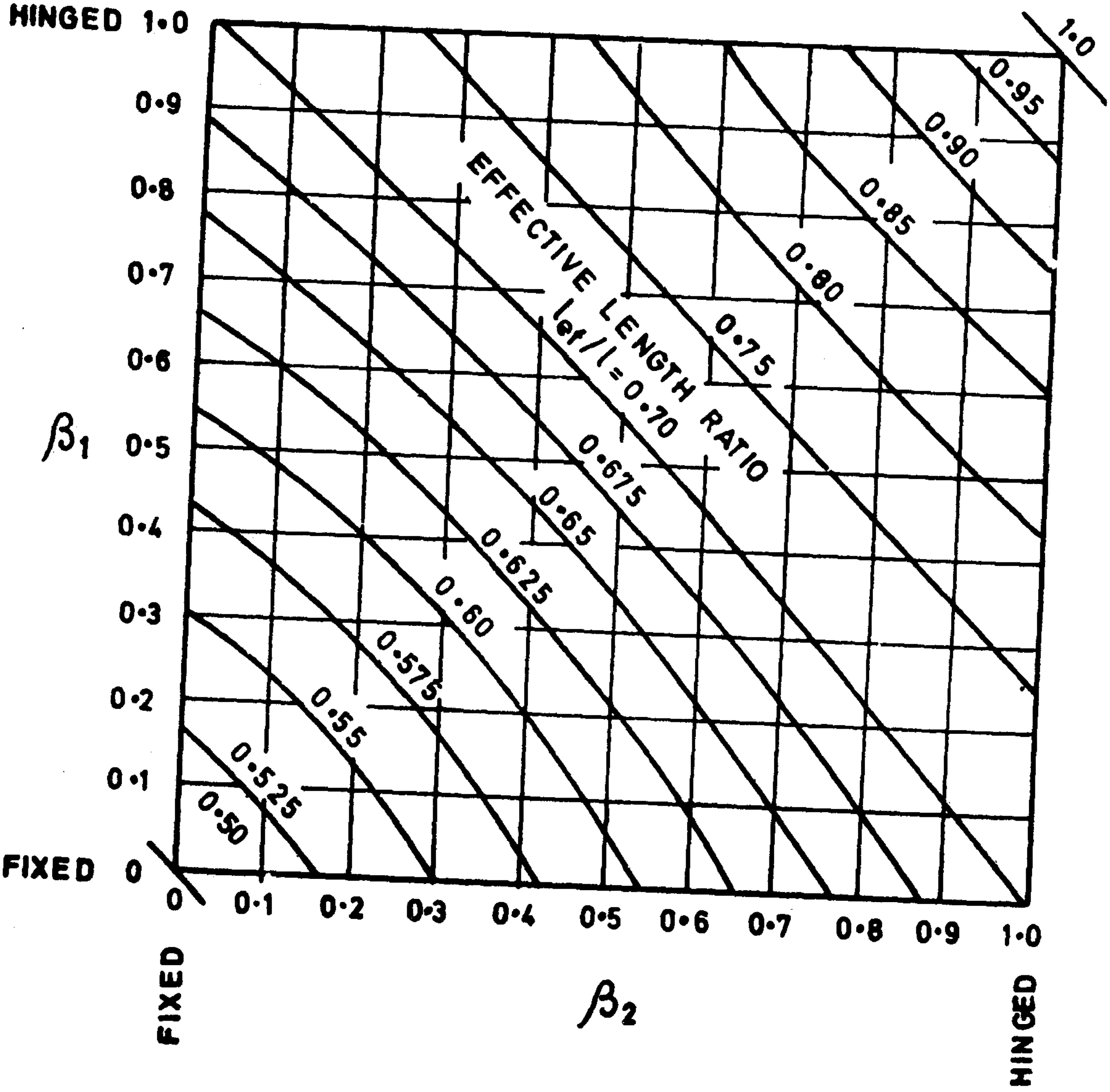


Chart 90 PERCENTAGE, AREA AND SPACING OF BARS IN SLABS



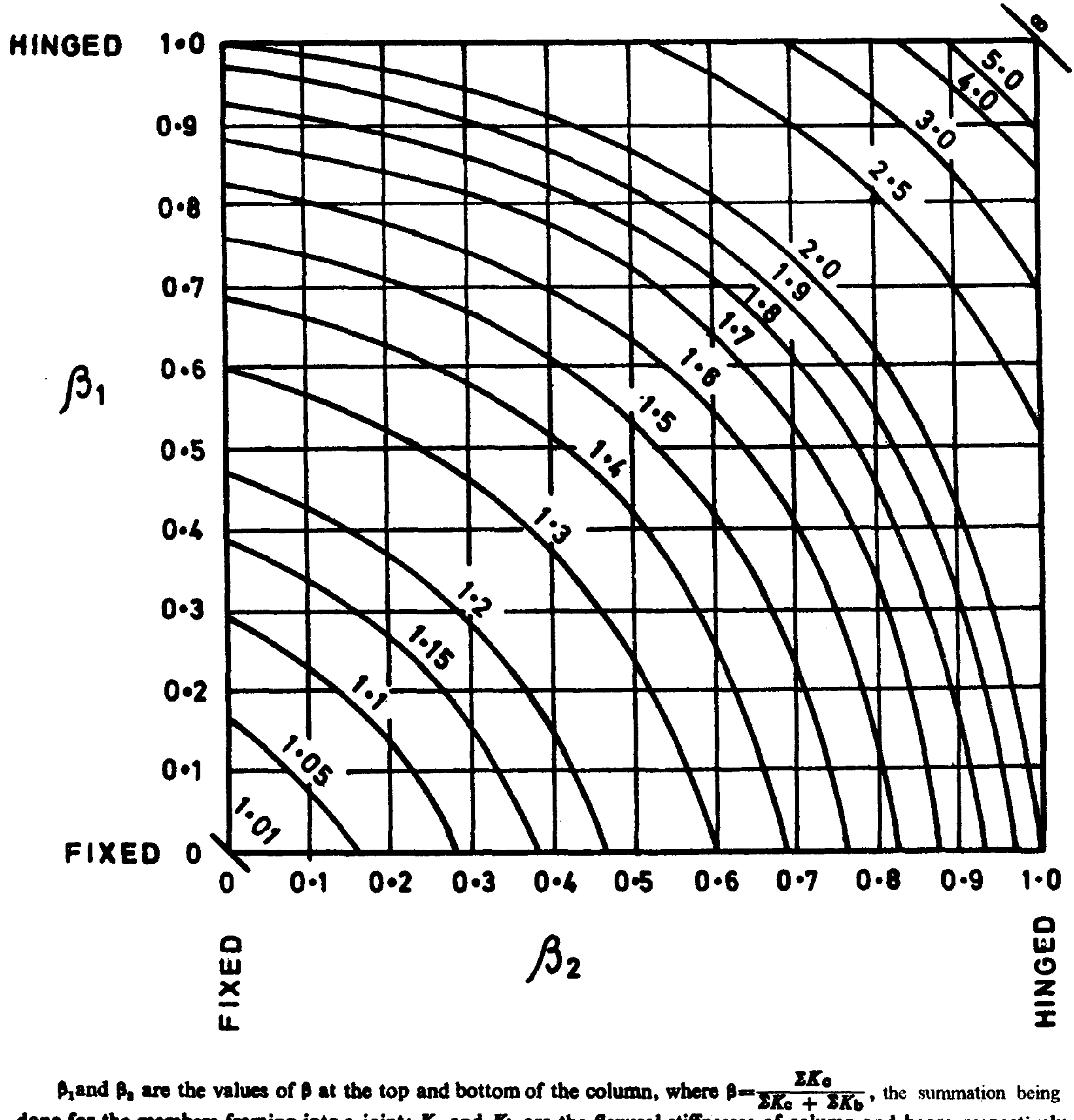
* USE EFFECTIVE DEPTH OR OVERALL WHICHEVER IS USED FOR CALCULATING p

Chart 91 EFFECTIVE LENGTH OF COLUMNS — Frame Restrained Against Sway



β_1 and β_2 are the values of β at the top and bottom of the column where, $\beta = \frac{\Sigma K_c}{\Sigma K_c + \Sigma K_b}$, the summation being done for the members framing into a joint; K_c and K_b are the flexural stiffnesses of column and beam respectively.

Chart 92 EFFECTIVE LENGTH OF COLUMNS — Frame Without Restraint to Sway



β_1 and β_2 are the values of β at the top and bottom of the column, where $\beta = \frac{\Sigma K_c}{\Sigma K_c + \Sigma K_b}$, the summation being done for the members framing into a joint; K_c and K_b are the flexural stiffnesses of column and beam respectively.

TABLE 86 MOMENT OF INERTIA — VALUES OF $bd^3/12\,000$

d, cm	b, cm								
	10	15	20	25	30	35	40	45	50
10	0·8	1·2	1·7	2·1	2·5	2·9	3·3	3·7	4·2
11	1·1	1·7	2·2	2·8	3·3	3·9	4·4	5·0	5·5
12	1·4	2·2	2·9	3·6	4·3	5·0	5·8	6·5	7·2
13	1·8	2·7	3·7	4·6	5·5	6·4	7·3	8·2	9·2
14	2·3	3·4	4·6	5·7	6·9	8·0	9·1	10·3	11·4
15	2·8	4·2	5·6	7·0	8·4	9·8	11·3	12·7	14·1
16	3·4	5·1	6·8	8·5	10·2	11·9	13·7	15·4	17·1
17	4·1	6·1	8·2	10·2	12·3	14·3	16·4	18·4	20·5
18	4·9	7·3	9·7	12·1	14·6	17·0	19·4	21·9	24·3
19	5·7	8·6	11·4	14·3	17·1	20·0	22·9	25·7	28·6
20	6·7	10·0	13·3	16·7	20·0	23·3	26·7	30·0	33·3
21	7·7	11·6	15·4	19·3	23·2	27·0	30·9	34·7	38·6
22	8·9	13·3	17·7	22·2	26·6	31·1	35·5	39·9	44·4
23	10·1	15·2	20·3	25·3	30·4	35·5	40·6	45·6	50·7
24	11·5	17·3	23·0	28·8	34·6	40·3	46·1	51·8	57·6
25	13·0	19·5	26·0	32·6	39·1	45·6	52·1	58·6	65·1
26	14·6	22·0	29·3	36·6	43·9	51·3	58·6	65·9	73·2
27	16·4	24·6	32·8	41·0	49·2	57·4	65·6	73·8	82·0
28	18·3	27·4	36·6	45·7	54·9	64·0	73·2	82·3	91·5
29	20·3	30·5	40·6	50·8	61·0	71·1	81·3	91·5	101·6
30	22·5	33·8	45·0	56·3	67·5	78·8	90·0	101·3	112·5
32	27·3	41·0	54·6	68·3	81·9	95·6	109·2	122·9	136·5
34	32·8	49·1	65·5	81·9	98·3	114·6	131·0	147·4	163·8
36	38·9	58·3	77·8	97·2	116·6	136·1	155·5	175·0	194·4
38	45·7	68·6	91·5	114·3	137·2	160·0	182·9	205·8	228·6
40	53·3	80·0	106·7	133·3	160·0	186·7	213·3	240·0	266·7
42	61·7	92·6	123·5	154·3	185·2	216·1	247·0	277·8	308·7
44	71·0	106·5	142·0	177·5	213·0	248·5	283·9	319·4	354·9
46	81·1	121·7	162·2	202·8	243·3	283·9	324·5	365·0	405·6
48	92·2	138·2	184·3	230·4	276·5	322·6	368·6	414·7	460·8
50	104·2	156·2	208·3	260·4	312·5	364·6	416·7	468·7	520·8
52	117·2	175·8	234·3	292·9	351·5	410·1	468·7	527·3	585·9
54	131·2	196·8	262·4	328·0	393·7	459·3	524·9	590·5	656·1
56	146·3	219·5	292·7	365·9	439·0	512·2	585·4	658·6	731·7
58	162·6	243·9	325·2	406·5	487·8	569·1	650·4	731·7	813·0
60	180·0	270·0	360·0	450·0	540·0	630·0	720·0	810·0	900·0
65	228·9	343·3	457·7	572·1	686·6	801·0	915·4	1029·8	1144·3
70	285·8	428·7	571·7	714·6	857·5	1000·4	1143·3	1286·2	1429·2
75	351·6	527·3	703·1	878·9	1054·7	1230·5	1406·3	1582·0	1757·8
80	426·7	640·0	853·3	1066·7	1280·0	1493·3	1706·7	1920·0	2133·3
85	511·8	767·7	1023·5	1279·4	1535·3	1791·2	2047·1	2303·0	2558·9
90	607·5	911·3	1215·0	1518·8	1822·5	2126·3	2430·0	2733·8	3037·5
95	714·5	1071·7	1429·0	1786·2	2143·4	2500·7	2857·9	3215·2	3572·4
100	833·3	1250·0	1666·7	2083·3	2500·0	2916·7	3333·3	3750·0	4166·7

TABLE 87 MOMENT OF INERTIA OF CRACKED SECTION — VALUES OF $I_r/\left(\frac{bd^3}{12}\right)$

$d'/d = 0{\cdot}05$

$p_t m$	$p_c(m-1)/(p_t m)$							
	0·0	0·1	0·2	0·3	0·4	0·6	0·8	1·0
1·0	0·100	0·100	0·100	0·100	0·100	0·100	0·100	0·100
1·5	0·143	0·144	0·144	0·144	0·144	0·145	0·145	0·145
2·0	0·185	0·185	0·186	0·186	0·186	0·187	0·188	0·188
2·5	0·224	0·225	0·225	0·226	0·227	0·228	0·229	0·230
3·0	0·262	0·263	0·264	0·264	0·265	0·267	0·269	0·270
3·5	0·298	0·299	0·300	0·302	0·303	0·305	0·308	0·310
4·0	0·332	0·334	0·336	0·338	0·339	0·343	0·346	0·348
4·5	0·366	0·368	0·371	0·373	0·375	0·379	0·383	0·386
5·0	0·398	0·401	0·404	0·407	0·409	0·414	0·419	0·424
5·5	0·430	0·433	0·437	0·440	0·443	0·449	0·455	0·460
6·0	0·460	0·465	0·469	0·472	0·476	0·483	0·490	0·496
6·5	0·490	0·495	0·500	0·504	0·509	0·517	0·525	0·532
7·0	0·519	0·525	0·530	0·535	0·540	0·550	0·559	0·567
7·5	0·547	0·554	0·560	0·566	0·571	0·582	0·592	0·602
8·0	0·575	0·582	0·589	0·596	0·602	0·614	0·626	0·636
8·5	0·601	0·610	0·617	0·625	0·632	0·646	0·659	0·670
9·0	0·628	0·637	0·645	0·654	0·662	0·677	0·691	0·704
9·5	0·653	0·663	0·673	0·682	0·691	0·708	0·723	0·738
10·0	0·678	0·689	0·700	0·710	0·720	0·738	0·755	0·771
10·5	0·703	0·715	0·727	0·738	0·748	0·769	0·787	0·804
11·0	0·727	0·740	0·753	0·765	0·777	0·798	0·818	0·837
11·5	0·750	0·764	0·778	0·792	0·804	0·828	0·850	0·869
12·0	0·773	0·789	0·804	0·818	0·832	0·857	0·880	0·902
12·5	0·795	0·812	0·829	0·844	0·859	0·886	0·911	0·934
13·0	0·818	0·836	0·853	0·870	0·885	0·915	0·942	0·966
13·5	0·839	0·859	0·877	0·895	0·912	0·943	0·972	0·998
14·0	0·860	0·881	0·901	0·920	0·938	0·972	1·002	1·030
14·5	0·881	0·904	0·925	0·945	0·964	1·000	1·032	1·061
15·0	0·902	0·926	0·948	0·969	0·990	1·027	1·062	1·093
15·5	0·922	0·947	0·971	0·994	1·015	1·055	1·091	1·124
16·0	0·942	0·968	0·994	1·018	1·040	1·083	1·121	1·155
17·0	0·980	1·010	1·038	1·065	1·090	1·137	1·179	1·217
18·0	1·018	1·051	1·082	1·111	1·139	1·191	1·237	1·278
19·0	1·054	1·090	1·125	1·157	1·188	1·244	1·294	1·340
20·0	1·089	1·129	1·166	1·202	1·235	1·296	1·351	1·400
21·0	1·123	1·167	1·207	1·246	1·282	1·348	1·408	1·461
22·0	1·156	1·203	1·248	1·289	1·328	1·400	1·464	1·521
23·0	1·188	1·239	1·287	1·332	1·374	1·451	1·519	1·581
24·0	1·220	1·274	1·326	1·374	1·419	1·502	1·575	1·640
25·0	1·250	1·309	1·364	1·415	1·464	1·552	1·630	1·699
26·0	1·280	1·342	1·401	1·456	1·508	1·602	1·685	1·758
27·0	1·308	1·376	1·438	1·497	1·552	1·651	1·739	1·817
28·0	1·337	1·408	1·475	1·537	1·595	1·701	1·794	1·876
29·0	1·364	1·440	1·510	1·576	1·638	1·750	1·848	1·934
30·0	1·391	1·471	1·546	1·615	1·681	1·798	1·902	1·993

TABLE 88 MOMENT OF INERTIA OF CRACKED SECTION — VALUES OF $I_r/\left(\frac{bd^3}{12}\right)$

$d'/d = 0.10$

$p_t m$	$p_c(m-1)/(p_t m)$ 0·0	0·1	0·2	0·3	0·4	0·6	0·8	1·0
1·0	0·100	0·100	0·100	0·100	0·100	0·100	0·100	0·100
1·5	0·143	0·143	0·144	0·144	0·144	0·144	0·144	0·144
2·0	0·185	0·185	0·185	0·185	0·185	0·186	0·186	0·186
2·5	0·224	0·224	0·225	0·225	0·225	0·226	0·226	0·227
3·0	0·262	0·262	0·263	0·263	0·263	0·264	0·265	0·266
3·5	0·298	0·298	0·299	0·300	0·300	0·302	0·303	0·304
4·0	0·332	0·333	0·334	0·335	0·336	0·338	0·340	0·341
4·5	0·366	0·367	0·369	0·370	0·371	0·373	0·376	0·378
5·0	0·398	0·400	0·402	0·403	0·405	0·408	0·411	0·413
5·5	0·430	0·432	0·434	0·436	0·438	0·442	0·445	0·448
6·0	0·460	0·463	0·466	0·468	0·470	0·475	0·479	0·483
6·5	0·490	0·493	0·496	0·499	0·502	0·507	0·512	0·517
7·0	0·519	0·523	0·526	0·530	0·533	0·539	0·545	0·550
7·5	0·547	0·551	0·556	0·560	0·563	0·571	0·577	0·583
8·0	0·575	0·580	0·584	0·589	0·593	0·601	0·609	0·616
8·5	0·601	0·607	0·612	0·618	0·622	0·632	0·640	0·648
9·0	0·628	0·634	0·640	0·646	0·651	0·662	0·671	0·680
9·5	0·653	0·660	0·667	0·673	0·680	0·691	0·702	0·712
10·0	0·678	0·686	0·693	0·701	0·708	0·720	0·732	0·743
10·5	0·703	0·711	0·720	0·727	0·735	0·749	0·762	0·774
11·0	0·727	0·736	0·745	0·754	0·762	0·778	0·792	0·805
11·5	0·750	0·760	0·770	0·780	0·789	0·806	0·822	0·836
12·0	0·773	0·784	0·795	0·805	0·815	0·834	0·851	0·866
12·5	0·795	0·808	0·820	0·831	0·841	0·861	0·880	0·896
13·0	0·818	0·831	0·844	0·856	0·867	0·889	0·908	0·926
13·5	0·839	0·854	0·867	0·880	0·893	0·916	0·937	0·956
14·0	0·860	0·876	0·891	0·905	0·918	0·943	0·965	0·986
14·5	0·881	0·898	0·914	0·929	0·943	0·969	0·993	1·015
15·0	0·902	0·920	0·936	0·952	0·968	0·996	1·021	1·044
15·5	0·922	0·941	0·959	0·976	0·992	1·022	1·049	1·074
16·0	0·942	0·962	0·981	0·999	1·016	1·048	1·077	1·103
17·0	0·980	1·003	1·024	1·045	1·064	1·099	1·131	1·160
18·0	1·018	1·043	1·067	1·089	1·111	1·150	1·185	1·217
19·0	1·054	1·082	1·108	1·133	1·157	1·200	1·239	1·274
20·0	1·089	1·120	1·149	1·176	1·202	1·249	1·292	1·330
21·0	1·123	1·157	1·189	1·218	1·247	1·298	1·344	1·386
22·0	1·156	1·193	1·227	1·260	1·291	1·347	1·396	1·441
23·0	1·188	1·228	1·266	1·301	1·334	1·394	1·448	1·496
24·0	1·220	1·263	1·303	1·341	1·377	1·442	1·500	1·551
25·0	1·250	1·296	1·340	1·381	1·419	1·489	1·551	1·606
26·0	1·280	1·329	1·376	1·420	1·461	1·535	1·601	1·660
27·0	1·308	1·362	1·412	1·458	1·502	1·582	1·652	1·714
28·0	1·337	1·394	1·447	1·496	1·543	1·627	1·702	1·768
29·0	1·364	1·425	1·481	1·534	1·583	1·673	1·752	1·821
30·0	1·391	1·455	1·515	1·571	1·623	1·718	1·801	1·875

TABLE 89 MOMENT OF INERTIA OF CRACKED SECTION — VALUES OF $I_r / \left(\frac{bd^3}{12}\right)$

$d'/d = 0{\cdot}15$

$p_t m$	$P_c(m-1)/(p_t m)$ 0·0	0·1	0·2	0·3	0·4	0·6	0·8	1·0
1·0	0·100	0·100	0·100	0·100	0·100	0·100	0·100	0·100
1·5	0·143	0·143	0·143	0·143	0·143	0·143	0·143	0·143
2·0	0·185	0·185	0·185	0·185	0·185	0·185	0·185	0·185
2·5	0·224	0·224	0·224	0·224	0·224	0·224	0·225	0·225
3·0	0·262	0·262	0·262	0·262	0·262	0·262	0·263	0·263
3·5	0·298	0·298	0·298	0·298	0·299	0·299	0·300	0·300
4·0	0·332	0·333	0·333	0·334	0·334	0·335	0·336	0·336
4·5	0·366	0·367	0·367	0·368	0·368	0·369	0·371	0·372
5·0	0·398	0·399	0·400	0·401	0·402	0·403	0·405	0·406
5·5	0·430	0·431	0·432	0·433	0·434	0·436	0·438	0·440
6·0	0·460	0·462	0·463	0·465	0·466	0·468	0·471	0·473
6·5	0·490	0·492	0·494	0·495	0·497	0·500	0·503	0·505
7·0	0·519	0·521	0·523	0·525	0·527	0·531	0·534	0·537
7·5	0·547	0·550	0·552	0·555	0·557	0·561	0·565	0·569
8·0	0·575	0·578	0·581	0·583	0·586	0·591	0·596	0·600
8·5	0·601	0·605	0·608	0·611	0·614	0·620	0·626	0·631
9·0	0·628	0·632	0·635	0·639	0·643	0·649	0·655	0·661
9·5	0·653	0·658	0·662	0·666	0·670	0·678	0·685	0·691
10·0	0·678	0·683	0·688	0·693	0·697	0·706	0·713	0·721
10·5	0·703	0·708	0·714	0·719	0·724	0·733	0·742	0·750
11·0	0·727	0·733	0·739	0·745	0·750	0·761	0·770	0·779
11·5	0·750	0·757	0·764	0·770	0·776	0·788	0·798	0·808
12·0	0·773	0·781	0·788	0·795	0·802	0·814	0·826	0·836
12·5	0·795	0·804	0·812	0·820	0·827	0·841	0·853	0·865
13·0	0·818	0·827	0·836	0·844	0·852	0·867	0·880	0·893
13·5	0·839	0·849	0·859	0·868	0·876	0·893	0·907	0·921
14·0	0·860	0·871	0·882	0·891	0·901	0·918	0·934	0·948
14·5	0·881	0·893	0·904	0·915	0·925	0·943	0·960	0·976
15·0	0·902	0·914	0·926	0·938	0·949	0·969	0·987	1·003
15·5	0·922	0·935	0·948	0·960	0·972	0·993	1·013	1·030
16·0	0·942	0·956	0·970	0·983	0·995	1·018	1·039	1·057
17·0	0·980	0·997	1·012	1·027	1·041	1·067	1·090	1·111
18·0	1·018	1·036	1·054	1·070	1·086	1·115	1·141	1·164
19·0	1·054	1·075	1·094	1·112	1·130	1·162	1·191	1·216
20·0	1·089	1·112	1·134	1·154	1·173	1·208	1·240	1·268
21·0	1·123	1·148	1·172	1·194	1·216	1·254	1·289	1·320
22·0	1·156	1·184	1·210	1·234	1·257	1·300	1·337	1·371
23·0	1·188	1·219	1·247	1·274	1·299	1·345	1·385	1·422
24·0	1·220	1·252	1·283	1·312	1·339	1·389	1·433	1·473
25·0	1·250	1·286	1·319	1·350	1·379	1·433	1·480	1·523
26·0	1·280	1·318	1·354	1·387	1·419	1·476	1·527	1·573
27·0	1·308	1·350	1·388	1·424	1·458	1·520	1·574	1·622
28·0	1·337	1·381	1·422	1·461	1·497	1·562	1·620	1·672
29·0	1·364	1·411	1·455	1·496	1·535	1·605	1·666	1·721
30·0	1·391	1·441	1·488	1·532	1·573	1·647	1·712	1·770

TABLE 90 MOMENT OF INERTIA OF CRACKED SECTION — VALUES OF $I_r / \left(\frac{bd^3}{12} \right)$

$d'/d = 0{\cdot}20$

$p_t m$	$p_c(m-1)/(p_t m)$ 0·0	0·1	0·2	0·3	0·4	0·6	0·8	1·0
1·0	0·100	0·100	0·100	0·100	0·100	0·100	0·100	0·100
1·5	0·143	0·143	0·143	0·143	0·144	0·144	0·144	0·144
2·0	0·185	0·185	0·185	0·185	0·185	0·185	0·185	0·185
2·5	0·224	0·224	0·224	0·224	0·224	0·224	0·224	0·224
3·0	0·262	0·262	0·262	0·262	0·262	0·262	0·262	0·262
3·5	0·298	0·298	0·298	0·298	0·298	0·298	0·298	0·298
4·0	0·332	0·333	0·333	0·333	0·333	0·333	0·333	0·333
4·5	0·366	0·366	0·366	0·367	0·367	0·367	0·367	0·368
5·0	0·398	0·399	0·399	0·399	0·400	0·400	0·401	0·401
5·5	0·430	0·430	0·431	0·431	0·432	0·432	0·433	0·434
6·0	0·460	0·461	0·462	0·462	0·463	0·464	0·465	0·466
6·5	0·490	0·491	0·492	0·492	0·493	0·495	0·496	0·497
7·0	0·519	0·520	0·521	0·522	0·523	0·525	0·526	0·528
7·5	0·547	0·548	0·550	0·551	0·552	0·554	0·556	0·558
8·0	0·575	0·576	0·578	0·579	0·580	0·583	0·586	0·588
8·5	0·601	0·603	0·605	0·607	0·608	0·612	0·614	0·617
9·0	0·628	0·630	0·632	0·634	0·636	0·639	0·643	0·646
9·5	0·653	0·656	0·658	0·660	0·663	0·667	0·671	0·675
10·0	0·678	0·681	0·684	0·687	0·689	0·694	0·699	0·703
10·5	0·703	0·706	0·709	0·712	0·715	0·721	0·726	0·731
11·0	0·727	0·730	0·734	0·737	0·741	0·747	0·753	0·758
11·5	0·750	0·754	0·758	0·762	0·766	0·773	0·779	0·785
12·0	0·773	0·778	0·782	0·787	0·791	0·799	0·806	0·812
12·5	0·795	0·801	0·806	0·811	0·815	0·824	0·832	0·839
13·0	0·818	0·823	0·829	0·834	0·839	0·849	0·857	0·865
13·5	0·839	0·846	0·852	0·858	0·863	0·874	0·883	0·892
14·0	0·860	0·867	0·874	0·881	0·887	0·898	0·908	0·918
14·5	0·881	0·889	0·896	0·903	0·910	0·922	0·933	0·943
15·0	0·902	0·910	0·918	0·926	0·933	0·946	0·958	0·969
15·5	0·922	0·931	0·940	0·948	0·955	0·970	0·983	0·994
16·0	0·942	0·952	0·961	0·969	0·978	0·993	1·007	1·020
17·0	0·980	0·992	1·002	1·012	1·022	1·039	1·055	1·070
18·0	1·018	1·031	1·043	1·054	1·065	1·085	1·103	1·119
19·0	1·054	1·068	1·082	1·095	1·107	1·129	1·150	1·168
20·0	1·089	1·105	1·120	1·135	1·148	1·173	1·196	1·216
21·0	1·123	1·141	1·158	1·174	1·189	1·217	1·241	1·264
22·0	1·156	1·176	1·195	1·212	1·229	1·259	1·287	1·311
23·0	1·188	1·210	1·231	1·250	1·268	1·302	1·331	1·358
24·0	1·220	1·244	1·266	1·287	1·307	1·343	1·376	1·405
25·0	1·250	1·276	1·301	1·324	1·345	1·384	1·419	1·451
26·0	1·280	1·308	1·334	1·359	1·383	1·425	1·463	1·497
27·0	1·308	1·339	1·368	1·395	1·420	1·465	1·506	1·542
28·0	1·337	1·370	1·400	1·429	1·456	1·505	1·549	1·587
29·0	1·364	1·399	1·433	1·463	1·492	1·545	1·591	1·632
30·0	1·391	1·429	1·464	1·497	1·528	1·584	1·633	1·677

TABLE 91 DEPTH OF NEUTRAL AXES — VALUES OF x/d BY ELASTIC THEORY

$d'/d = 0{\cdot}05$

$p_t m$	$p_c(m-1)/(p_t m)$							
	0·0	0·1	0·2	0·3	0·4	0·6	0·8	1·0
1·0	0·132	0·131	0·131	0·130	0·130	0·128	0·127	0·126
1·5	0·159	0·158	0·157	0·156	0·155	0·153	0·152	0·150
2·0	0·181	0·180	0·178	0·177	0·176	0·173	0·171	0·169
2·5	0·200	0·198	0·197	0·195	0·194	0·190	0·187	0·185
3·0	0·217	0·215	0·213	0·211	0·209	0·205	0·202	0·198
3·5	0·232	0·230	0·227	0·225	0·223	0·218	0·214	0·210
4·0	0·246	0·243	0·240	0·238	0·235	0·230	0·225	0·221
4·5	0·258	0·255	0·252	0·249	0·246	0·241	0·235	0·230
5·0	0·270	0·267	0·263	0·260	0·257	0·251	0·245	0·239
5·5	0·281	0·277	0·274	0·270	0·267	0·260	0·253	0·247
6·0	0·292	0·287	0·284	0·280	0·276	0·268	0.261	0·255
6·5	0·301	0·297	0·293	0·288	0·284	0·276	0.269	0·262
7·0	0·311	0·306	0·301	0·297	0·292	0·284	0.276	0·268
7·5	0·319	0·314	0·309	0·305	0·300	0·291	0.282	0·274
8·0	0·328	0·323	0·317	0·312	0·307	0·298	0.289	0·280
8·5	0·336	0·330	0·325	0·319	0·314	0·304	0·294	0·285
9·0	0·344	0·338	0·332	0·326	0·321	0·310	0·300	0·291
9·5	0·351	0·345	0·339	0·333	0·327	0·316	0·305	0·295
10·0	0·358	0·352	0·345	0·339	0·333	0·321	0·310	0·300
10·5	0·365	0·358	0·351	0·345	0·339	0·326	0·315	0·304
11·0	0·372	0·365	0·358	0·351	0·344	0·332	0·320	0·309
11·5	0·378	0·371	0·363	0·356	0·349	0·336	0·324	0·313
12·0	0·384	0·377	0·369	0·362	0·355	0·341	0·328	0·316
12·5	0·390	0·382	0·374	0·367	0·359	0·345	0·332	0·320
13·0	0·396	0·388	0·380	0·372	0·364	0·350	0·336	0·324
13·5	0·402	0·393	0·385	0·377	0·369	0·354	0·340	0·327
14·0	0·407	0·398	0·390	0·381	0·373	0·358	0·344	0·330
14·5	0·413	0·403	0·394	0·386	0·378	0·362	0·347	0·333
15·0	0·418	0·408	0·399	0·390	0·382	0·365	0·350	0·336
15·5	0·423	0·413	0·404	0·395	0·386	0·369	0·354	0·339
16·0	0·428	0·418	0·408	0·399	0·390	0·373	0·357	0·342
17·0	0·437	0·427	0·416	0·407	0·397	0·379	0·363	0·347
18·0	0·446	0·435	0·425	0·414	0·404	0·386	0·368	0·352
19·0	0·455	0·443	0·432	0·421	0·411	0·392	0·374	0·357
20·0	0·463	0·451	0·439	0·428	0·417	0·397	0·379	0·362
21·0	0·471	0·459	0·446	0·435	0·424	0·403	0·383	0·366
22·0	0·479	0·466	0·453	0·441	0·429	0·408	0·388	0·370
23·0	0·486	0·472	0·459	0·447	0·435	0·413	0·392	0·373
24·0	0·493	0·479	0·465	0·453	0·440	0·417	0·396	0·377
25·0	0·500	0·485	0·471	0·458	0·445	0·422	0·400	0·380
26·0	0·507	0·491	0·477	0·463	0·450	0·426	0·404	0·384
27·0	0·513	0·497	0·482	0·468	0·455	0·430	0·407	0·387
28·0	0·519	0·503	0·488	0·473	0·459	0·434	0·411	0·390
29·0	0·525	0·508	0·493	0·478	0·464	0·437	0·414	0·392
30·0	0·531	0·514	0·498	0·482	0·468	0·441	0·417	0·395

TABLE 92 DEPTH OF NEUTRAL AXIS — VALUES OF x/d BY ELASTIC THEORY

$d'/d = 0{\cdot}10$

$p_t m$	$p_c(m-1)/(p_t m)$							
	0·0	0·1	0·2	0·3	0·4	0·6	0·8	1·0
1·0	0·132	0·132	0·131	0·131	0·131	0·130	0·130	0·130
1·5	0·159	0·158	0·158	0·157	0·157	0·156	0·155	0·154
2·0	0·181	0·180	0·179	0·179	0·178	0·176	0·175	0·174
2·5	0·200	0·199	0·198	0·197	0·196	0·194	0·192	0·190
3·0	0·217	0·215	0·214	0·213	0·211	0·209	0·206	0·204
3·5	0·232	0·230	0·228	0·227	0·225	0·222	0·219	0·216
4·0	0·246	0·244	0·242	0·240	0·238	0·234	0·231	0·227
4·5	0·258	0·256	0·254	0·252	0·249	0·245	0·241	0·237
5·0	0·270	0·268	0·265	0·262	0·260	0·255	0·251	0·246
5·5	0·281	0·278	0·275	0·273	0·270	0·265	0·260	0·255
6·0	0·292	0·288	0·285	0·282	0·279	0·273	0·268	0·263
6·5	0·301	0·298	0·294	0·291	0·288	0·282	0·276	0·270
7·0	0·311	0·307	0·303	0·299	0·296	0·289	0·283	0·277
7·5	0·319	0·315	0·311	0·307	0·304	0·296	0·290	0·283
8·0	0·328	0·324	0·319	0·315	0·311	0·303	0·296	0·289
8·5	0·336	0·331	0·327	0·322	0·318	0·310	0·302	0·295
9·0	0·344	0·339	0·334	0·329	0·325	0·316	0·308	0·300
9·5	0·351	0·346	0·341	0·336	0·331	0·322	0·313	0·305
10·0	0·358	0·353	0·347	0·342	0·337	0·327	0·318	0·310
10·5	0·365	0·359	0·354	0·348	0·343	0·333	0·323	0·314
11·0	0·372	0·366	0·360	0·354	0·349	0·338	0·328	0·319
11·5	0·378	0·372	0·366	0·360	0·354	0·343	0·333	0·323
12·0	0·384	0·378	0·371	0·365	0·359	0·348	0·337	0·327
12·5	0·390	0·383	0·377	0·370	0·364	0·352	0·341	0·331
13·0	0·396	0·389	0·382	0·375	0·369	0·357	0·345	0·335
13·5	0·402	0·394	0·387	0·380	0·374	0·361	0·349	0.338
14·0	0·407	0·400	0·392	0·385	0·378	0·365	0·353	0.342
14·5	0·413	0·405	0·397	0·390	0·382	0·369	0·357	0.345
15·0	0·418	0·410	0·402	0·394	0·387	0·373	0·360	0.348
15·5	0·423	0·414	0·406	0·398	0·391	0·377	0·363	0.351
16·0	0·428	0·419	0·411	0·403	0·395	0·380	0·367	0·354
17·0	0·437	0·428	0·419	0·411	0·403	0·387	0·373	0·360
18·0	0·446	0·437	0·427	0·418	0·410	0·394	0·379	0·365
19·0	0·455	0·445	0·435	0·426	0·417	0·400	0·384	0·370
20·0	0·463	0·453	0·442	0·433	0·423	0·406	0·389	0·375
21·0	0·471	0·460	0·449	0·439	0·429	0.411	0·394	0·379
22·0	0·479	0·467	0·456	0·445	0·435	0.416	0·399	0·383
23·0	0·486	0·474	0·462	0·451	0·441	0.421	0·403	0·387
24·0	0·493	0·481	0·469	0·457	0·446	0.426	0·408	0·391
25·0	0·500	0·487	0·475	0·463	0·452	0.431	0·412	0·394
26·0	0·507	0·493	0·480	0·468	0·457	0·435	0·416	0·398
27·0	0·513	0·499	0·486	0·473	0·461	0·439	0·419	0·401
28·0	0·519	0·505	0·491	0·478	0·466	0·443	0·423	0·404
29·0	0·525	0·510	0·496	0·483	0·470	0·447	0·426	0·407
30·0	0·531	0·516	0·501	0·488	0·475	0·451	0·429	0·410

TABLE 93 DEPTH OF NEUTRAL AXIS – VALUES OF x/d BY ELASTIC THEORY

$d'/d = 0{\cdot}15$

$p_t m$	$p_c(m-1)/(p_t m)$ 0·0	0·1	0·2	0·3	0·4	0·6	0·8	1·0
1·0	0·132	0·132	0·132	0·132	0·132	0·133	0·133	0·133
1·5	0·159	0·159	0·159	0·159	0·159	0·158	0·158	0·158
2·0	0·181	0·181	0·180	0·180	0·180	0·179	0·179	0·178
2·5	0·200	0.199	0·199	0·198	0·198	0·197	0·196	0·195
3·0	0·217	0·216	0·215	0·214	0·214	0·212	0·211	0·209
3·5	0.232	0·231	0·230	0·229	0·228	0·226	0·224	0·222
4·0	0.246	0·244	0·243	0·242	0·241	0·238	0·236	0·234
4·5	0·258	0·257	0·255	0·254	0·252	0·249	0·247	0·244
5·0	0·270	0·268	0·266	0·265	0·263	0·260	0·257	0·254
5·5	0·281	0·279	0·277	0·275	0·273	0·269	0·266	0·262
6·0	0·292	0·289	0·287	0·285	0·282	0·278	0·274	0·270
6·5	0·301	0·299	0·296	0·294	0·291	0·287	0·282	0·278
7·0	0·311	0·308	0·305	0·302	0·299	0·294	0·290	0·285
7·5	0·319	0·316	0·313	0·310	0·307	0·302	0·297	0·292
8·0	0·328	0·324	0·321	0·318	0·315	0·309	0·303	0·298
8·5	0·336	0·332	0·329	0·325	0·322	0·315	0·309	0·304
9·0	0·344	0·340	0·336	0·332	0·329	0·322	0·315	0·309
9·5	0·351	0·347	0·343	0·339	0·335	0·328	0·321	0·315
10·0	0·358	0·354	0·349	0·345	0·341	0·334	0·326	0·320
10·5	0·365	0·360	0·356	0·351	0·347	0·339	0·332	0·324
11·0	0·372	0·367	0·362	0·357	0·353	0·344	0·336	0·329
11·5	0·378	0·373	0·368	0·363	0·358	0·349	0·341	0·333
12·0	0·384	0·379	0·374	0·369	0·364	0·354	0·346	0·338
12·5	0·390	0·385	0·379	0·374	0·369	0·359	0·350	0·342
13·0	0·396	0·390	0·384	0·379	0·374	0·364	0·354	0·345
13·5	0·402	0·396	0·390	0·384	0·378	0·368	0·358	0·349
14·0	0·407	0·401	0·395	0·389	0·383	0·372	0·362	0·353
14·5	0·413	0·406	0·400	0·393	0·387	0·376	0·366	0·356
15·0	0·418	0·411	0·404	0·398	0·392	0·380	0·369	0·360
15·5	0·423	0·416	0·409	0·402	0·396	0·384	0·373	0·363
16·0	0·428	0·420	0·413	0·407	0·400	0·388	0·376	0·366
17·0	0·437	0·429	0·422	0·415	0·408	0·395	0·383	0·372
18·0	0·446	0·438	0·430	0·422	0·415	0·401	0·389	0·377
19·0	0·455	0·446	0·438	0·430	0·422	0·408	0·395	0·382
20·0	0·463	0·454	0·445	0·437	0·429	0·414	0·400	0·387
21·0	0·471	0·462	0·452	0·444	0·435	0·419	0·405	0·392
22·0	0·479	0·469	0·459	0·450	0·441	0·425	0·410	0·396
23·0	0·486	0·476	0·466	0·456	0·447	0·430	0·415	0·401
24·0	0·493	0·482	0·472	0·462	0·452	0·435	0·419	0·405
25·0	0·500	0·489	0·478	0·468	0·458	0·440	0·423	0·408
26·0	0·507	0·495	0·484	0·473	0·463	0·444	0·427	0·412
27·0	0·513	0·501	0·489	0·478	0·468	0·448	0·431	0·415
28·0	0·519	0·506	0·494	0·483	0·472	0·453	0·435	0·419
29·0	0·525	0·512	0·500	0·488	0·477	0·457	0·438	0·422
30·0	0·531	0·517	0·505	0·493	0·481	0·460	0·442	0·425

TABLE 94 DEPTH OF NEUTRAL AXIS — VALUES OF x/d BY ELASTIC THEORY

$d'/d = 0{\cdot}20$

$p_t m$	$p_c(m-1)/(p_t m)$ 0.0	0.1	0.2	0.3	0.4	0.6	0.8	1.0
1·0	0·132	0·132	0·133	0·133	0·134	0·135	0·135	0·136
1·5	0·159	0·159	0·160	0·160	0·160	0.161	0·161	0·162
2·0	0·181	0·181	0·181	0·182	0·182	0.182	0·182	0·183
2·5	0·200	0·200	0·200	0·200	0·200	0.200	0·200	0·200
3·0	0·217	0·217	0·216	0·216	0·216	0.216	0·215	0·215
3·5	0·232	0·231	0·231	0·231	0·230	0·230	0·229	0·228
4·0	0·246	0·245	0·244	0·244	0·243	0·242	0·241	0·240
4·5	0·258	0·258	0·257	0·256	0·255	0·254	0·252	0·251
5·0	0·270	0·269	0·268	0·267	0·266	0·264	0·262	0·261
5·5	0·281	0·280	0·279	0·277	0·276	0·274	0·272	0·270
6·0	0·292	0·290	0·289	0·287	0·286	0·283	0·280	0·278
6·5	0·301	0·300	0·298	0·29[illegible]	0·295	0·291	0·289	0·286
7·0	0·311	0·309	0·307	0·305	0·303	0·300	0·296	0·293
7·5	0·319	0·317	0·315	0·313	0·311	0·307	0·303	0·300
8·0	0·328	0·325	0·323	0·321	0·319	0·314	0·310	0·306
8·5	0·336	0·333	0·331	0·328	0·326	0·321	0·317	0·313
9·0	0·344	0·341	0·338	0·335	0·333	0·328	0·323	0·318
9·5	0·351	0·348	0·345	0·342	0·339	0·334	0·329	0·324
10·0	0·358	0·355	0·352	0·348	0·345	0·340	0·334	0·329
10·5	0·365	0·362	0·358	0·355	0·351	0·345	0·340	0·334
11·0	0·372	0·368	0·364	0·361	0·357	0·351	0·345	0·339
11·5	0·378	0·374	0·370	0·366	0·363	0·356	0·349	0·343
12·0	0·384	0·380	0·376	0·372	0·368	0·361	0·354	0·348
12·5	0·390	0·386	0·382	0·377	0·373	0·366	0·359	0·352
13·0	0·396	0·391	0·387	0·383	0·378	0·370	0·363	0·356
13·5	0·402	0·397	0·392	0·388	0·383	0·375	0·367	0·360
14·0	0·407	0·402	0·397	0·392	0·388	0·379	0·371	0·364
14·5	0·413	0·407	0·402	0·397	0·392	0·383	0·375	0·367
15·0	0·418	0·412	0·407	0·402	0·397	0·387	0·379	0·371
15·5	0·423	0·417	0·411	0·406	0·401	0·391	0·382	0·374
16·0	0·428	0·422	0·416	0·410	0·405	0·395	0·386	0·377
17·0	0·437	0·431	0·425	0·419	0·413	0·402	0·393	0·384
18·0	0·446	0·439	0·433	0·427	0·421	0·409	0·399	0·389
19·0	0·455	0·448	0·441	0·434	0·428	0·416	0·405	0·395
20·0	0·463	0·456	0·448	0·441	0·434	0·422	0·410	0·400
21·0	0·471	0·463	0·455	0·448	0·441	0·428	0·416	0·405
22·0	0·479	0·470	0·462	0·454	0·447	0·433	0·421	0·409
23·0	0·486	0·477	0·469	0·461	0·453	0·439	0·426	0·414
24·0	0·493	0·484	0·475	0·467	0·459	0·444	0·430	0·418
25·0	0·500	0·490	0·481	0·472	0·464	0·449	0·435	0·422
26·0	0·507	0·496	0·487	0·478	0·469	0·453	0·439	0·426
27·0	0·513	0·502	0·492	0·483	0·474	0·458	0·443	0·429
28·0	0·519	0·508	0·498	0·488	0·479	0·462	0·447	0·433
29·0	0·525	0·514	0·503	0·493	0·484	0·466	0·450	0·436
30·0	0·531	0·519	0·508	0·498	0·488	0·470	0·454	0·439

TABLE 95 AREAS OF GIVEN NUMBERS OF BARS IN cm^2

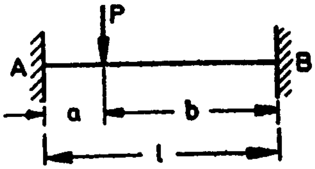
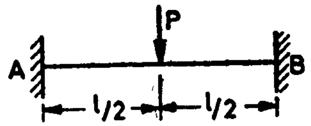
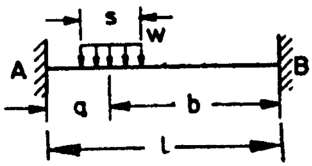
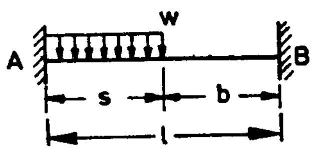
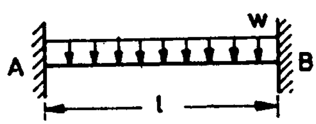
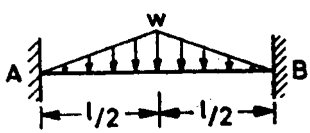
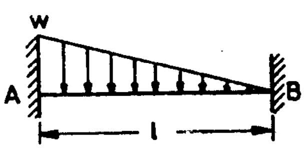
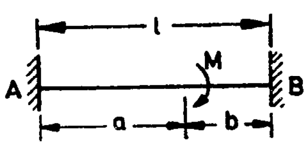
Number of Bars	Bar Diameter, mm												
	6	8	10	12	14	16	18	20	22	25	28	32	36
1	0·28	0·50	0·79	1·13	1·54	2·01	2·54	3·14	3·80	4·91	6·16	8·04	10·18
2	0·56	1·00	1·57	2·26	3·07	4·02	5·08	6·28	7·60	9·81	12·31	16·08	20·35
3	0·84	1·50	2·35	3·39	4·61	6·03	7·63	9·42	11·40	14·72	18·47	24·12	30·53
4	1·13	2·01	3·14	4·52	6·15	8·04	10·17	12·56	15·20	19·63	24·63	32·17	40·71
5	1·41	2·51	3·92	5·65	7·69	10·05	12·72	15·70	19·00	24·54	30·78	40·21	50·89
6	1·69	3·01	4·71	6·78	9·23	12·06	15·26	18·85	22·80	29·45	36·94	48·25	61·07
7	1·97	3·51	5·49	7·91	10·77	14·07	17·81	21·99	26·60	34·36	43·10	56·29	71·25
8	2·26	4·02	6·28	9·04	12·31	16·08	20·35	25·13	30·41	39·27	49·26	64·34	81·43
9	2·54	4·52	7·06	10·17	13·85	18·09	22·90	28·27	34·21	44·17	55·41	72·38	91·60
10	2·82	5·02	7·85	11·31	15·39	20·10	25·44	31·41	38·01	49·08	61·57	80·42	101·78
11	3·11	5·52	8·63	12·44	16·93	22·11	27·99	34·55	41·81	53·99	67·73	88·46	111·96
12	3·39	6·03	9·42	13·57	18·47	24·12	30·53	37·69	45·61	58·90	73·89	96·51	122·14
13	3·67	6·53	10·21	14·70	20·01	26·13	33·08	40·84	49·41	63·81	80·04	104·55	132·32
14	3·95	7·03	10·99	15·83	21·55	28·14	35·62	43·98	53·21	68·72	86·20	112·59	142·50
15	4·24	7·54	11·78	16·96	23·09	30·15	38·17	47·12	57·02	73·63	92·36	120·63	152·68
16	4·52	8·04	12·56	18·09	24·63	32·17	40·71	50·26	60·82	78·54	98·52	128·68	162·86
17	4·80	8·54	13·35	19·22	26·17	34·18	43·26	53·40	64·62	83·44	104·67	136·72	173·03
18	5·08	9·04	14·13	20·35	27·70	36·19	45·80	56·54	68·42	88·35	110·83	144·76	183·21
19	5·37	9·55	14·92	21·48	29·24	38·20	48·34	59·69	72·22	93·26	116·99	152·80	193·39
20	5·65	10·05	15·70	22·62	30·78	40·21	50·89	62·83	76·02	98·17	123·15	160·85	203·57

TABLE 96 AREAS OF BARS AT GIVEN SPACINGS

Values in cm^2 per Meter Width

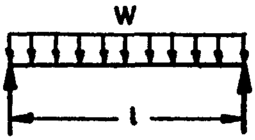
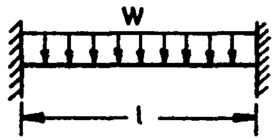
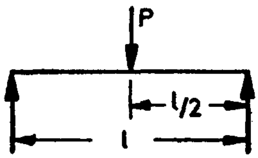
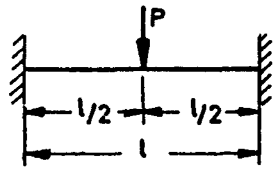
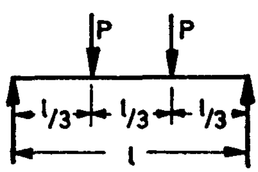
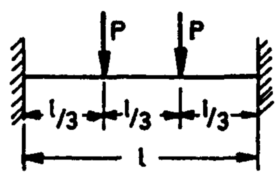
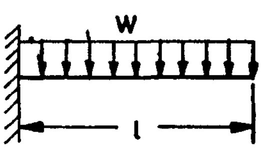
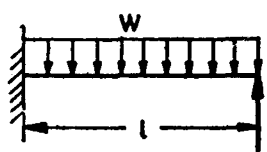
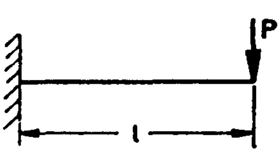
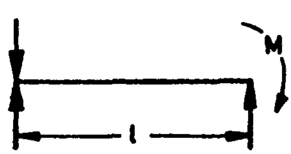
Spacing cm	Bar Diameter, mm											
	6	8	10	12	14	16	18	20	22	25	28	32
5	5·65	10·05	15·71	22·62	30·79	40·21	50·89	62·83	76·03	98·17	123·15	160·85
6	4·71	8·38	13·09	18·85	25·66	33·51	42·41	52·36	63·36	81·81	102·68	34·04
7	4·04	7·18	11·22	16·16	21·99	28·72	36·35	44·88	54·30	70·12	87·96	14·89
8	3·53	6·28	9·82	14·14	19·24	25·13	31·81	39·27	47·52	61·36	76·9	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	24·21	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	4·28	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	7·07	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 97 FIXED END MOMENTS FOR PRISMATIC BEAMS

LOAD TYPE	M_{FA}	M_{FB}
	$+\frac{Pab^2}{l^2}$	$-\frac{Pa^2b}{l^2}$
	$+\frac{Pl}{8}$	$-\frac{Pl}{8}$
	$+\frac{ws}{12l^2}[12ab^2+s^2(l-3b)]$	$-\frac{ws}{12l^2}[12a^2b+s^2(l-3a)]$
	$+\frac{ws^2}{12l^2}[2l(3l-4s)+3s^2]$	$-\frac{ws^3}{12l^2}(4l-3s)$
	$+\frac{wl^2}{12}$	$-\frac{wl^2}{12}$
	$+\frac{5wl^2}{96}$	$-\frac{5wl^2}{96}$
	$+\frac{wl^2}{20}$	$-\frac{wl^2}{30}$
	$-M\frac{b}{l}(2-\frac{3b}{l})$	$-M\frac{a}{l}(2-\frac{3a}{l})$

Note:- w is the load per unit length

Table 98 DEFLECTION FORMULAE FOR PRISMATIC BEAMS

	$\frac{5}{384} \times \frac{Wl^3}{EI}$		$\frac{Wl^3}{384\,EI}$
	$\frac{Pl^3}{48\,EI}$		$\frac{Pl^3}{192\,EI}$
	$\frac{23\,Pl^3}{648\,EI}$		$\frac{5\,Pl^3}{648\,EI}$
	$\frac{Wl^3}{8\,EI}$		$\frac{Wl^3}{185\,EI}$
	$\frac{Pl^3}{3\,EI}$		$\frac{1}{16} \times \frac{Ml^2}{EI}$

Note:- W is total distributed load