Government of West Pakistan

PUBLIC WORKS DEPARTMENT
BUILDINGS & ROADS BRANCH

CHIEF ENGINEER'S WEST PAKISTAN
TECHNICAL MEMO. NO. 16

REINFORCED CONCRETE IN BUILDING

1st EDITION

1957
CHIEF ENGINEER, WEST PAKISTAN BUILDINGS AND ROADS
DEPARTMENT

Preface to 1st Edition

Concrete construction has now become a usual feature in any building programme. There are now so many codes and practices available on the subject. Much literature and research publications are now available in this field. Study and practice in this field of engineering is itself whole time job. As the officers and staff of Buildings and Roads Department are occupied in so many fields of activity, it is not possible for them to keep pace with the latest trends and developments. This Technical Memo., therefore, has been prepared to give them a fair idea about concrete and Reinforced Concrete work and to provide a fair guide in working designs of simple structural elements.

2. This manual in the present form has been compiled to give a general view of the subject. In very broad outlines the subject matter has been presented about the concept and basic design, the principles involved and the points to be kept in view in design and construction. A general review about concrete making has also been presented.

3. This memo has been prepared by Mr. G. D. Habib, P.S.E.I., Superintending Engineer, of this department. I am personally extremely grateful to him to have spared so much time.

4. In order to standardise the system of office and out-door works in the entire West Pakistan, the Chief Engineer, West Pakistan's Technical Memos, on the following subjects are being published:—

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5. These Memoranda are available at Government Publication with the Superintendent, Government Printing Press, Government of West Pakistan, Lahore, and his Sales Depots throughout the West Pakistan.

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Chief Engineer, West Pakistan  
Buildings and Roads Department,  
Lahore  

Dated Lahore, the 1958
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REINFORCED CONCRETE IN BUILDINGS, ITS DESIGN AND CONSTRUCTION

CHAPTER I—BASIS OF DESIGN

SECTION I—NOTATIONS

AC : Area of steel in compression.
At : Area of steel in tension.
b : Breadth of beam or column or breadth of rib of flanged beam.
Bs : Breadth of T-beam flange.
dc : Thickness of slab of flanged beam.
d : Effective depth of slab and beam.
e : Eccentricity.
Ja or a : Lever arm or distance between centroids of forces producing the resistance moment.
I : Effective span of beam.
M : Bending moment.
MR : Moment of resistance.
m : Modular ratio \( \frac{(E_s)}{E_c} \)

r : Tensile steel ratio \( \frac{A_t}{B_d} \)
n : Depth of neutral axis.
S : Shear force.
s : Shearing stress.
C : Total compression in section.
T : Total tension in section.
c : Stress in concrete.
t : Tensile stress in reinforcement.
tc : Compressive stress in concrete.
w : Load per unit area of slab or per ft. length of beam.
Y : Distance of neutral axis to extreme fibre.
Aw : Area of web or shear reinforcement.
O : Sum of perimeters of reinforcement.
p : Pitch.
tw : Tensile stress in web or shear reinforcement.
sb : Bond stress between concrete and steel.
A : Total cross-sectional area of column.
K : Stiffness in column.

n : \( \frac{n}{d} \) where d is length of side of column.
P : Total axial load on column.
Sp : Punching shear stress on column footing.
p : Intensity of bearing pressure.

SECTION II—PERMISSIBLE STRESS IN REINFORCED CONCRETE

1. Permissible tensile stress in steel used in tension ..... 18,000 Lbs./in.²

2. Permissible compressive stress in steel used in compression in beams or columns ..... 18,000 Lbs./in.²
3. Permissible compressive stress in concrete
   1 : 2 : 4          ...          ...          750 Lbs./in.²
4. Permissible bond stress between concrete
   and steel (bars hooked at ends)          ...          ...          100 Lbs./in.²
5. Permissible bond stress at change of bending
   moment or at the point of maximum shear
   ...          ...          ...          150 Lbs. /in.²
6. Permissible shear stress in concrete
   ...          ...          75 Lbs./in.²
   (without reinforcement)
7. Permissible maximum shear stress in concrete
   with the provision of stirrups          ...          ...          300 Lbs./in.²
8. Permissible punching shear          ...          ...          150 Lbs./in.²
9. Modular ratio          ...          ...          15
10. Bearing in plain concrete          ...          ...          400 Lbs./in.²
11. Permissible steel stress in shear reinforcement in beams
   ...          ...          12,000 Lbs./in.²

(i) The maximum permissible working stress in compression
    due to bending is one-third of the 28 days strength of cubes
    made on the site.

(ii) The maximum direct compressive stress in columns is 80
     per cent of the allowable bending stress. This stress applies
     only if the ratio of the effective length of least radius of
     gyration does not exceed 50. For ratios between 50 and
     120 the stress is further reduced.

(iii) The shear stress on the lever arm section of a beam should not
     exceed 10 per cent of the maximum permissible compressive
     stress in bending. When the whole of the diagonal tension
     due to the shearing forces is taken on the reinforcement,
     the shear stress should not exceed 0.4 C. Punching shear
     should be limited to 0.2 C.

(iv) The permissible bond stress is equal to the shear stress
     plus 25 Lbs./in.². The bond stress caused by variation of
     tensile stress due to bending is twice the tabulated bond stress.

(v) Compression reinforcement in beams can be stressed to 18,000
     Lbs./in², if designed on steel beam theory, that is neglecting
     the compressive resistance of the concrete.

(vi) For compression in beams and columns where the compres- 
     sive resistance of the concrete is taken into account, 
     the stress in the reinforcement is taken as the stress in 
     the surrounding concrete multiplied by the modular 
     ratio.

(vii) When a member is subject to bending moment combined with 
     direct thrust it is usual to work to the same permissible 
     compressive stress in the concrete as if the members were 
     acted on by bending alone. In a column subject to direct 
     thrust and bending moment the compressive stress should 
     not exceed 600 Lbs./in² for direct load alone, nor exceed 
     750 Lbs./in² when the direct load is combined with bending 
     moment.

SECTION III—FORMULAE

(i) Rectangular beam or Slab singly reinforced

Position of neutral axis

\[
\frac{1}{2}bn^2 = mA (d-n) 
\]

(1)

Lever arm or arm of resistance moment.

\[
'a' = Jd = \left( d \frac{n}{3} \right) 
\]

(2)
Maximum compressive stress at extreme concrete fibre.
\[ \frac{1}{2} \text{ ben} = \tau A_t \quad (3) \]
Tensile stress in tension reinforced.
\[ t = \frac{\left( d - n \right) m}{n} = \frac{M}{A_t Jd} \quad (4) \]
Moment of resistance.
\[ M = Q bd^2 = t \times A_t \times a \quad (5) \]

(ii) Rectangular beam doubly reinforced.
R. M. of compression concrete and steel.
\[ \left( \frac{1}{2} \text{ ben} a \right) + A_c (m-1) \frac{n-de}{n} c(d-de) \quad (1) \]

Where \( a = \left( d - \frac{n}{3} \right) \)
Lever arm of combined compressive stress.
\[ = \frac{14 A_t C + 1/2 \text{ ben}}{M} \quad (2) \]
\[ A_t = \frac{b \times \text{ Lever arm.}}{M} \quad (3) \]
\[ I = 1/3 \text{ bn } 3 + m A_c (n-de)^2 - m A_t (d-n)^2 \quad (5) \]
\[ C = \frac{B_n}{I} = \frac{n t}{m-(d-n)} \quad (6) \]
\[ t = \frac{B_n}{I} \left( d - \frac{n}{3} \right) \quad (7) \]
Lever arm ‘Jd’ = \( \left( d - \frac{n}{3} \right) \quad (8) \)
\[ B = c \times Jd \times \text{ area of compressive steel} \]
\[ = t \times A_t \times Jd. \quad (9) \]
Total compression in concrete and steel
\[ \frac{b}{d} \text{ ben} + (m-1) \frac{n-de}{n} A_c \quad (10) \]

(iii) T-Beam singly reinforced.
Economic depth \( d = \sqrt{\frac{r \times M}{b \times b}} + \frac{d s}{2} \quad (1) \)
\[ r = \frac{\text{Cost of 1 Cft. of steel}}{\text{Cost of 1 Cft. of concrete}} \]
'B' is taken the least of the following three—
(i) \( 1/3 \) L
(ii) or the distance between the centre of ribs of the (2) T-beam.
(iii) or \( 12 \sqrt{dc+b} \), whichever is less.
Approximate \( Jd = (d + 2ds) \quad (3) \)
\[ n^2 = \frac{2 A_t m \times d + (b_x \times d)_x^2}{2 (A_t m + B_x \times d)} \quad (4) \]
\[ Jd = d \left( \frac{dc}{3} \times \frac{3n-2dc}{2 n-de} \right) \quad (5) \]
\[ t = \frac{M}{A_t Jd} \quad (6) \]
\[ C = \frac{n \times t}{m (d-n)} \quad (7) \]
(iv) T-Beam doubly reinforced

\[ a = \frac{B, d c^2 + 2m (Ac, d) \times A_t, d)}{2 (B, d c + m (A_t + A))} \tag{1} \]

\[ I = \frac{1}{3} B n^2 - \frac{1}{3} B^2 \times 3 + m A_t (d - n)^2 + m Ac (d - d_t)^2 \tag{2} \]

\[ B = \frac{C, I}{n} = \frac{t, I}{m (d - n)} = t A (d - 4d_e) \tag{3} \]

(v) Shear and Bond Stress

Maximum shear stress for rectangular beam (1)

\[ S = \frac{S}{b, Jd} \tag{2} \]

Maximum shear stress in flanged beam.

\[ S = \frac{b, Jd}{b, Jd} \tag{2} \]

Shear force resisted by bars bent up at angle \( \theta \) in continuous system.

\[ S = \frac{a}{P} \times t_s, A_t (\sin \theta + \cos \theta) \tag{3} \]

Shear force resisted by bars bent up at angle \( 45^\circ \) in continuous system.

\[ S = \sqrt{2} \times a \times t_s, A_t \tag{4} \]

Shear force resisted by bar bent up at angle \( \theta \) in intermittent system.

\[ S = t_s, A_t \sin \theta \tag{5} \]

Shear force resisted by vertical stirrups.

\[ S = \frac{a}{P} \times t_s, A_t \tag{6} \]

Maximum Bond stress \( s_b = \frac{S}{\theta, Jd} \tag{7} \]

Spacing of bent up bars.

\[ P = \frac{\cos \theta + \sin \theta}{b, w} \times t_s, A_t \tag{8} \]

When \( \theta = 30^\circ \)

or \( P = \frac{1.306}{t_s, A_t} \) when \( \theta = 45^\circ \)

Spacing of vertical stirrups.

\[ P = \frac{Jd, A_t}{S} \times 12, Jd. \tag{9} \]

\( A_t \): Combined area of the arms of the stirrups.

(vi) Columns

Safe axial load on column.

\[ P = C (A - A_t) + t_s, A_t \tag{1} \]

Stresses \( t_s \) and \( c \) are multiplied by reduction coefficient where applicable.

Moment of inertia of column.

\[ I = \frac{bd^3}{12} + (m - 1) A_t d^3 \left( \frac{1}{3} - K \right)^2 \tag{2} \]

Where \( K \) = distance of centre of steel from outside edge of column/d.

Maximum compressive stress on an eccentrically loaded column when no tension produced.

\[ C = \frac{P}{bd + (m - 1) A_t} + \frac{P \times e \times 12, d}{I} \tag{3} \]

Column under eccentric load producing tension.

Load \( P = \frac{1}{2} \ln^2 \frac{d_e + \frac{1}{2} A_t (n' - K)}{n'} \left( \ln 1 \right) C \]

\[ - \frac{1}{2} A_t \left( \frac{1 - K - n'}{n'} \right) mc \tag{4} \]
Column under eccentric load producing tension.
\[ P \times e = 4\bar{b}n^3 \int \left( \frac{1}{3} - \frac{n'}{3} \right) + \frac{1}{4} A_e \left( \frac{n'' - K}{n''} \right) (m-1) c \]
\[ + \frac{(1-K-n')}{n''} mc \left( \frac{1}{4} - k \right) d \quad (5) \]

Purchasing shear stress on columns footing.
Safe column load 
\[ P = S \times 2 (b+d) \times ds + pbd \quad (6) \]

Circular Columns—
\[ A_n = \frac{\Pi D^2}{4} + (m-1) A \quad (7) \]
\[ I = \frac{\Pi D^4}{6^2} \quad (8) \]
\[ I = \frac{A_s D^4}{8} \text{ If the reinforcing bars are arranged} \quad (9) \]

at short distance apart in a Circle of main diameter \( D_2 \) we can consider these as replaced by a ring of steel of the same total area.
\[ A_s = \text{Area of steel in ring}. \quad (10) \]
\[ IE = I + I_s \]
\[ = \frac{\Pi D^4}{6^2} + \left( m-1 \right) A_s D_2^4 \quad (10) \]

\[ K = \text{Radius of gyration}. \quad (10) \]
\[ = \left( I + (m-1) I_s \right) \]
\[ \left( A_e + (m-1) A_s \right) \]

For short columns radius of gyration
\[ e = \frac{M}{P} \text{ where } M = \text{Bending moment}. \quad (50) \]

\[ P = \text{Total axial load}. \quad (50) \]

\[ C_{\text{max}} = \frac{P}{A} + \frac{MY}{I} = \frac{P}{A_e + (m-1) A_s} + \frac{P e X Y}{K^2} \quad (11) \]

\[ C_{\text{min}} = \frac{P}{A_e + (m-1) A_s} \quad (12) \]

\[ D = \frac{P e Y X}{2 K^2} \quad (13) \]

Maximum \( t = (m-1) C \quad (vi) \text{Wind Pressure} \]

\[ P = \frac{V^2}{600} \sqrt{1+0.06 (h-s)} \quad (1) \]

\[ V = \text{Wind velocity in miles per hour at a height of 40 ft. above} \]
\[ \text{ground on an open site}. \]

\[ h = \text{the height in feet of the windward vertical face above the} \]
\[ \text{general ground level measured to cover top of parapet} \]
\[ \text{wall}. \]

\[ s = \text{The height in feet above general ground level assumed to be} \]
\[ \text{sheltered by permanent obstacles such as buildings. The} \]
\[ \text{value of 's' should not be taken as more than 0.5} \]
\[ \text{h. The value of } P \text{ is not taken less than 7 lbs.} \]
\[ \text{per sq. foot}. \]

\[ \text{SECTION IV—CUBAL AND PRACTICAL RULES IN DESIGN} \]
\[ \text{OF REINFORCED CONCRETE} \]

(i) Floor slabs are designed as rectangular beams of 12" width. However, short the span may be the maximum overall thickness of a slab should not be less than 3". For uniformly distributed loads on a slab simply supported at ends, the depth should be 1/2" for every foot of span with a limitation of 7" for a 14 foot span. If the span exceeds this, some means to reduce it, and eventually the depth, should be adopted;
such as providing intermediate beams or introducing two way reinforcement for a slab. For a cantilever slab with a uniformly distributed load the thickness of slab at support should not be less than \( \frac{\text{span}}{8} \).

(ii) **Cover**—The thickness of concrete cover on a steel bar shall be:

(a) For slab reinforcing bars, not less than 1/2", nor less than the bar diameter.
(b) For longitudinal reinforcing bars in beams not less than 1", nor less than the bar diameter.
(c) For a longitudinal reinforcing bar in a column not less than 1\( \frac{1}{4} \)", nor less than the bar diameter.
(d) At each end of a reinforcing bar not less than 1", nor less than twice the diameter.

(iii) **Distance between bars**—(a) The pitch of the main reinforcing bars in beams and slabs shall not exceed twice the effective depth or 12" whichever is less.
(b) The minimum distance between the bars shall not be less than the diameter of the bar or 1/2" more than the maximum size of coarse aggregate.
(c) The vertical distance between horizontal main reinforcing bars shall be at least 1/2" inch except at splices or where transverse bars are in contact.

(iv) **End Anchorage**—(a) At the end of any bar, end anchorage shall be provided either:

(i) in the form of a semicircular hook of a minimum dimensions shown. The anchorage value of this hook is taken as equivalent to 16D.
(ii) By a 90° bend with an additional straight length of 8 D.

To develop its full strength a bar under stress must be impeded in concrete for a distance of at least 45D on each side of the point at which the maximum tensile or compressive stress occurs.

(v) **Effective Span**—The effective span of a beam or slab should be taken as the lesser of (a) the distance between centres of supports, (b) the clear distance between supports plus the effective depth of the beam or slab.

(vi) **Minimum dimensions of a beam**—(a) The depth of a rectangular beam should not be less than \( \frac{1}{20} \)th of clear span.
(b) Neither the breadth of a rectangular beam nor the flange breadth of a T-beam shall be less than \( \frac{1}{20} \)th of clear span. If it be less, beams shall be secured laterally so that the breadth is not less than \( \frac{1}{20} \) (length of beam between lateral support).

(vii) **Distribution Reinforcement in Slabs spanning in one direction**—

(a) The aggregate cross-sectional area of distribution reinforcement shall be at least 20 per cent of the main tensile reinforcement.
(b) The pitch of distribution bars shall not exceed four times the effective depth of slab.
(c) These bars may be of 1/4" diameter and are tied on the top of the main steel at every crossing by pieces of wire.

(viii) **Longitudinal Reinforcement on a Column**—(a) The cross-sectional area of longitudinal reinforcement on a column shall not be less than 0.8 per cent, nor more than 8.0 per cent of the gross cross-sectional area of the column.

(b) At all points in longitudinal reinforcement, the bars shall overlap for a distance of not less than 24 times the diameter of the smaller bar.

(ix) **Transverse Reinforcement on a Column**—(a) The diameter of transverse reinforcement shall not be less than one quarter the diameter of the main longitudinal bars and in no case less than 3/16".
(b) The pitch of transverse reinforcement shall not be more than the least of the following three dimensions:—

(i) the least lateral dimensions and the column,
(ii) twelve times the diameter of the smallest longitudinal reinforcement in the column,
(iii) twelve inches.

(c) The volume of transverse reinforcement shall not be less than—

(i) 0.2 per cent of the gross volume of a column, having up to 2 per cent area of main longitudinal steel.
(ii) 0.4 per cent of gross volume of a column having area 2 per cent of main longitudinal steel.

SECTION V—LOADINGS IN BENDINGS

For design of individual members of a building, assessment of loads on a structure is necessary. Two types of loads act on a structure, namely: dead or permanent load and superimposed or live load.

(a) Dead Loads—Dead loads include the weight of reinforced structure itself together with all other permanent weight, e.g., roofs, walls, stairs, ceiling, etc. where concrete members such as lintels support brick or masonry walls, dead load is calculated by considering only the amount of brickwork contained in an equilateral triangle, whose base in the effective span of the lintel. In case of partition walls, full weight of the partition is taken.

(b) Super-imposed Loads—Super-imposed loads are taken as given in Table 4. The minimum alternative loads have the effect of increasing the super-imposed load to be carried on slabs and beams over relatively short spans. These loads should be used in place of those in column 3 where the former cause higher stresses. The minimum loading is a safeguard against local loading due to concentrated loads which might occur in practice. The reduction of column loads for a building of more than two storeys in height is given in Table 5.

(c) Corridors, Stair and Landing—These are designed for the same loading class as the floors or other space to which they give access. Class VII is, however, taken for corridors, stairs and landings leading to space of public assembly with fixed seating. This class is the maximum taken for any corridor, stair or landing.

(d) Super-imposed Roof Load—On flat roofs and sloping roofs up to 10 degrees, a super-imposed load including a load to provide for loose snow up to 2 feet, depth of 30 lbs. per sq. ft. measured on plan should be allowed for subject to a minimum load of 240 Lbs. uniformly distributed per foot width on roof slabs or roof covering. The combined effect of snow loads and wind pressure on roofs are taken into account.

(e) Wind Pressure—Wind pressure and suction are the pressure in lbs. per sq. foot which is the equivalent intensity of wind loading in the windward direction. This is given by formula I-VII under Wind Pressure.

CHAPTER II—DESIGN PROCEDURE

SECTION VI—DESIGN OF SLAB OR BEAM SINGLEY REINFORCED

A. It is the usual practice to design floor slabs as rectangular beams reinforced in tension only, and to make them a whole number of half inches thick 4", 4.5", 5", 5.5", etc. To design such slab or beam the procedure adopted is as follows:

For a slab consider a 12" wide stirrup parallel to the span, i.e. h=12 inches. Find effective span l ft. and calculate 'w', total load in Lbs. per ft. run (if load is uniform). Calculate max B—M = \( \frac{w^2 l^2}{8} \times 12 \) in Lbs. for a single span, simply supported.
Equate moment of resistance in terms of $bd^2$ with $M$ and find minimum value of $d'$ required. Derive $a'$ and find tensile steel area from formula 5(1). Find suitable size and number of bars. From mean $d'$, bar size, and assumed cover, decide total depth to the nearest $\frac{1}{8}$ inch and calculate actual $d$. If actual $d'$ differs greatly from assumed $d'$ repeat the above procedure. Decide distribution bars at 20 per cent of area of main bars.

**Example 1**—Design a floor slab to carry a total load of 100 lbs. per sq. ft. over a 12 ft. clear span.

Permissible stress in steel and concrete:

$$t = 18,000 \text{ lbs.}/\text{in.}^2$$

$$c = 750 \text{ lbs.}/\text{sft}.$$  

Loads—Assume a 6 inch thick slab.

Live load — 100 lbs./sft.

Slab weight \[ \frac{144}{2} = \frac{72}{172} \text{ lbs.}/\text{sft}. \]

Consider a 12" wide strip of slab parallel to the span. Assume effective depth $d'$ of slab as 5 inches.

Effective span is therefore $12' + 5' = 12 \text{ ft.} 5''$.

Maximum bending moment,

$$M = \frac{wpl^2}{8} = \frac{172(12.41)^2 \times 12}{8} = 39,732 \text{ in. lbs.}$$

Movement of resistance of slab $= 125 \text{ bd}^2$  

$125 \text{ bd}^2 = 39,732$  

$d = 5$ inches

Movement of resistance in steel in tension:

$$t A_x \times a = M$$

$$A_x = \frac{39,732}{18,000 \times 0.87 \times 5.04} = 0.52 \text{ in.}$$

Use $\frac{1}{8}$ in. diameter bars at $\frac{3}{4}$ inch centres.

Total thickness of slab $= 5.75$ inches.

Steel percentage $= \frac{0.52 \times 100}{12 \times 5} = 0.86$ per cent.

**Distribution Reinforcement**—

20 per cent of area of main reinforcement.

$$= \frac{20 \times 0.5}{100} = 0.1 \text{ sq. inch.}$$

Use 5/16 inch dia. bars at 9" centres.

**Shear Stress**—

$$\frac{S}{bld} = \frac{1,067}{4 \times 1.88 \times 5} = 60 \text{ lbs./in.}$$

**Example 2**—Design a singly reinforced rectangular beam to carry a super-imposed load of 1,000 lbs. per ft. run over 18 ft. clear span.

Assume dimensions—

$$b = \frac{\text{Span}}{20} = \frac{18}{20} = \text{Say 12 inches}$$

$$d = 2b = 24 \text{ inches.}$$

Total depth $= 20$ inches.

Loads — Live 1,000 lbs. ft. run.

Dead $= \frac{12 \times 28}{144} \times 144 = 312$ lbs.

Total $= 1,312 \text{ lbs./ft. run.}$
Clear span = 18 ft.
Effective span (clear span + d) = 20 ft.
Maximum B.M. = \( \frac{1.312 \times (20)^2 \times 12}{8} = 787,200 \) lbs.

Moment of resistance of beam = \( 125 \times b d^2 = 865,000 \) lbs. in.

The size of the beam is safe in concrete strength.

Moment of resistance in tension,
\( t \times A_t \times a = 787,200 \)
\( A_t = 2 \text{ sq. inches, four } \frac{3}{8}'' \text{ bars} \)

The beam as designed is therefore safe, i.e. 26'' \( \times \) 12'' with four 7/8'' bars.

B. Design of Slab simply supported on four sides—A slab supported on all four sides where the length of the slab is less than twice its breadth is designed with tensile reinforcement in both directions at right angles. For a uniformly loaded simply supported slab, the proportion of the total bending moment to be designed for in each direction may be taken from table 3.

The maximum bending moment in a strip of unit width on each span is taken as

\[ M_1 = Z_1 \frac{wL^2}{8} \text{ on long span L,} \]

\[ M_2 = Z_2 \frac{wL^2}{8} \text{ on short span L,} \]

Where \( W \) = total uniformly distributed load per unit area of slab.
\( Z_1 \) and \( Z_2 \) are B.M. coefficients across long and short spans respectively.

To determine the support reaction on the edges of the slab the total load is proportioned geometrically.

Load carried by short support

\[ = \frac{wL^2}{4} = W_s \]

Load carried by long support

\[ = \frac{W_1 L}{2} = W_a \]

To design a slab supported on all four sides calculate the bending moment and steel required on 12'' wide strip in each direction. The long span reinforcement is placed above the short span reinforcement to form a mat.

Example 2—Design a floor slab simply supported on all four sides to take a super-imposed load of 100 lbs. per sq. ft. The size of the room is 15 ft. \( \times \) 18 ft.

Permissible stress

\[ i = 18,000 \text{ lbs/in.}^2 \]
\[ e = 750 \text{ lbs/in.}^2 \]
\[ n = 15 \]

Ratio of sides

\[ = \frac{18}{15} = 1.2 \]

From table 3
\[ Z_1 = 0.48 \]
\[ Z_2 = 0.32 \]
Assume overall depth of slab = 7 inches.
\[ d = 6 \text{ inches.} \]

Loads—

Weight of slab
\[ \frac{7}{12} \times 144 = 84 \text{ lbs./sq. ft.} \]
\[ = 100 \text{ lbs. sq./ft.} \]

Total = 184
Short Span—Consider a strip of slab of breadth
\[ b = 12 \text{ inches}. \]

Effective span \[ = 15 \text{ ft.} + 6 \text{ inches} = 15'5 \text{ ft.} \]

Max. B.M. \[ = \frac{Z_1 w l^2}{8} = \frac{0.68 \times 184 \times (15.5)^2 \times 12}{8} \]
\[ = 45,000 \text{ lbs. ins.} \]

\[ d = \sqrt{\frac{45,000}{125 \times 12}} = 5.5'' \]

\[ (t \times A_t \times a) = 45,000 \]

\[ a = 6'' \]

\[ A_t = \frac{45,000}{18,000 \times 0.85 \times 6} = 0.3 \text{ sq. inches.} \]

Provide \( \frac{1}{2} \) inch dia. bars at \( 4\frac{1}{2} \) inches centre.

Long Span—
Assuming same size of bar
\[ d = d \text{ dia. of bar} \]
\[ = 6 - \frac{5}{2} = 5 \text{.375 inches} \]

Effective span \( = 18'5 \text{ ft.} \)

Max. S.M. \[ = \frac{Z_4 w l^2}{8} = \frac{0.325 \times 84 \times (18.5)^2 \times 12}{8} \]
\[ = 30,700 \text{ inches.} \]

\[ d = \sqrt{30,700} = 4.5 \text{ inches.} \]

Actual ‘d’ is greater than this

\[ A_t = \frac{30,700}{18,000 \times 0.85 \times 5.375} = 0.49 \text{ sq. inches.} \]

Provide \( \frac{1}{2} \) inch dia. bars at \( 5\frac{3}{4} \) inches centres.

C. Design of slab restrained on all four sides—Where the corners of a slab are prevented from lifting adequate provision for tension is made and bending moment may be assumed to have the values given below:

The maximum B.M. per unit width in the middle strip of a slab

\[ M_L = Z_L w L^2 \]

\[ M_M = Z_M w L^2 \]

\[ M_L \] and \[ M_M \] are the maximum bending moments on strip of unit width in the direction of spans L and M respectively.

\[ Z_L \] and \[ Z_M \] are coefficients given in table (3).

Reinforcement is provided at the corners of the slab. The effective area of such reinforcement per unit width is equal to that required for the maximum positive moment in the middle strip. This amount is provided near both the top and bottom faces of the slab for a distance of one-fifth of the short span in both directions from the corner. The effective area of the reinforcement is the normal area multiplied by the size of the angle which the reinforcement makes with the critical sections. In the top of the slab the critical section is perpendicular to the diagonal. In the bottom of the slab it is parallel to the diagonal.

D. Design of continuous slabs—A slab spanning three or more supports is treated as continuous slab. To provide negative moment over supports, tensile bars in the bottom of the slab when no longer required for positive bending moment are bent up and carried over the supports in the top of the slab. It is a common practice to bend up half the number of tensile bars and carry the remaining half through to the support in the bottom of the slab. Shear reinforcement is never necessary in slabs. The shear strength of the section may be increased by bending up bars and it may be taken as \( 4/3 \) of shear strength of concrete. If the slab is not strong
enough in shear, the depth is increased to get the necessary extra strength. Bars in slabs are bent up at 30 degree to the central axis. These should be bent up not further than 0.21 from the center of a support, and should extend on the top of the slab, a distance of 0.251 or the bend distance whichever is greater than the center of an interior span. The distance 0.251 is measured from the commencement of the hook. For an end span, top bars should run to the center of the end support. Bending moment coefficients for continuous slabs with various number of spans, and conditions of loading are given on Table 6.

Example 4—Design a floor slab continuous over a number of spans of T-beams placed at 7 ft. clear spacing to stand a super-imposed load of 200 lbs./sq. ft.

Assumed slab thickness 4 inches

<table>
<thead>
<tr>
<th>Loads</th>
<th>Live load</th>
<th>= 200 lbs. per sq. ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dead load</td>
<td>= 48 lbs.</td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>= 248</td>
</tr>
</tbody>
</table>

Bending Moment—Considering a 12 inches wide strip of slab, load per ft. run = 248 lbs. on a continuous slab for interior span maximum bending moment is at mid span or at support

\[ \frac{w l^2}{12} \]

Hence

\[ \frac{w l^2}{12} = \frac{248 \times (7.3)^2 \times 12}{12} = 13,300 \text{ in lbs.} \]

Moment of resistance due to concrete in compression.

\[ 125 \times 12 \times d^2 = 13,300 \]

\[ d = 3 \text{ inches} \]

Total depth \( = \frac{3}{16} + \frac{9}{16} = \frac{1}{2} \)

Tensile steel

\[ t \times a = 13,300 \]

\[ a = 0.87 \times d \]

\[ A_t = \frac{13,300}{18,000 \times 0.87 \times 3.3} = 0.257 \]

Use \( \frac{3}{4} \) inches dia. main bars, \( t \times 5 \) inches centres.

Distribution steel—20 per cent of 0.257 sq. in. = 0.065 sq. inches.

Use \( \frac{3}{4} \) inches dia. distribution bars at 10 inches centres.

Shear Stress—

Maximum shear

\[ \frac{248 \times 7.3}{2} = 905 \text{ lbs.} \]

Maximum shear stress

\[ = \frac{905}{12 \times 0.87 \times 3.3} = 26 \text{ lbs.} \]

Maximum Bond Stress—

Perimeter of tensile top bars available for bond in a 12 inches width of slab at the

Support

\[ = 1.178 \times \frac{1.2}{5} \]

Safe bond stress

\[ = 0.87 \times 3.3 \times 1.188 \times 12 = 110 \text{ lbs./in}^2 \]

against 150 lbs./in².
SECTION VIII—Floor Beams

All beams which support and are monolithic with a slab are designed as T-beams or L-beams. In designing T-beam, the flange forms part of the slab spanning across the beam. The slab is designed first, and the flange thickness is thus decided. To design the rib a likely economic depth and breadth of the rib is first calculated at the start. The ratio of rib breadth to overall depth is usually taken as 1:3 to 2:3.

A large floor of a building generally consists of a reinforced concrete slab spanning across secondary T-beams, which frame into and are supported by main beams. The main beams are supported partly on walls and partly on columns. The slab is then designed as a continuous slab spanning across secondary beams. Both secondary and main beams are designed as continuous beams, but different factors affect the design of each category of beam. The depth of the main beams is kept greater than that of secondary beams as the tensile bars in the secondary have to pass over the tensile bars in the main beams. The loads on the beam are a series of point loads due to the reaction of secondary beams on the main beams either at mid points or at the third points. The moments at the mid span and supports may be considered equal and the design is considered for the worst condition. For secondary beams, in order to make bending moment in end spans approximately equal to moments in interior spans, end spans are usually made equal to 0·9 of an interior span.

At the supports, T-beam is designed as a doubly reinforced rectangular beam with tension flange uppermost. At this place the beam is usually weak in compression and an excessive amount of compression steel exceeding 3 per cent may make concreting difficult.

In such a case, splay may be given to the beams near the support. The tensile bars from mid spans carried through the supports are bent into the splay. The layer arm and moment of resistance of the beam is thus increased at the support, and the addition of further steel is avoided. Half of the bars are bent up to act as compressive steel. The breadth of such a beam is ‘b’ and the depth is equal to the total depth of the rib and the slab. Its effective depth is measured from the bottom of the rib to the centre of the top bars.

A. T-Beam Design—There is no method of designing a T-beam straightforward like slab, as its strength depends on the depth, breadth and percentage of main tension steel, and percentage of any of compression steel. It is, therefore, necessary to commence by assuming certain dimensions, and then proceed with calculations, and see whether the assumptions are reasonable. We are then in a position to determine the economical depth of T-beam from the formula (iii)(1). This formula gives the total depth from which depth of the steel is obtained after deducting the thickness of the slab. If this depth works out different from the assumed dimensions, it may be necessary to apply the formula again using a bending moment figure obtained from accurate assumption. To determine the steel for main tension reinforcement, it is necessary to work out an approximate value of Jd from the formula (iii)(3). The steel thus obtained is provided in the stem using 1” or 1½” steel round bars. The neutral axis of the section is worked from the formula (iii)(4). Correct ‘Jd’ is then calculated from the formula (iii)(5). The value of ‘t’ and ‘e’ is obtained from the formulae (iii)(6) and (iii)(7). This value is at the centre of gravity of the system, and the maximum stress will be in the bottom layer of the bars. The next step is to investigate the shear force which is given by formula (v)(1). In T-beam the shear stress so calculated is generally above the permissible of 75 lbs./sq. in. It is therefore, arranged to provide shear reinforcement to withstand the whole of the shear force. This shear reinforcement is usually of two kinds, i.e., bent up bars to act as tension member of the lattice system, and stirrups to take up shear.
Example 7—Design a T-beam for a room with effective span of 30 ft. carrying a uniformly distributed load of 750 lbs. per foot run and to stand stresses of

\[ t = 18,000 \text{ lbs./in}^2 \text{ and } c = 750 \text{ lbs./in}^2. \]

Maximum bending moment,

\[ = 750 \times 30 \times 30 \times \frac{12}{8} = 1,125,000 \text{ in. lbs.} \]

Economic depth \[ \sqrt{\frac{60 \times 1,125,000}{18,000 \times 12}} = 18^{\prime} \]

Total depth \[ = 18 \text{ inches.} \]

Effective \[ = 18 - 2\frac{1}{8} = 15 \frac{1}{8} \text{ inches.} \]

Approximate \[ J_d = 15 \frac{1}{8} - 4 \times 6 = 13 \frac{1}{4} \text{ inches.} \]

Area of tensile steel \[ A_t = \frac{1,125,000}{18,000 \times 13 \frac{1}{4}} = 5.5 \text{ ins.}^2. \]

Depth of neutral axis,

\[ n = \frac{(2 \times 5 \times 15 \times 15) + (60 \times 4 \times 4)}{2(5 \times 15 + 60 \times 4)} = 5.2 \text{ ins.}^2. \]

\[ J_d = 15 \frac{1}{4} - \frac{3 \times 5 \times 2 - 2 \times 4}{(2 \times 5 \times 2) - 4} = 14.0 \text{ ins.} \]

\[ e = \frac{1,125,000}{5 \times 2 \times 11 \times 0} = 15,500 \text{ lbs./ins.}^2. \]

\[ Shear \text{ force } = 750 \times 15 = 11,250 \text{ lbs.} \]

Shear stress \[ = \frac{11,250}{12 \times 14} = 70 \text{ lbs./in}^2. \]

Shear reinforcement is thus worked out as explained under section IX.

B. L = Beam Design—As L = beam is of the shape shown in the figure, it is commonly used either as a bressumner along the edge of a floor or as an outside beam along the edge of a deck slab. The design of L-beam is similar to that of a T-beam, and the same formulas apply. The allowable breadth Bs of compressive flange is, however, different. As the section is not symmetrical a bending moment applied is a vertical plane produces tension. This is resisted by the floor slab and other beams which frame into such a beam. To help resist the twisting moment, top steel is provided along its length and also it needs a fairly strong system of stirrups. The breadth of the compression flange in L-beam is taken the least of the following three:

(i) \[ \frac{1}{6} \text{ effective span}. \]

(ii) \[ b + \frac{1}{8} \text{ (distance between ribs)}. \]
(iii) $b = 0.4$ in.

*Example 8*—Design a L-beam used on the roof of a staircase shall given the following data. Take

$t = 16,000$ lbs/in.² and

$c = 600$ lbs/in.²

Length of span $= 11$ ft.

Effective span $= 12$ ft.

Spacing between beams $= 10$ ft.

Load coming on slab $= 265$ lbs/ft.²

$n = \frac{16}{1} = 0.9$

Load on shorter span

\[ \frac{205 \times 10 \times 10}{4} = 5,140 \text{ lbs.} \]

Load coming from longer span

\[ = 5,140 \times \frac{2 - 0.9}{0.9} = 6,270 \text{ lbs.} \]

Therefore, load coming on to beam $= 6,270$ lbs.

Load of beam

\[ = 6 \times 12 \times \frac{150 \times 12}{144} = 900 \text{ lbs.} \]

Total load

\[ = 7,170 \text{ lbs.} \]

Maximum bending moment

\[ = \frac{7,170 \times 12 \times 12}{10} = 10,100 \text{ in. lbs.} \]

Economic depth

\[ = \sqrt{\frac{60 \times 16,000}{600 \times 100}} + 1.25 = 10'' \]

Depth of stem

\[ = 10 + 1.5 - 8.5 = 3'' \]

Adopt $6'' \times 12''$

\[ d = 0.4 \text{ in.} \]

\[ \text{Jd} = 11.5 - \frac{4.8}{3} = 9.9'' \]

\[ t = \frac{104,000}{9.9 \times 1} = 10,290 \text{ lbs/in.²} \]

\[ c = \frac{10,290 \times 4.82}{15(12.5 - 4.82)} = 490 \text{ lbs/in.²} \]

Shear force

\[ = \frac{7,170}{2} = 3,585 \text{ lbs.} \]

Shear stress

\[ = 3,585 \div 12 \times 9.9 \approx 30 \text{ lbs/in.²} \]

Bond stress

\[ = \frac{3,585}{735} \times 3.14 \times 9.9 \approx 90 \text{ lbs/in.²} \]

Use main bars $1''$ dia., 2 Nos. and $3/8''$ dia. shear links.
SECTION IX—SHEAR STRESS IN BEAMS.

Inclined Bars—It is the usual practice to bend the bars from tensile flange into compressive flange to take shear either at 45° or 30°. This practice is possible because the bending moment diminishes from the mid span towards the support, and the number of main tensile bars to resist bending moment are reduced near the ends. The vertical shear is resisted by the vertical component of the stress in the inclined bars which develop full shear strength being adequately anchored at both ends to afford them the necessary grip length. The zone of action of an inclined bar between two struts extends over the distance on either side of it equal to the horizontal projection of the inclined concrete struts. Any vertical plane in this zone cuts either one diagonal steel tension bar or one imaginary concrete strut. As the shear is maximum wear the support the first inclined bar near the support should have its bottom bent at a distance not greater than (2d) from the centre of the support, if the inclination is 45° or (2·73d), if the inclination is of 30°. The horizontal distance between any two steel bars should not exceed 2d, 2·73d, accordingly as the inclination is 45° or 30°.

If the shear force is so heavy that extra inclined bars are required, and if it is possible to spare more bars to be bent, what is known as continuous shear system of inclined bars is adopted. In this system one or more inclined bars are provided at equal intervals between the inclined steel bars of the single shear system, so that any vertical section on the zone cuts two inclined bars or one inclined bar and one inclined imaginary concrete strut. This system increases the shear strength to double or treble accordingly as one or two additional inclined bars are introduced between those of the single shear system. When the pitch of bent up bars exceeds the lever arm the system ceases to be a continuous system, and becomes an intermittent system.

Vertical Stirrups—Vertical stirrups are used as shear reinforcement instead of or together with bent up bars. Where stirrups are used in conjunction with bent up bars the total shear strength is the sum of the separate shear strengths of each. Stirrups must not be spaced farther apart than a distance equal to the lever arm. In the mid-span portion of a beam the concrete alone may be strong enough to take the shear without shear reinforcement. Light stirrups at wide spacing are, however, inserted in the mid-span portion to act as ties.

Example 9—Design the shear reinforcement for a T-beam with slab depth 4 inches and a rib 14 inches deep and 10 inches wide, given the following loading and dimensions:—

Show the arrangement of steel in slab and T-beam.

(See diagram No. 1.)

Effective span = 21·25 ft.

n = 6 inches.
d = 15·75 inches.
a = 13·75 inches.

Tensile steel = 8 Nos. 1" dia. bars.

Total loading on T-beam = 2,000 lbs. per ft. run.

Maximum shear force on a T-beam = \( \frac{2,000 \times 21\cdot25}{2} \)

Shear stress = \( \frac{S}{b \times a} = \frac{21,250}{10 \times 13\cdot75} \) = 155 lbs. per sq. in.

This exceeds permissible shear stress of 75 lbs. /in².

Safe shear strength of concrete = 75 \times 10 \times 13\cdot75 = 10,300 lbs.

Bars to be bent up—

Maximum bond stress at support = \( \frac{S}{E \cdot o \cdot Jd} \) and this must not exceed 150 lbs./in². Hence sum of the perimeters of tensile
bars at support is \( E = \frac{21,300}{13.75 \times 150} = 10.32 \) inches.

No. of tensile bars to be run through to support in bottom of beam
\[ \frac{10.22}{3.142} = 4 \text{ bars.} \]

Therefore, bars available for bending up for shear = 4 Nos.

**Curvature of bars—**
From table (8) for freely supported span \( L = 21.25 \)

<table>
<thead>
<tr>
<th>Number of bars in bottom of beam</th>
<th>Maximum distance of bend from centre of support</th>
</tr>
</thead>
<tbody>
<tr>
<td>7 (1 bent up)</td>
<td>( 0.3 \text{ L} = 6' )</td>
</tr>
<tr>
<td>6 (2 bent up)</td>
<td>( 0.25 \text{ L} = 5' )</td>
</tr>
<tr>
<td>5 (3 bent up)</td>
<td>( 0.19 \text{ L} = 4' )</td>
</tr>
<tr>
<td>4 (4 bent up)</td>
<td>( 0.15 \text{ L} = 3' )</td>
</tr>
</tbody>
</table>

To form a "continuous system" pitch of bars must not exceed lever arm. Try a pitch of 12".

**Strength of bent up bars—**
Bond distance for 1" dia. bar = 45 d = 45" in case the bar is fully stressed to 18,000 lbs./in. Anchorage value of a hook = 16 D = 16 ins. If the bars nearest support is bent up at 3 ft. from centre of support, bond distance of this bar = 16 ins. (hook) + 26 inches (hook to top head) + 6 inches (down to neutral axis) = 48". Thus full bond distance is available. Four bars will, therefore, be bent up singly at 45" at 1 ft. centres between 3 ft. and 6 ft. from centre of support to form a continuous system.

Shear taken up by bent up bars using formula \( V(4) \)

\[ = \sqrt{2} \times \frac{13.75 \times 18,000 \times 0.78}{12} = 23,000 \text{ lbs.} \]

Actual maximum shear = 21,250 lbs.

Try 3" dia. stirrups in the end of the beam beyond bent up bars.
\[ A_w = 0.44 \text{ sq. inches. four leg stirrups.} \]

Pitch \[ = \frac{a}{8} \]

\[ t_w A_w = \frac{13.75 \times 18,000 \times 0.441}{21,250} \]

Use 3/8" four-legged stirrups at 5" centres.

As effective depth \( d = 15.75" \), this pitch does not exceed the recommended maximum pitch of shear stirrups of 1/2 effective depth. In the beam where concrete shear strength is adequate, stirrups are spaced a distance apart approximately equal to the effective depth to serve as ties and to avoid a sudden change from one type of shear reinforcement to another.

**SECTION X—DESIGN OF STAIRCASES**

A staircase may be designed (a) slab spanning on stringer beams or walls, (b) slab spanning on trimmer beams, (c) steps as cantilever over wall.

In case of 1st arrangement, the weight of the staircase is carried by stringer beams. For a non-residential building the load to be carried is taken as 100 lbs./sft. with a minimum of 800 lbs. per foot width of the whole tread. For a staircase inside residential building the superimposed load may be taken as 50 lbs./ft² with a minimum of 50 lbs. for the whole tread.

In the 2nd arrangement the staircase is designed as a slab spanning in the long direction. The span is taken as the horizontal projection of the staircase between trimmer beam. The waist is made equal to the minimum thickness of the slab so designed and is usually about 6 inches. As the load is inclined to the axis of the staircase a direct thrust is produced in addition to the bending which is small, and is usually neglected. Reinforcement for negative bending moment due to continuity at the trimmer beams is provided. See diagram 4 (b).
In the 3rd arrangement, steps are fixed inside the wall to act as cantilever.

**Example 10**—Design a staircase with 5 feet wide steps—

(i) Cantilevering over the walls.
(ii) Steps spanning over the two walls.

**Permissible load**: $16,000 \text{ lbs. /in}^2 = 600 \text{ lbs. /in}^2$.

(i) **Width of staircase** = 5 ft.

- **Length of staircase** = 12 ft.
- **No. of steps** = 13 Nos.
- **Rise of step** = 7 inches.
- **Tread** = 11 inches.

Consider 1 foot of staircase length as a cantilever.

**Dead load of cantilever**

$$\frac{(7+14)}{2} \times 11 \times \frac{150 \times 5}{144} = 600 \text{ lbs.}$$

**Live load at 100 lbs./ft²** = 500 lbs.

which is less than 800 lbs. and hence total live load is taken as 800 lbs.

**Total load** = 1,400 lbs.

**Maximum bending moment**

$$= \frac{1,400 \times 5 \times 5 \times 12}{2} = 46,200 \text{ lbs. inches.}$$

**Effective depth** $d = \sqrt{\frac{46,200}{12 \times 63}} = 6'4''$

**Take total depth** = 7'' and

**Effective depth** = 0''

At $= \frac{46,200}{16,000 \times 0.88 \times 6} = 6.54 \text{ in.}^2$

**Bond stress** = $\frac{1,400}{3 \times 1.57 \times 0.88 \times 6} = 59 \text{ lbs./in}^2$

Provide $\frac{1}{4}'' \text{ dia. 3/4'' c/c as main bars and 3/8'' dia. 9'' c/c as temp. steel.}$

The steps are cantilevering over wall 27'' thick.

In order to check for overturning, the weight of the wall coming over the step is calculated. Say 9,305 lbs.

**Counterbalance moment** = $9,305 \times \frac{27}{2} = 120,000 \text{ lbs. in. as against 46,200 lbs. in. Hence the design is safe.}$

(ii) Consider the steps to be simply supported on stronger beams. **See Diagram 4 (a).**

**Effective span** = 5.5 feet.

- **Load of each step**

  $$= \frac{(5+12)}{2} \times 11 \times \frac{150}{144} = 97 \text{ lbs. per ft. length.}$$

- **Load on 5 gear length** = 5 x 97 = 485 lbs.

- **Live load** = 800 lbs.

- **Maximum bending moment**

  $$= \frac{1,285 \times 5 \times 5 \times 12}{8} = 10,601 \text{ in. lbs.}$$

$$d = \sqrt{\frac{10,601}{12 \times 95}} = 3.09''$$
Take effective depth  =  3\frac{1}{2} \text{ inches.}
Total depth  =  4\frac{1}{2} \text{ inches.}

At  10,000 \times 3 \times 0.88 = 25 \text{ in.}^2

Provide 1/2" bars at 6" centres and 3/8" bars +12" centres as temperature reinforcement.

SECTION XI—DESIGN OF HOLLOW TILE FLOORS

Hollow clay tiles are used extensively for floors carrying moderate super-imposed loads, particularly in steel framed buildings. In a normal reinforced concrete slab, the concrete below the neutral axis being in tension is neglected in design, as it serves no useful purpose as far as strength is concerned. Part of this concrete is therefore replaced by a light and cheap material which allows saving on both the weight and cost of the floor. The hollow tiles achieve this object. These tiles are ribbed on its outer surface, and thus keys in with the surrounding concrete. A common size of tile is 12" x 12" or 9" x 9" with heights varying from 4" to 16". The thickness of the concrete cover over the tiles is not made less than 1\frac{1}{2}". The breadth of the rib 'd' is not kept less than 3\frac{1}{2}" inches when one bar per rib is used, and not less than 3\frac{1}{2}" when 2 bars per rib is used. The largest convenient size of bar is 3/4" or 7/8". In a continuous floor, alternate bars are bent up at the supports to provide for negative bending moment. Alternatively separate steel may be provided on the top of the floor to cater for negative bending moment. Some little distance, say about 6" from the supports the tiles are stopped off, and from this point to the support, the floor is made solid, which gives extra strength to take larger shear forces at the support. At the first and last tile of each row a stop is provided by placing a flat tile to cover the cavity at the end of the tile. See diagram 5 for detail of hollow tile slab.

To design a hollow tiled floor, calculate maximum bending moment as continuous spans
\[ M = \frac{wL^2}{10} \]
For a preliminary design taking the dead weight as 80 lbs. per cubic feet of the floor find total depth 'd' for a normal slab. Then decide the depth of tile and floor to be adopted and calculate the actual floor weight, and total bending moments. Assuming a size of bar find the actual effective depth 'd' and calculate depth of neutral axis from formula (i) (4). If the neutral axis lies within the depth of the compression flange, design the floor as a reinforced concrete slab. If the neutral axis lies below the depth of the compression flange, design the floor as series of T-beams whose rib breadth is 'b'. The maximum shear at the support is then checked which is resisted by the area of rib only. The concrete above the tile is provided to take compression in addition to 1/2" of tile top which also takes same compression.

Example 11—Design a hollow tile slab of a floor of a house to span over a number of consecutive 12 feet spans.

Assume 6 inches hollow tile slab.
Super-imposed load  =  80 lbs. per ft.\(^2\)
6" hollow tile slab  =  48 lbs. per ft.\(^2\)
Floor finish  =  10 lbs. per ft.\(^2\)
Ceiling finish  =  6 lbs. per ft.\(^2\)

Total  =  144 lbs. per ft.\(^2\)

With ribs 4" wide and tiles 12" wide the distance between the centres of ribs  =  1 ft. 4 in.

Hence load per rib  =  1 \times 33 \times 144 = 101 \text{ lbs. ft.}

Mid span B. M.  =  \frac{101 \times 12 \times 5 \times 12}{10} = 35,808 \text{ in. lbs.}
Shear force on ribs (at 12" from centre of support).

\[ = 5 \times 191 = 955 \text{ lbs.} \]

With stresses of 18,000 and 750 Lbs./in.\(^2\)

Neutral axis depth \[ = 0 \cdot 39 \times 4 \cdot 89 = 1 \cdot 83 \text{ in.} \]

Shear stress in rib. \[ = \frac{955}{4 \times 3 \cdot 89} = 61 \text{ lbs./in.}^2 \]

(This should not exceed 75 Lbs./inches\(^2\) without shear reinforcement).

\[ A_1 = \frac{35,808}{18,000 \times 3.89} = 0 \cdot 50 \text{ inches}^2 \]

Provide one 3/4" bar.

The neutral axis is less than slab thickness of 2". The moment of
resistance of the compression of concrete \[ MR = 125 \times 16 \times 4 \cdot 60^2 = 44,000 \]
which is ample for the bending moment. There is no need to provide binders
in the ribs. A light mesh may, however, be provided in top slab. One of
the bars in the rib is bent up at each support to provide a nominal amount
of steel at the support.

**SECTION XII—DESIGN OF LINTELS**

Lintels for openings in walls are designed as singly reinforced rectan-
gular beams of breadth 'b' equal to the thickness of wall and depth 'd'
equal to a multiple of 3 inches or whatever depth of brick is being used in
the wall. In case sufficient head-room is not available depth of the lintel
may be reduced by doubling reinforcing it. Due to the arching effect of a
brick wall over a lintel, the dead load of the wall on the lintel is only taken
as net weight of brick work contained in an equilateral triangle whose base
is the effective span of the lintel. The maximum bending moment with
triangular loading is thus \[ \frac{wL^2}{6} \] where \( w \) is the total load. The weight of
equilateral triangle is \[ (\cdot 49L^2 \times t \times 120) \] Lbs. where \( L \) = span of the opening
and 't' is the thickness of the masonry wall. In cases where the length of the
wall does not extend on either side of the opening, the total weight of the
wall on the effective span of the opening may be taken into consideration.
In diagram 5, fig. 1, the lintel carries only the hatched equilateral triangle
of the wall. The other three figs. are special cases of the Rule. In these
the shaded area illustrates the active loads on the lintel girders.

**SECTION XIII—DESIGN OF COLUMNS**

There are two general shapes of columns i.e., rectangular column and
round column. Each of this type has main reinforcement consisting of
longitudinal bars and secondary reinforcement consisting of transverse
links in the rectangular column and spiral reinforcement in round columns.
A column may have to carry load as axial load only or an axial load combined
with eccentric loading which causes compression as well as bending stresses.
The column is short when the unsupported length does not exceed 15 times
the least lateral dimensions and it is long column where the length is more
than 15 times the least width.

Rectangular columns to carry an axial load are normally made square.
Columns under bending may be made with unequal sides, as the larger side
gives increased resistance to the bending moment. Permissible stresses
in concrete in columns depend upon its grade and mix. The maximum
permissible stress due to direct loading is slightly less than that due to bending
and is constant over the whole concrete cross-section. The cross-sectional
area of longitudinal reinforcement varies between 0.8 per cent to 8 per cent
of the gross cross-sectional area of column. From 1 to 2 per cent of steel
is, however, commonly used. The diameter of longitudinal bars varies
from 1/2 inches to 1/4" and transverse bars from 1/4 to 3/16 inches. The
volume of transverse reinforcement should not be less than 0.2 per cent
of the gross volume of the column with 2 per cent of main bars and 0.4 per cent
of column with over 2 per cent of main bars. Diameter of transverse
Take effective depth = 3½ inches.
Total depth = 4½ inches.
At
\[ \frac{10,000 \times 3 \times 0.88}{15,000} = 25 \text{ in.} \]

Provide 1/2" bars at 6" centres and 3/8" bars +12" centres as temperature reinforcement.

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To design a hollow tile floor, calculate maximum bending moment as continuous spans

\[ \text{M} = \frac{wL^2}{16} \]

For a preliminary design taking the dead weight as 80 lbs. per cubic foot of the floor find total depth ‘d’ for a normal slab. Then decide the depth of tile and floor to be adopted and calculate the actual floor weight, and total bonding moments. Assuming a size of bar find the actual effective depth ‘d’ and calculate depth of neutral axis from formula (3) (4). If the neutral axis lies within the depth of the compression flange, design the floor as a reinforced concrete slab. If the neutral axis lies below the depth of the compression flange, design the floor as series of T-beams whose rib breadth is ‘b’. The maximum shear at the support is then checked which is resisted by the area of rib only. The concrete above the tile is provided to take compression in addition to 1/2" of tile top which also takes some compression.

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Assume 6 inches hollow tile slab.

Super-imposed load = 80 lbs. per ft².
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Floor finish = 10 lbs. per ft².
Ceiling finish = 6 lbs. per ft².
Total = 144 lbs. per ft².

With ribs 4" wide and tiles 12" wide the distance between the centres of ribs = 1 ft. 4 in.

Hence load per rib = 1·33 x 144 = 191 lbs. /ft.
Mid span B. M. = \[ \frac{101 \times 12\times 5^2 \times 12}{10} = 35,808 \text{ in. lbs.} \]
Shear force on ribs (at 12" from centre of support).

\[ = 5 \times 191 = 955 \text{ lbs.} \]

With stresses of 18,000 and 750 Lbs./in.²

Neutral axis depth

\[ = 0.39 \times 4.69 = 1.83 \text{ in.} \]

Shear stress in rib

\[ = \frac{955}{4 \times 3.89} = 61 \text{ lbs./in.²} \]

(This should not exceed 75 Lbs./inches² without shear reinforcement).

\[ A_1 = \frac{35.808}{18,000 \times 3.89} = 0.50 \text{ inches}^2 \]

Provide one 3/4" bar.

The neutral axis is less than slab thickness of 2". The moment of resistance of the compression of concrete \( MR = 125 \times 16 \times 4.69^2 = 44,000 \) which is ample for the bending moment. There is no need to provide binders in the ribs. A light mesh may, however, be provided in top slab. One of the bars in the rib is bent up at each end to provide a nominal amount of steel at the support.

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Lintels for openings in walls are designed as singly reinforced rectangular beams of breadth 'b' equal to the thickness of wall and depth 'd' equal to a multiple of 3 inches or whatever depth of brick is being used in the wall. In case sufficient head-room is not available depth of the lintel may be reduced by doubling reinforcing it. Due to the arching effect of a brick wall over a lintel, the dead load of the wall on the lintel is only taken as net weight of brick work contained in an equilateral triangle whose base is the effective span of the lintel. The maximum bending moment with triangular loading is thus \(-\frac{wL^2}{6}\) where \( w \) is the total load. The weight of equilateral triangle is \((431L^2 \times t \times 120) \) Lbs. where \( L \) = span of the opening and \( t \) is the thickness of the masonry wall. In cases where the length of the wall does not extend on either side of the opening, the total weight of the wall on the effective span of the opening may be taken into consideration.

In diagram 5, fig. 1, the lintel carries only the hatched equilateral triangle of the wall. The other three figs. are special cases of the Rule. In these the shaded area illustrates the active loads on the lintel girders.

SECTION XIII—Design of Columns

There are two general shapes of columns i.e., rectangular column and round column. Each of this type has main reinforcement consisting of longitudinal bars and secondary reinforcement consisting of transverse links in the rectangular column and spiral reinforcement in round columns.

A column may have to carry load as axial load only or an axial load combined with eccentric loading which causes compression as well as bending stresses. The column is short when the unsupported length does not exceed 15 times the least lateral dimensions and it is long column where the length is more than 15 times the least width.

Rectangular columns to carry an axial load are normally made square. Columns under bending may be made with unequal sides, as the larger side gives increased resistance to the bending moment. Permissible stresses in concrete in columns depend upon its grade and mix. The maximum permissible stress due to direct loading is slightly less than that due to bending and is constant over the whole concrete cross-section. The cross-sectional area of longitudinal reinforcement varies between 0.8 per cent to 8 per cent of the gross cross-sectional area of column. From 1 to 2 per cent of steel is, however, commonly used. The diameter of longitudinal bars varies from 1/2 inches to \( 1\frac{1}{4} \) and transverse bars from 1/4 to 3/10 inches. The volume of transverse reinforcement should not be less than 0.2 per cent of the gross volume of the column with 2 per cent of main bars and 0.4 per cent of column with over 2 per cent of main bars. Diameter of transverse
reinforcement is 1/4th dia of main bars. Pitch of the links should not be more than the least lateral dimension of column or 12 x dia of smallest longitudinal reinforcement or 12 inches.

The ratio of the effective length 1 to the actual length of the column depends upon the end fixity. The rigidity of reinforced concrete ensures good conditions of end fixity, and the effective length may normally be taken as between 0.75 L and 1.25 L. Each case must be judged on its own merits. The number and size of beams framing into a column at floor level and the type of foundation in the case of ground floor columns will affect the degree of fixity. In a long column, the ratio of effective length to radius of gyration, i.e., L/K should be between 50 and 150 and must not exceed 150. In the case of systematically rectangular column the coefficient obtained from the value of

\[
\frac{1}{d} \quad \text{Shorter side of column.}
\]

(A) Axially loaded column—The safe axial load \( P \) on a column reinforced with longitudinal bars and transverse reinforcement is given by the formula (eq) (1).

Example 12—Design a square column 25 ft. long with both ends fixed in position and direction to carry an axial load of 100 tons.

(i) Effective length of column = 0.75 x 25 = 18.75.

In order to find reduction coefficient the side 'd' of the column must be known. Rough calculations are, therefore, made first neglecting neglecting neglecting

Assume 1% main steel.
Then steel area = 0.01 \( d^2 \)
Safe load \( P \) = \( C(A - Ae) + te \times Ae \)
224,000 = \( (750 \times 0.99 d^2) + (18,000 \times 0.01 d^2) \) lbs.

\[
= (742 + 160) d^2
\]

. . \( d = 15.5 \) inches.

Use 16" x 16" section.

Hence \( \frac{1}{d} = \frac{18.25 \times 12}{16} = 14 \)

Since 1/d is less than 15, reduction coefficient is not applicable.
Steel area = 0.01 \( d^2 = 2.4 \) sq. in.
Use four bars of 7/8 inches dia.
Diameter of link. It should not be less than 1/4 dia of main bars or \( 4 \times \frac{7}{8} = 1/4 \) dia.

Maximum pitch 'p' should not exceed.
(a) Side of column = 16"
(b) 12 x dia main bar = 10½ inches.
(c) 12".

Hence pitch must not exceed 10\( \frac{1}{2} \)".

Volume of links = 0.2 per cent of column.
Concrete cover on main bars = 1½ inches.
Hence length of one side of link = 16 - 3 = 13 ins.
Length of one link = 4 x 13 = 52 inches.
Cross-sectional area of 1/4 inch link = 0.049 sq. ins.
Volume of links (cu. in) = \( 0.049 \times 52 \times \text{Column length in inches} \)
Pitch in inches.

Volume of column (cu. in) = 16 x 16 x Column length in inches.

\[
\text{Volume of links} = \frac{0.049 \times 4 \times 13}{16 \times 16 \times P} = 0.2 \text{ per cent.}
\]

\[
P = \frac{0.049 \times 4 \times 13}{16 \times 16 \times 0.002} = 5 \text{ inches.}
\]

This pitch does not exceed the maximum permissible pitch 10\( \frac{1}{2} \) inches.
Use 1/4 inch dia links at 5 inch pitch.
(B) Column under Eccentric Loading—(i) Eccentric loading on a column is caused by a transverse load such as wind pressure, or by the floors beams framing into the column, or by a load on a bracket attached to the column. The rigidity of a loaded beam at the junction with a column, causes a negative bending moment at the end of the beam which moment is transmitted to the columns and makes them to bend. If the eccentric load P acts at a distance ‘e’ from the axis of the column and if ‘e’ is less than about 0·2d, no tension will occur in the column. The maximum compressive stress on the concrete will occur at the edge near the applied load and is given by formula (vi) (2) and (vi) (3).

(ii) If ‘e’ is greater than about 0·2d a tensile stress occurs in the edge further from the applied load. By assuming likely suitable values for ‘b’ and ‘d’ and knowing the load ‘P’ and its eccentricity ‘e’ from the column axis ; P/bd and e/d are calculated and ‘a’ and ‘r’ are found from the formula (vi) (5). This formula is, however, complicated and does not lend itself to practical design. To find ‘a’, the solution of a cubic equation is involved. Hence for practical use Diagram (2) is employed and percentage of steel is read directly from it. As the graph has been plotted for full permissible compressive stresses of concrete in bending, the effect of reduction co-efficient if applicable, is obtained by dividing the load by the reduction co-efficient, and the column is designed from the graph using this increased value P.

Application of Graph—Work out ‘e’ from the bending moment and the axial load as =M/P. If the load is eccentric then ‘e’ is obtained straightaway. Assume likely dimensions for the sides of the column, and find out its effective length and the reduction co-efficient if necessary. Then find out the equivalent load P = reduction factor. Calculate e/d and P/bd. From the graph read the value of steel percentage 100 r for these values of e/d and P/bd. The steel required is Ac=rbd. Design transverse reinforcement as usual.

Example 13. Design a R. C. C. column 15 feet long with both ends fixed in position but not in direction, and to stand an axial load 100 tons and a bending moment of 400,000 lbs. in permissible compressive stress in bending = 1,000 lbs. in².

C = M/P = \frac{400,000}{100 \times 22\,340} = 1.8 \text{ inches.}

Assume a column of 18" x 16" in section the longer side being in the direction of the bending moment.

\text{Effective length} = \frac{15 \times 12}{16} \text{ which is less than 15. No reduction co-efficient need be applied.}

\frac{e}{d} = \frac{1.8}{18} = 0.1 \text{ d. As ‘e’ is less than 0.2d, no tension occurs.}

\frac{P}{bd} = \frac{100 \times 2,240}{16 \times 18} = 780 \text{ Lbs. /in}^2.

From graph 100 r = 1.5 per cent.

Ac = rbd = 0.15 \times 16 \times 18 = 4.33 \text{ in}^2.

Use six number 1" diameter main bars. The cover to main bars = 1\frac{1}{2} \text{ inches.}

Hence K = \frac{2}{18} = 0.11.

From formulae (vi) (4).

I = \frac{16 \times (18)^3}{12} + [(15-1) \times 4.71 \times 13^3 \times (0.5 - 0.11)^3]

= 7750 + 3250 = 11,000 \text{ in}^4.

C = \frac{2,24,000}{(16 \times 18) \times (14 \times 4.71)} + \frac{1,800 \times 2,240 \times \frac{1}{3} \times 18}{11,000}

= 633 + 330 = 963 \text{ lbs/in}.

This is less than the permissible of 1,000 lbs/in².
Example 14—Design a R.C.C. column of length 15 ft. both ends being fixed in position but not in direction to take an axial load of 50 tons and a bending moment of 360 in tons. Permissible compressive stress in bending = 1,000 lbs/in.

\[
\sigma = \frac{M}{P} = \frac{360}{50} = 7.2 \text{ in.} = 0.4 \text{ d inches.}
\]

\[
P = \frac{50 \times 2,240}{16 \times 18} = 360 \text{ Lbs. per sq. inches.}
\]

From graph 100 r = 2.3 per cent.
Ae = 0.23 \times 16 \times 18 = 6.63 \text{ in.}^2

Use six 1\frac{1}{2} \text{ inches dia bars.}

Check design from formula (vi) (5)

From graph 'n' = 0.66
Concrete cover = 1\frac{1}{2} \text{ inches.}

\[
K = \frac{2.125}{18} = 0.12
\]

From formula vi (5),

\[
P = C \left( \frac{1}{2} \times 16 \times 0.66 \times 18 + 3.68 \times 0.54 \times 14 - 3.68 \times 0.22 \times 15 \right) \frac{0.66}{0.66}
\]

\[
= C (95 \times 18 - 18) = 119 \text{ C.}
\]

\[
C = \frac{50 \times 2,240}{119} = 940 \text{ lbs/in.}^2
\]

This is less than the permissible stress and the design is safe.

(C) Design of column footing—Bearing Pressure—The bearing pressure decides the minimum area of the footing in contact with the ground, and must not exceed the permissible pressure on the ground. Punching shear contracts the depth of the footing. Section of maximum shear is shown on BB.

Bending moment on column footing—A square column of side ‘d’
carries a load P and is supported on a square footing slab of side l. Considering half the footing, column load P/2 may be taken to act at a point distant d/4 from the centre line xx. The resultant of the ground bearing pressure acts at L/4 from xx. Let the bearing pressure on ground = p

Resultant of ground pressure on half the slab = \frac{pl^2}{2}

Maximum B. M. at centre line of slab = \frac{pl^2}{8} (l-d)

Max. B. M. per foot width of slab = \frac{Pl^2}{8} \text{ approximately as ‘d’ is small compared with ‘l’}.

Example 15—Design a R.C.C. Column footing to carry 18” \times 18” column carrying a load of 100 tons and permissible bearing pressure on ground 3 tons per sq. ft. Permissible punching shear = 150 lbs/in.²
Area of footing required for bearing
\[ \frac{100}{3} = 33 \text{ sq. ft.} \]

Try a 6' × 6' slab with its thickness 'ds' inches.

Actual bearing pressure on ground 'p' = \[ \frac{100 \times 2,240}{36} = 6,200 \text{ lbs./sq. ft.} \]

For punching shear from formula—
\[ 100 \times 2,240 = (150 \times 4 \times 18 \times ds) + \frac{(6,200 \times 18^2)}{144} \]
\[ ds = 20 \text{ ins.} \]

B.M. \[ \frac{p1}{8} = \frac{6,200 \times 36}{8 \times 12} = 335,000 \text{ lbs. in.} \]
\[ d = \sqrt{\frac{335,000}{125 \times 12}} = 15'' \]

As 'ds' must not be less than 20 ins., 'd' will be about 18'. Lever arm = 0.87 × 18

\[ A_t = \frac{335,000}{18,000 \times 0.87 \times 18} = 1.19 \text{ in.}^2 \]

To ensure 45 diameters bond distance for the centre of the slab either end of a bar length of bar must be 2 × 45 D inches, i.e., bar diameter must be at least 72/30 = 8 inches, = 3/4".

Use 3/4" dia. bars at 4½'' centres.

CHAPTER III—(Concrete Mix)

SECTION XIV—Quality Control of Cement

(A) Modern concrete control requires correct mix design, accurate batching by weight, and vibration and compaction. The principal object in concrete control is to ensure that the concrete in the finished structure will be uniform, and of adequate quality for the service conditions, at minimum cost. A higher quality than needed will increase costs unnecessarily and too low quality will lead to unsatisfactory service or excessive maintenance cost. The required quality for any given set of service conditions is governed by various qualifications of the final concrete such as, strength, impermeability, appearance, etc. Selection of the right combination of concrete materials contributes much towards providing concrete of the required quality for a given set of job conditions. After the combination of materials has been selected, then the job is to control the quantity and quality of each ingredient from batch to batch as uniformly as practicable. In selecting aggregates for use in concrete, there are many important factors to be kept in mind. For example are they physically active, i.e., do they contain siliceous constituents which will react with alkalis in cement to cause excessive expansion and cracking in concrete? Aggregates which have thermal co-efficients of expansion and contraction differing widely from those of hardened Portland cement paste may result in concrete having low resistance to freezing and thawing. The surface texture of some aggregate particles promote high bond with the cementing matrix, others have smooth, hard surfaces which bond poorly. The size and interconnection of voids and the internal structure of aggregate particles have an important relation to the durability of concrete. The grading of sand used in concrete has a marked effect on its strength, cement and water requirements and durability. Therefore, it is important that the average grading fall within limits known from experience to produce acceptable results. Unless aggregate grading limits are rigidly adhered to and limits on mixing water/cement ratio, slump, vibration, transportation, and curing are rigidly controlled, a high-quality concrete cannot be obtained.

(ii) Placing and compaction—Concrete should be placed as nearly as possible to its final location, and it should not be made to move through the forms. Concrete should be placed in horizontal layers
wherever possible. Each layer should be soft when a new layer is placed upon it. The compaction of concrete may be carried out by hand or mechanical means. Hand compaction is carried out by rodding or ramming. Rodding consists of inserting a suitable rod vertically into the concrete and moving it up and down until the concrete is thoroughly worked into place. Special attention should be given to corners, reinforcement and any awkward spots. Slabs should be tamped with tamping tool which serves the double purpose of finishing concrete to the required level and of compacting the concrete. Compaction by mechanical vibration permits the use of much drier mixers than is possible with hand compaction. The benefits of drier mixers is lower water cement ratio and higher concrete strength. To meet the needs of different types of work various forms of vibrating equipment are available. The higher the frequency of vibration the shorter the time required for full compaction. The minimum frequency for effective compaction is taken as 3,600 R.P.M. and practically all standard equipments work at this or higher frequencies. Internal vibrators should be used vertically and should penetrate the full depth of lift. They should be both inserted and withdrawn, slowly and be removed from the concrete as soon as cement paste starts to appear on the surface. Form work should not be touched with a vibrator which should be kept a few inches away from it. Concrete placing should be continued without avoidable interruptions until the placement is completed or until satisfactory construction joints can be made. It should not in any case be deposited faster than the vibrator crew can properly consolidate it. As long as the rise of concrete in the forms does not exceed about five feet per hour in warm weather, and three feet per hour in cold weather, concrete hardens at a sufficient rate to permit placement to any height in forms without creating excessive fluid pressures. Limiting temperature for concreting is 40 degrees F when temperature is falling; and 38 degrees F when temperature is rising. Higher temperature increases the water requirement of the mix, and reduces its ability to resist weathering.

(iii) Water Cement Ratio—The well-known water cement ratio “law” states that with given materials and conditions of test the ratio of the quality of mixing water to the quantity of cement alone determines the strength of concrete so long as the mix is workable. If the aggregate is stronger than the cement paste, and there is sufficient cement to surround the aggregate and full compaction is obtained the strength is primarily dependent upon the water cement ratio. The water required for reaction with cement is about 20 per cent, i.e., a water cement ratio of 0.2; any extra water used is to lubricate the mix sufficiently to be worked in position. It is exceedingly difficult to obtain anything like full consolidation with a water cement ratio of 0.2. In the field it is possible to compact, by machine, concrete with a water cement ratio of 0.4 and by hand a concrete of water cement ratio of 0.5, though more frequently it is found to vary from 0.6 to 0.7.

(iv) Curing—Concrete hardens as a result of the chemical reaction between the cement and water which process continues so long as the conditions such as suitable temperature and the presence of moisture are available. Nearly all concretes contain water sufficient for chemical hydration, and the problem of curing is the problem of compensating the loss of water by evaporation. There are two stages in curing, the initial stage when the concrete is not firm enough to support anything on its surface and the concrete has to be protected from wind, sun or rain by some covering such as tarpaulins held above the concrete on a light frame work, and the final stage, when the concrete has hardened sufficiently for it to be covered with wet sand, Russian canvas, waterproof paper or similar materials. The length of curing time depends upon the weather conditions and the type of cement used. For rapid hardening cement the normal curing period is seven days, and for ordinary cement it is 14 days. When concrete is exposed to freezing it should be protected for not less than 72 hours so that it is maintained at a temperature of at least 56° F. In severe hot weather concrete should be shielded from the direct rays of the sun for at least 3 days after placement. Where concrete
has hardened sufficiently to resist damage any of the following methods may
be used for curing.—

(a) Ponding—Cover the surface of the concrete with water to a depth
of one to two inches, the water being retained by small clay dams built
around the edges of the slab.

(b) Periodical spraying with water—The spraying is repeated at such
intervals as are necessary to ensure that concrete is kept constantly moist
during curing period.

(c) Covering with a 2 to 3 inches of wet sand, earth or saw dust.

(d) Covering with hessian, sacking or bundles of straw. These cover-
ings are kept constantly moist during curing period.

(B) Field tests for quality—(a) Aggregate—(1) The aggregates are
rubbed in hands, when dirt will show up. Also stir a handful of sand or stone
in a glass jar containing water. The sand or stone will settle and the presence
of sand will be clear from the colour of the water. Shake some of the sand
in a white glass bottle with a 3 per cent solution of caustic soda, and allow
the mixture to stand for 24 hours, the amount of solution having about the
same bulk as that of the sand. If the solution turns marked yellow or brown,
it indicates the presence of organic matter.

(b) Reinforcement—The bar is doubled over either by pressure or by
blows from a hammer until internal radius is not greater than 14 times the
thickness of the test piece, and sides are parallel. The crack should not
appear at the bend.

(c) Consistency—Proper consistency of cement concrete mix is the
degree of its density and solidity at which it is workable and possesses
maximum strength. Consistency can be indicated by a slump test. Slump
mould is a frustum of cone made of black iron sheet with dimensions as shown
in the diagram No. 3. It is placed on a flat iron sheet. To carry out the
slump test, the concrete is filled into the mould in 4 layers. Each layer is
lightly proded with a 1/4 diameter bar. The top layer is struck off level
with the top of the mould. The mould is then withdrawn by means of the
handles provided on it by steady upwards pull. The concrete pile will settle
to an extent depending on its consistency. The amount of the settlement
below the height of the mould is termed the "slump" and is measured in
inches. The bottom and top of mould shall be open, parallel to each other
and at right angles to the axis of the cone. The internal surface shall be
smooth and thoroughly clean and dry. Care shall be taken that representa-
tive sample is taken. The consistency is recorded in terms of inches of sub-
sidence of the specimen during the test.

(d) Test of Cement Concrete—In order to find the compressive stress
of the concrete a test specimen of 6 inch cubes is made from the representativ-
e sample and the cubes after proper curing are sent to the laboratory for crush-
ing test. But the crushing test is by no means the only criterion by which
to judge the safety. In many cases the tensile strength is just as
important. If a concrete has a high tensile strength, its compressive
strength is bound to be good but the reverse is not necessarily true.
In these circumstances, the tensile strength gives a much better indication
of quality than crushing strength. It gives a measure of the cohesion,
of the adherence of the aggregate particles, of the plasticity and
consequently of the adaptability and the resistance to cracking which it is
important to avoid. For carrying out a test for tensile strengths, briquettes
of concrete are prepared in the field, and after proper curing are sent to the
Laboratory for test.

SECTION XV—DESIGN OF CONCRETE MIXES

The problem of the design of a mix for a given purpose may be reduced
in its simplest form to the question of obtaining at the lowest cost by a suit-
able choice of materials and of the proportions in which they are used, a
concrete of the required strength and workability. The accompanying table 12
provides a means of arriving at a reasonably satisfactory and economical
choice of proportion. The method of design may be summarised as fol-
lowa.
Minimum strength of the concrete for a particular job is specified. Allowance is then made for the normal variation in strength on the field. The amount of this variation depends upon the accuracy of the batching and central operation and on the uniformity of raw materials used. The minimum strength of concrete that may be normally expected to occur under volume batching of aggregates and fair field supervision may not be taken less than 50 per cent of the average strength. The second step is to select water/cement ratio necessary to produce required average strength. This is obtained from Table 12 which shows the relationship between water/cement ratio and strength for ordinary cement, when the concrete is fully compacted and cured at normal temperature. The third step is to decide degree of workability necessary for full compaction under particular conditions. For normally reinforced work without vibration or heavily reinforced work with vibration a "medium" workability with slump varying from 2 to 4 is satisfactory and may be taken. The fourth step is to determine suitable aggregate/cement ratio required for medium degree of workability and the known water/cement ratio. Table 10 gives these ratios for different gradings from where most suitable value is picked up. The percentage of gradings from the graph corresponding to the ratio is then read. The fifth and last step is to make analysis of the available fine course aggregates and combine them in such proportion as to give an overall gradings similar to the one required. In order to achieve this end it may be necessary to reduce sand by about 10 per cent of the total aggregates or increase it by certain extent to produce a grading approximating to the one required.

Assume that the aggregates are river sand and river gravels of 3/4" maximum gauge and irregular in size and shape. Assume the sieve analysis of sand and gravel is as tabulated below:

<table>
<thead>
<tr>
<th>B.S. Sieve</th>
<th>Percentage passing</th>
<th>B.S. Sieve</th>
<th>Percentage passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td></td>
<td>Sand 3/16&quot;</td>
<td>Nil.</td>
</tr>
<tr>
<td>No. 109</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 32</td>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 25</td>
<td>45</td>
<td>3/32&quot;</td>
<td>49</td>
</tr>
<tr>
<td>No. 14</td>
<td>67</td>
<td>3/32&quot;</td>
<td>109</td>
</tr>
<tr>
<td>No. 7</td>
<td>87</td>
<td>3/4&quot;</td>
<td></td>
</tr>
<tr>
<td>3/16&quot;</td>
<td>160</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

When sand and gravel are mixed the sieve analysis shows the following gradings:

<table>
<thead>
<tr>
<th>B.S. Sieve</th>
<th>Percentage passing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Aggregates containing 30% of sand</td>
</tr>
<tr>
<td>No. 109</td>
<td>0.6</td>
</tr>
<tr>
<td>No. 32</td>
<td>3.0</td>
</tr>
<tr>
<td>No. 25</td>
<td>13.5</td>
</tr>
<tr>
<td>No. 14</td>
<td>20.1</td>
</tr>
<tr>
<td>No. 7</td>
<td>20.1</td>
</tr>
<tr>
<td>3/16&quot;</td>
<td>30.6</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>36.0</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>190.0</td>
</tr>
</tbody>
</table>
Comparison of these analyses with the figures obtained from the curve and the best combination is obtained. Batching proportion is then weighted by weight and converted to proportions by volume.

Example 16—Design a cement concrete mix using ordinary Portland cement for the construction of a slab and T-beam floor to have a minimum strength of 2,000 lbs. in. in 28 days. Volume batching and hand action are being done and a fair supervision is available.

Required average strength after 28 days taking minimum strength as 50 per cent of the average.

To attain a strength of 4,000 lbs./in.² after 28 days = 4,000 lbs./in.² a water cement ratio of 0.6 is obtained from the table 10.

Considering that the reinforced concrete is being mixed under normal conditions without vibration, the workability of the mix required will be 

As the aggregates used are 3/4" irregular gravel, Table 12 gives the gradation ratio required for each of the four gradings shown on the diagram graph for medium workability and water cement ratio of 0.6. These 

| Grading No. I | Grading No. III | 6:1 |
| Grading No. II | Grading No. IV | 5.6:1 |

Grading No. II is the most economical in cement, and so, grading of combined aggregates should be made to approximate this grading. The analysis of grading as obtained after mixing 35 per cent of sand tally with this grading. Gradings obtained from curve II are:

<table>
<thead>
<tr>
<th>D. S. Sieve</th>
<th>Percentage passing</th>
<th>D. S. Sieve</th>
<th>Percentage passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>.100</td>
<td>6.8</td>
<td>2/16&quot;</td>
<td>35</td>
</tr>
<tr>
<td>.42</td>
<td>3.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>.25</td>
<td>14.0</td>
<td>3/8&quot;</td>
<td>55</td>
</tr>
<tr>
<td>.14</td>
<td>21.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.7</td>
<td>38.0</td>
<td>3/4&quot;</td>
<td>100</td>
</tr>
</tbody>
</table>

Batching Proportions.

1 part of cement is mixed with 6 parts of aggregates. As batching is no separately for sand and gravel in the field.

Sand required = $\frac{6 \times 35}{100} = 2.10$

Gravel required = $\frac{6 \times 65}{100} = 3.70$

The proportion of concrete mix by weight is 1:2.1:3.7.

In order to convert the proportion by weight to proportion by volume, quantities of gravel and sand are determined by actual experiment. Let these 100 lbs./ft. 3 and 116 lbs./ft. 3 respectively for gravel and sand.

Therefore proportion by volume is:

- Cement = 1 cwt. = 1
- Sand = $\frac{2.1 \times 112}{116} = 2$
- Gravel = $\frac{3.7 \times 112}{109} = 3.8$

i.e., 1:2:3.8.
SECTION XVI—SPECIAL TYPES OF CONCRETE

(A) Light Weight Concretes—A lightweight concrete is one in which lightness is obtained by the presence of voids provided by leaving the interstitial spaces between the aggregate particles unfilled or by the cells of the vesicular aggregate or by the formation of gas bubbles in the cemenitious matrix. Such concretes are, therefore, called (i) "no-fines" concrete (ii) the lightweight aggregate concrete and (iii) the aerated concretes. The lightweight concretes are used for their thermal insulating properties and low density.

(i) No-fine Concrete—One method of reducing the weight of concrete is to use aggregate in which the particles are substantially of the same size only sufficient cement being used to provide a thin coating to the particles leaving the interparticles voids unfilled. "No-fines" concrete of this kind is being used for domestic buildings. It is normally made with gravel or crushed stones graded between 1/4" and 3/4" gauge. The density of such a concrete is about two-thirds to three-quarters of that of a normally graded concrete made with the same aggregate. An external rendering on such concrete is essential owing to the open texture. Drying shrinkage and expansion on wetting are generally low, and strengths adequate for normal houses can be obtained.

(ii) Lightweight Aggregate Concrete—In its idealised form, a lightweight aggregate concrete relies entirely upon the presence of cells or voids within the aggregate to provide the necessary lightness. In some cases only the coarse aggregate, e.g., clinker saw dust, and in others, both coarse and fine constituents are lightweight.

(iii) Aerated Concrete—The aeration of concrete is achieved by gas generation by adding aluminium powder which when mixed with the wet cement causes the evolution of hydrogen, the bubbles of the gas being entrapped in the mass and causing it to expand. It may be obtained by adding foaming agents to the mixing water and on vigorously working the mix it is possible to cause sufficient aeration to produce lightweight concrete.

(B) Waterproof Concrete—Concrete is often required in structures to retain or keep out water. In all these structures, cement concrete is made completely waterproof. In order to achieve this end the factors to be considered are porosity and permeability. Porosity depends on the amount of voids while permeability depends upon the size of the voids and their intercommunications. Those voids in the concrete are due to entrained air and water. In dry concrete the air filled voids are larger while in wet concrete the water filled spaces are greater. The tightness is thus obtained by grading the aggregate and proportioning the cement to obtain a dense concrete. The densest wet portland cement mortar has about 40 to 45 per cent voids, but is absolutely impervious owing to the voids being very small and uniformly distributed. Water tightness is also obtained by the following expedients:

(a) Enriching the concrete with cement.
(b) Gravel makes more watertight than broken stone.
(c) The larger the size of aggregates the less the permeability. Grading must be such as will produce a completely dense concrete.
(d) Water cement ratio should not be greater than 0.54.
(e) Segregation must be avoided. Compaction must be complete and use of vibrations is most desirable. Curing should be carefully carried out and construction joints should be avoided.
(f) By mixing waterproofing materials such as slaked lime, pozzolan cement or iron oxide which generally act as pore fillers, wetting agents or water repelling agents.
(g) Applying a waterproofing coating to the concrete after it is placed. This coating may consist of a wash of alum and soap or a bituminous shield, or a cement coating applied to concrete when yet green, such as 1:2 with steel travel, which gives a glossy finish and prevents external pores.
SECTION XVII—JOINTS IN REINFORCED CONCRETE.

(A) Expansion Joints—Expansion joints in R.C. work are necessary due to changes in volume of concrete caused by shrinkage during hardening and drying and temperature changes. The cracks occurring due to such changes are prevented by restricting the length of the concrete cast in any one direction. These joints are so designed that the necessary movement occurs with the minimum resistance at the joint. The structure adjacent to joints shall preferably be supported on separate columns and walls. Reinforcement does not extend across the joint and the break between the two sections is complete. At expansion joints, the design of slabs must provide sufficient bearing area on supports as well as for extra bending and shearing strength required to compensate for the break in continuity. Whenever expansion joints are provided in the main structure such joints must be provided in the concrete flooring immediately above them. One of the first things to consider about expansion joints is that of the best location for them from the standpoint of their proper functioning. They should be provided at points where concrete will tend to crack if shrinkage and temperature deformations are restrained or prevented. The second consideration is that of co-ordination with the pouring schedule and avoidance of extra construction joints. The third matter is that of satisfactory details.

Where there is a considerable shearing force at the joint, a keyway joint is used with a space between the abutting ends of the concrete. This can be provided readily by the use of premoulded mastic fillers. The edges of the keyways should be beveled slightly and they should be coated with mastic paint which will break the bond but is not thick enough to destroy the bearing value of the key. When the joints are likely to leak, they are sealed with copper flashing of 10 oz. This copper is folded into the joints so as to permit it to open slightly without rupturing the flashing, and is also strong enough to hold its position during the placing of the concrete. Wherever it is possible to do so, expansion joints are kept entirely open with an air space of 1 to 2 inches between the concrete sections. This is especially desirable where considerable motion occurs. V-cuts 1" to 2" are used at the joints to guard against spalling of the edges because of the compressive forces which are caused at the joints by expansion. A sliding joint is provided to form a horizontal joint entirely separating the structure from the wall. Under each of beam the bearings are slightly raised, and under these four thin rectangular plates of metal of suitable size are placed; the two outer plates being generally of steel about 3/8th inch thick. They need not be attached either to the beam above or the base below, or in any way to one another. The edges of the plates should be wrapped in grease paper to avoid contamination of ends by cement mortar. When movement takes place, the two upper plates slide over the two lower plates. See diagram (5) which shows three different types of expansion joints for slabs.

Asphalt is found to meet the essential requirements for a joint filler. Pure asphalt of the right grade is the best filler but where it is considered unsightly, a patent asphalt filler may be used. Certain patent premoulded fillers are manufactured which have the great advantage of being easily handled, and installed, requiring no heating plant, causing no interruption in the work and reducing the waste through handling to the very minimum. These premoulded fillers are available in various thickness from $\frac{1}{8}$" to 1" and in varying widths and lengths.

(B) Construction Joints—Construction joints are joints in a structure caused by a stop in the placing of concrete. Failure to make such joints correctly will result in the formation of a line of cleavage through the concrete mass, and may be the cause of subsequent failure of the structure. When construction joints are unavoidable, concreting must be stopped as near as possible at the centre of the beams or slabs where shearing stresses are small and where plane is normal to the stress and at right angle to span. If this cannot be done the joint must in any case be within the middle third of the beam or slab. A column should be finished with a level surface a few inches below junction with beam. The construction should coincide with the structural joints wherever possible. A key shall be made where resistance to horizontal shear is essential and vertical construction joints shall have
special reinforcement placed at right angles to joint and extending for 4 diameter on either side. When joining new to old concrete which has already full or partially set, the necessary precautions should be observed in order to obtain through adhesion between the two, and to obviate the risk of the joint opening by unequal contraction or drying. The concrete which will be immediately in contact with the old work shall not be so dry that the bottom of new layer will remain porous, nor so wet as to cause the formation of laitance segregation or excessive shrinkage in hardening. In all cases, construction joints are cleaned thoroughly before the next pour is made. All laitance is removed, using wire brushes, water under high pressure, and other means. It is often desirable to coat the joints with a little mortar just prior to the placing of the concrete upon it. Layer of concrete that have to be left incomplete should be finished against a properly built stop board to ensure a clear vertical face. The concrete shall be well rammed against the stop board. The practice of allowing concrete layers to taper off in a haphazard manner is bad and should not be allowed.

CHAPTER IV—(Use of Tables and Graphs)

Table No. 1—This table gives the areas, perimeters and weights of both round and square bars which are used in the design of reinforcement.

Table No. 2—This table gives spacing of round bars in slabs, given diameter and sectional area of steel per ft. of slab.

Table No. 3—This table gives the bending moment co-efficients for slabs spanning in two directions at right angles simply supported on all four sides and also for rectangular panels supported on four sides with provision for portion at corners.

Table No. 4—This table gives the super-imposed load on floors for various classes of loading.

Table No. 5—This table gives total reduction of super-imposed loads on columns which support the weight coming from a number of floors.

Table No. 6—This table gives bending moment co-efficients for continuous beams and slabs with various Nos. of spans and conditions of loading with ends freely supported.

Table No. 7—This table gives the reduction co-efficients for stresses in long columns as used in design.

Table No. 8—This table gives the positions for curtailments of tensile bars for provision of reinforcement for shear.

Table No. 9—This table gives the size and spacings of shear binders for various values of shear occurring in beams. It also gives the value of shear resistance provided by inclined bars.

Table No. 10—This table gives the permissible stresses used in design of R. C. Concrete.

Table No. 11—This table gives the size and provision of reinforcement in T-beams required for various bending moments.

Table No. 12—This table gives the water cement ratio and aggregate cement ratio for medium workability. It also gives a graph for gradings of aggregates to be used in design of concrete Mix.

Diagram 1—This diagram shows the distribution of Tensile and Shear reinforcement in a slab and T Beams.

Diagram 2—This diagram gives the curves to obtain depth of neutral axis and percentage of steel in design of columns eccentrically loaded.

Diagram 3—This fig shows the apparatus for carrying out the slump test of the cement concrete in the field and also indicates how this test is to be carried out.

Diagram 4—These figs show details of staircases and three types of expansion joints usually provided on floors.

Diagram 5—This diagram gives details of hollow tile roof slabs and different arrangement of lintels supporting brick walls.
CONTINUOUS SLABS

Clear Span '70'

(a) Cross section T-Beam and slab (at mid-span of beam)

CONTINUOUS SLAB - BAR ARRANGEMENT

Bond distance

Bond distance

(0.75) or (0.25)

(a) Arrangement 1

Plan

DESIGN OF SHEAR REINFORCEMENT FOR T-BEAM

2 cover

8/10 deep slab

Neutral Axis

Centre of Support

4/10 bars at 2/8 centres

4/10 bars bent up singly

3/8 bars at 3/8 centres

3/8 bars to carry stirrups

3/8 bars at 5/8 centres

1/8 bars at 5/8 centres

1/8 bars at 10/8 centres

1/8 between bars

6/10 main bars

4 leg stirrups

8/9 bars at 3/4 centres

5/9 bars at 3/4 centres

22'

14'

10'

6'

4/6 bars to carry stirrups over central 1 feet

1/2 bars at 10/9

8/9 bars at 5/9

3/8 bars at 10/8

3/8 bars at 3/4

4 leg stirrups
DIAGRAM 3

SLUMP TEST

Top diameter 1 1/2"

Height 12"

Truncated metal cone smooth inside. Placed to one side after the removal from first position marked 'A'.

Concrete placed in four layers 3" thick punched 25 times with punning rod 24" long 5/8" diameter bullet end.

Concrete when cone removed
DIAGRAM 4

WIDTH OF FLIGHT = 4'-0" CLEAR

DISTRIBUTION STEEL
1/2" @ 12" OC.

SUPPORTED ON STRONGER BEAMS.

1/2 TREAD (4'-0" CLEAR WIDTH)
1/2 RISER
1/2 COVER
6" WAIST
1/2 COVER
6" COVER
12'-0"

SPANNING BETWEEN BEAMS A AND B.

Fig. 24—Details of Stairs.

COPPER STAMP
ASPHALT (OR SIMILAR JOINT FILLING)

JOINT IN TANK FLOOR (ON SOLID).

BITUMEN PILING
HESSIAN
ASPHALT

WATERPROOFED PAPER
CEILINGLAID

TANLED SEPARATION STRIP
ASPHALT

WATERPROOFED PAPER
BITUMEN

MAIN BEAM

ALTERNATIVE DESIGNS OF JOINTS IN ROOF SLAB.
DIAGRAM: 5

DETAIL OF HOLLOW-TILE SLAB

LINTELS SUPPORTING BRICK WALLS.

FIG. 1

FIG. 2

FIG. 3

FIG. 4
### Table 1

**Areas, Perimeters, and Weights of Rods**

<table>
<thead>
<tr>
<th>Size (inches)</th>
<th>Round Rods</th>
<th></th>
<th></th>
<th></th>
<th>Square Rods</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Area (square inches)</td>
<td>Perimeter (inches)</td>
<td>Weight per foot (pounds)</td>
<td>Area (square inches)</td>
<td>Perimeter (inches)</td>
<td>Weight per foot (pounds)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/4</td>
<td>0.0491</td>
<td>0.785</td>
<td>0.17</td>
<td>0.0625</td>
<td>1.00</td>
<td>0.21</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5/32</td>
<td>0.0767</td>
<td>0.982</td>
<td>0.26</td>
<td>0.0977</td>
<td>1.25</td>
<td>0.33</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3/8</td>
<td>0.1104</td>
<td>1.178</td>
<td>0.38</td>
<td>0.1406</td>
<td>1.50</td>
<td>0.48</td>
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</tr>
<tr>
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<td>0.1503</td>
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<td>0.1563</td>
<td>1.571</td>
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<td>1.26</td>
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<td>3</td>
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<td>30.09</td>
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</tr>
<tr>
<td>Diameter inches</td>
<td>2 in</td>
<td>3 in</td>
<td>4 in</td>
<td>5 in</td>
<td>6 in</td>
<td>7 in</td>
<td>8 in</td>
<td>9 in</td>
</tr>
<tr>
<td>----------------</td>
<td>------</td>
<td>------</td>
<td>------</td>
<td>------</td>
<td>------</td>
<td>------</td>
<td>------</td>
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</tr>
<tr>
<td>1/4</td>
<td>0.29</td>
<td>0.20</td>
<td>0.175</td>
<td>0.15</td>
<td>0.13</td>
<td>0.12</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5/32</td>
<td>0.345</td>
<td>0.31</td>
<td>0.265</td>
<td>0.23</td>
<td>0.20</td>
<td>0.18</td>
<td>0.17</td>
<td>0.15</td>
</tr>
<tr>
<td>3/32</td>
<td>0.36</td>
<td>0.335</td>
<td>0.285</td>
<td>0.25</td>
<td>0.22</td>
<td>0.20</td>
<td>0.18</td>
<td>0.16</td>
</tr>
<tr>
<td>7/64</td>
<td>0.39</td>
<td>0.36</td>
<td>0.315</td>
<td>0.28</td>
<td>0.26</td>
<td>0.24</td>
<td>0.22</td>
<td>0.20</td>
</tr>
<tr>
<td>1/8</td>
<td>0.435</td>
<td>0.405</td>
<td>0.36</td>
<td>0.33</td>
<td>0.31</td>
<td>0.29</td>
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</tr>
<tr>
<td>9/64</td>
<td>0.46</td>
<td>0.43</td>
<td>0.39</td>
<td>0.36</td>
<td>0.34</td>
<td>0.32</td>
<td>0.30</td>
<td>0.28</td>
</tr>
<tr>
<td>5/32</td>
<td>0.505</td>
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<td>0.44</td>
<td>0.41</td>
<td>0.39</td>
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<td>0.35</td>
<td>0.33</td>
</tr>
<tr>
<td>11/64</td>
<td>0.53</td>
<td>0.50</td>
<td>0.47</td>
<td>0.44</td>
<td>0.42</td>
<td>0.40</td>
<td>0.38</td>
<td>0.36</td>
</tr>
<tr>
<td>3/16</td>
<td>0.56</td>
<td>0.54</td>
<td>0.51</td>
<td>0.49</td>
<td>0.47</td>
<td>0.45</td>
<td>0.43</td>
<td>0.41</td>
</tr>
<tr>
<td>13/64</td>
<td>0.59</td>
<td>0.57</td>
<td>0.54</td>
<td>0.52</td>
<td>0.50</td>
<td>0.48</td>
<td>0.46</td>
<td>0.44</td>
</tr>
<tr>
<td>7/32</td>
<td>0.62</td>
<td>0.60</td>
<td>0.58</td>
<td>0.56</td>
<td>0.54</td>
<td>0.52</td>
<td>0.50</td>
<td>0.48</td>
</tr>
<tr>
<td>15/64</td>
<td>0.65</td>
<td>0.63</td>
<td>0.61</td>
<td>0.59</td>
<td>0.57</td>
<td>0.55</td>
<td>0.53</td>
<td>0.51</td>
</tr>
<tr>
<td>1/4</td>
<td>0.675</td>
<td>0.66</td>
<td>0.64</td>
<td>0.62</td>
<td>0.60</td>
<td>0.58</td>
<td>0.56</td>
<td>0.54</td>
</tr>
<tr>
<td>1/8</td>
<td>0.71</td>
<td>0.69</td>
<td>0.67</td>
<td>0.65</td>
<td>0.63</td>
<td>0.61</td>
<td>0.59</td>
<td>0.57</td>
</tr>
<tr>
<td>9/64</td>
<td>0.74</td>
<td>0.72</td>
<td>0.70</td>
<td>0.68</td>
<td>0.66</td>
<td>0.64</td>
<td>0.62</td>
<td>0.60</td>
</tr>
<tr>
<td>5/32</td>
<td>0.77</td>
<td>0.75</td>
<td>0.73</td>
<td>0.71</td>
<td>0.69</td>
<td>0.67</td>
<td>0.65</td>
<td>0.63</td>
</tr>
<tr>
<td>11/64</td>
<td>0.80</td>
<td>0.78</td>
<td>0.76</td>
<td>0.74</td>
<td>0.72</td>
<td>0.70</td>
<td>0.68</td>
<td>0.66</td>
</tr>
<tr>
<td>3/16</td>
<td>0.84</td>
<td>0.82</td>
<td>0.80</td>
<td>0.78</td>
<td>0.76</td>
<td>0.74</td>
<td>0.72</td>
<td>0.70</td>
</tr>
<tr>
<td>13/64</td>
<td>0.87</td>
<td>0.85</td>
<td>0.83</td>
<td>0.81</td>
<td>0.79</td>
<td>0.77</td>
<td>0.75</td>
<td>0.73</td>
</tr>
<tr>
<td>7/32</td>
<td>0.91</td>
<td>0.89</td>
<td>0.87</td>
<td>0.85</td>
<td>0.83</td>
<td>0.81</td>
<td>0.79</td>
<td>0.77</td>
</tr>
<tr>
<td>15/64</td>
<td>0.94</td>
<td>0.92</td>
<td>0.90</td>
<td>0.88</td>
<td>0.86</td>
<td>0.84</td>
<td>0.82</td>
<td>0.80</td>
</tr>
<tr>
<td>1/2</td>
<td>0.98</td>
<td>0.96</td>
<td>0.94</td>
<td>0.92</td>
<td>0.90</td>
<td>0.88</td>
<td>0.86</td>
<td>0.84</td>
</tr>
</tbody>
</table>
TABLE 3
Bonding moment coefficients for slabs spanning in two directions at right angles simply supported on four sides

<table>
<thead>
<tr>
<th>$L$</th>
<th>1.0</th>
<th>1.1</th>
<th>1.2</th>
<th>1.3</th>
<th>1.4</th>
<th>1.5</th>
<th>1.75</th>
<th>2.0</th>
<th>2.5</th>
<th>3.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Z_L$</td>
<td>0.50</td>
<td>0.41</td>
<td>0.32</td>
<td>0.26</td>
<td>0.21</td>
<td>0.16</td>
<td>0.10</td>
<td>0.06</td>
<td>0.03</td>
<td>0.02</td>
</tr>
<tr>
<td>$Z_T$</td>
<td>0.50</td>
<td>0.59</td>
<td>0.68</td>
<td>0.74</td>
<td>0.79</td>
<td>0.84</td>
<td>0.90</td>
<td>0.94</td>
<td>0.97</td>
<td>0.98</td>
</tr>
</tbody>
</table>

Table 10. Bending moment coefficients for rectangular panels supported on four sides with provision for torsion at corners

<table>
<thead>
<tr>
<th>Type of panel and moments considered</th>
<th>Short span $Z'$ Values $Z'$</th>
<th>Long span $Z' = Z_T$ Values $Z_T$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1. Interior panels. Negative moment at continuous edge. Positive moment at mid-span.</td>
<td>0.032 0.046 0.048 0.055 0.058 0.069 0.071 0.077 0.082 0.085</td>
<td>0.055 0.068 0.068 0.072 0.075 0.078 0.081 0.084 0.088 0.091</td>
</tr>
<tr>
<td>Case 2. One edge discontinuous. Positive moment at continuous edge. Positive moment at mid-span.</td>
<td>0.040 0.045 0.047 0.055 0.061 0.065 0.075 0.085 0.091 0.098</td>
<td>0.041 0.043 0.045 0.052 0.054 0.056 0.062 0.068 0.074 0.080</td>
</tr>
<tr>
<td>Case 3. Two adjacent edges discontinuous. Negative moment at continuous edge. Positive moment at mid-span.</td>
<td>0.045 0.046 0.047 0.050 0.053 0.056 0.062 0.068 0.074 0.080</td>
<td>0.049 0.049 0.050 0.053 0.056 0.062 0.068 0.074 0.080 0.086</td>
</tr>
<tr>
<td>Case 4. Two short edge discontinuous. Negative moment. Positive moment.</td>
<td>0.045 0.046 0.047 0.049 0.051 0.053 0.055 0.060 0.066 0.072</td>
<td>0.055 0.056 0.057 0.058 0.059 0.060 0.061 0.066 0.072 0.078</td>
</tr>
<tr>
<td>Case 5. Two long edges discontinuous. Positive moment.</td>
<td>0.055 0.059 0.065 0.068 0.071 0.074 0.077 0.080 0.084 0.088</td>
<td>0.056 0.057 0.059 0.061 0.063 0.065 0.067 0.069 0.072 0.076</td>
</tr>
<tr>
<td>Case 6. Three edges discontinuous. Negative moment at continuous edge. Positive moment at mid-span.</td>
<td>0.065 0.067 0.069 0.071 0.073 0.075 0.077 0.080 0.083 0.086</td>
<td>0.065 0.066 0.067 0.068 0.069 0.072 0.074 0.076 0.079 0.081</td>
</tr>
<tr>
<td>Case 7. Four edges discontinuous. Positive moment at mid-span.</td>
<td>0.065 0.067 0.069 0.071 0.073 0.075 0.081 0.083 0.085 0.087</td>
<td></td>
</tr>
<tr>
<td>Leading Class.</td>
<td>Floor Space Occupancy</td>
<td>Superimposed Load per sq ft</td>
</tr>
<tr>
<td>---------------</td>
<td>-----------------------</td>
<td>-----------------------------</td>
</tr>
<tr>
<td>I</td>
<td>Private Dwellings of not more than two stories</td>
<td>100 lb</td>
</tr>
<tr>
<td>II</td>
<td>Rooms in private dwellings of more than two stories, including flats, hotels, rooms and wards, bedrooms and private sitting rooms in hotels and tenements, houses, and similar occupancies</td>
<td>30 lb</td>
</tr>
<tr>
<td>III</td>
<td>Rooms used as offices</td>
<td>50 lb</td>
</tr>
<tr>
<td>IV</td>
<td>Classrooms in schools and colleges; minimum for light workshops</td>
<td>60 lb</td>
</tr>
<tr>
<td>V</td>
<td>Banking halls and offices where the public may congregate</td>
<td>70 lb</td>
</tr>
<tr>
<td>VI</td>
<td>Retail shops; places of assembly with fixed seating; churches and chapels; restaurants; garages for vehicles not exceeding 2 tons gross weight (private cars, light vans, etc.); circulation space in machinery halls, power stations, pumping stations, etc. when not occupied by plant or equipment</td>
<td>80 lb</td>
</tr>
<tr>
<td>VII</td>
<td>Places of assembly without fixed seating (public rooms in hotels, dance halls, etc.); minimum for filing or record rooms in offices; light workshops generally, including light machinery</td>
<td>100 lb</td>
</tr>
<tr>
<td>VIII</td>
<td>Garages to take all types of vehicles</td>
<td>100 lb</td>
</tr>
<tr>
<td>IX</td>
<td>Light storage space in commercial and industrial buildings; medium workshops</td>
<td>150 lb</td>
</tr>
<tr>
<td>X</td>
<td>Minimum for warehouses and general storage space in commercial and industrial buildings; heavy workshops. (Thrusts imposed by heavy plant &amp; machinery should be determined &amp; allowed for)</td>
<td>230 lb</td>
</tr>
</tbody>
</table>

* Minimum load for slabs becomes operative at spans of less than 8 ft.
Minimum load for beams becomes operative on areas less than 64 sq. ft.
Beams, ribs & joists spaced at not more than 3 ft. centres may be calculated for slab loading.

† Fixed seating implies that the removal of the seating & the use of the space for other purposes is improbable.


**TABLE 5**

Reductions of total superimposed floor loads on columns, etc. See Fig 42

<table>
<thead>
<tr>
<th>Number of floors carried by member under consideration</th>
<th>Per cent reduction of superimposed load on all floors above the member under consideration</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>10</td>
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<tr>
<td>3</td>
<td>20</td>
</tr>
<tr>
<td>4</td>
<td>30</td>
</tr>
<tr>
<td>5</td>
<td>40</td>
</tr>
<tr>
<td>6 or more</td>
<td>50</td>
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</tbody>
</table>
### TABLE 6

Bending moment coefficients for continuous beams and slabs with various numbers of spans and conditions of loading with ends freely supported

<table>
<thead>
<tr>
<th>Loading</th>
<th>Dead Loads (All Spans Loaded)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>BM = Coefficient x W/L</td>
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</tbody>
</table>

#### Load at center point

<table>
<thead>
<tr>
<th>0.107</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.098</td>
</tr>
<tr>
<td>0.085</td>
</tr>
<tr>
<td>0.063</td>
</tr>
<tr>
<td>0.041</td>
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<tr>
<td>0.019</td>
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<tr>
<td>0.007</td>
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<tr>
<td>0.003</td>
</tr>
<tr>
<td>0.001</td>
</tr>
<tr>
<td>0.000</td>
</tr>
</tbody>
</table>

#### Load at midspan

<table>
<thead>
<tr>
<th>0.140</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.128</td>
</tr>
<tr>
<td>0.116</td>
</tr>
<tr>
<td>0.105</td>
</tr>
<tr>
<td>0.094</td>
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<tr>
<td>0.084</td>
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<tr>
<td>0.074</td>
</tr>
<tr>
<td>0.064</td>
</tr>
<tr>
<td>0.054</td>
</tr>
<tr>
<td>0.045</td>
</tr>
</tbody>
</table>

#### Load at end points

<table>
<thead>
<tr>
<th>0.140</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.128</td>
</tr>
<tr>
<td>0.116</td>
</tr>
<tr>
<td>0.105</td>
</tr>
<tr>
<td>0.094</td>
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<tr>
<td>0.084</td>
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<tr>
<td>0.074</td>
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<tr>
<td>0.064</td>
</tr>
<tr>
<td>0.054</td>
</tr>
<tr>
<td>0.045</td>
</tr>
</tbody>
</table>

#### Superimposed Loads (in worst possible cases)

<table>
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<tr>
<th>0.107</th>
</tr>
</thead>
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</tr>
</tbody>
</table>
# TABLE 7

REDUCTION COEFFICIENT FOR STRESSES IN LONG COLUMNS

<table>
<thead>
<tr>
<th>Least lateral dimension of column</th>
<th>Least radius of gyration of column, $r_a$</th>
<th>Reduction coefficient.</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>50</td>
<td>1.0</td>
</tr>
<tr>
<td>16</td>
<td>60</td>
<td>0.9</td>
</tr>
<tr>
<td>21</td>
<td>70</td>
<td>0.8</td>
</tr>
<tr>
<td>24</td>
<td>80</td>
<td>0.7</td>
</tr>
<tr>
<td>27</td>
<td>90</td>
<td>0.6</td>
</tr>
<tr>
<td>30</td>
<td>100</td>
<td>0.5</td>
</tr>
<tr>
<td>33</td>
<td>110</td>
<td>0.4</td>
</tr>
<tr>
<td>36</td>
<td>120</td>
<td>0.3</td>
</tr>
<tr>
<td>39</td>
<td>130</td>
<td>0.2</td>
</tr>
<tr>
<td>42</td>
<td>140</td>
<td>0.1</td>
</tr>
<tr>
<td>45</td>
<td>150</td>
<td>0.0</td>
</tr>
</tbody>
</table>

* Only to be used for rectangular columns with symmetrical cross sections and without re-entrants.
TABLE 8
CURTAILMENT OF TENSILE BARS
Sections of which bottom bars can be stopped (or bent up) - uniformly distributed load

Max distance from centre of support to P - K2, where P - point of stopping or bending up
if bar is not bent up sufficient bend distance must be provided from point of max stress
in bar to point of curtailment (see sec. 16.6)

![Diagram of freely supported span, end span, and interior span](image)

<table>
<thead>
<tr>
<th>VARIOUS SPANS</th>
<th>K1</th>
<th>K2</th>
<th>K3</th>
<th>K4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1st</td>
<td>2nd</td>
<td>3rd</td>
<td>4th</td>
</tr>
<tr>
<td>1</td>
<td>0</td>
<td>-11</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>15</td>
<td>-24</td>
<td>-11</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>21</td>
<td>-27</td>
<td>-11</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>25</td>
<td>-30</td>
<td>-11</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>27</td>
<td>-32</td>
<td>-11</td>
<td>0</td>
</tr>
<tr>
<td>6</td>
<td>30</td>
<td>-34</td>
<td>-11</td>
<td>0</td>
</tr>
<tr>
<td>7</td>
<td>31</td>
<td>-36</td>
<td>-11</td>
<td>0</td>
</tr>
<tr>
<td>8</td>
<td>32</td>
<td>-38</td>
<td>-11</td>
<td>0</td>
</tr>
</tbody>
</table>
## TABLE 9

### SHEAR REINFORCEMENT

**Shear Value of Single Binders (Two Arms)**

\[
V = \frac{S}{a} = \frac{A_t \cdot t}{p}
\]

<table>
<thead>
<tr>
<th>Diameter of Binder (in)</th>
<th>Spacing of Binders (inches)</th>
<th>Shear Resistance ( p = a ) (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{3}{16} )</td>
<td>3</td>
<td>142</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>124</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>110</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>96</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>82</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>72</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>63</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>56</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>41</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>9.95</td>
</tr>
</tbody>
</table>

### Maximum Spacing of Binders

- Normally \( p = 3 \) binders around compression bars: \( 3 \cdot 12 = 36 \) in bar.
- Ditto, ditto (Steel Beam Theory) = Biaxial Bar.

### Shear Resistance of Inclined Bars (lb)

<table>
<thead>
<tr>
<th>Diameter of Bar (in)</th>
<th>( \theta = 45^\circ )</th>
<th>( \theta = 30^\circ )</th>
<th>NOTES</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{1}{4} )</td>
<td>2,500</td>
<td>1,750</td>
<td>SINGLE SYSTEM</td>
</tr>
<tr>
<td></td>
<td>3,900</td>
<td>2,750</td>
<td>DOUBLE SYSTEM</td>
</tr>
<tr>
<td>( \frac{2}{16} )</td>
<td>5,600</td>
<td>3,900</td>
<td>SINGLE SYSTEM</td>
</tr>
<tr>
<td></td>
<td>7,600</td>
<td>5,400</td>
<td>DOUBLE SYSTEM</td>
</tr>
<tr>
<td>( \frac{1}{4} )</td>
<td>7,600</td>
<td>5,400</td>
<td>SINGLE SYSTEM</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>DOUBLE SYSTEM</td>
</tr>
<tr>
<td>( \frac{1}{2} )</td>
<td>10,000</td>
<td>7,050</td>
<td>SINGLE SYSTEM</td>
</tr>
<tr>
<td></td>
<td>12,300</td>
<td>8,950</td>
<td>DOUBLE SYSTEM</td>
</tr>
<tr>
<td>( \frac{1}{4} )</td>
<td>15,600</td>
<td>11,050</td>
<td>SINGLE SYSTEM</td>
</tr>
<tr>
<td></td>
<td>18,900</td>
<td>13,350</td>
<td>DOUBLE SYSTEM</td>
</tr>
<tr>
<td>( \frac{1}{2} )</td>
<td>22,500</td>
<td>15,900</td>
<td>SINGLE SYSTEM</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>DOUBLE SYSTEM</td>
</tr>
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</table>

- Make value of D
  - 1.41d
  - 0.71d
  - 2.00d
  - 1.00d
## TABLE 10
PERMISSIBLE STRESSES

<table>
<thead>
<tr>
<th>Mix</th>
<th>Volumetric proportion</th>
<th>Minimum crushing strength after 28 days, lb/in. sq.</th>
<th>Bearing pressure, T/in.²</th>
<th>Compressive stress due to bending, lb/in.²</th>
<th>Direct compressive stress, lb/in.²</th>
<th>Shear, lb/in.²</th>
<th>Bond, lb/in.²</th>
</tr>
</thead>
<tbody>
<tr>
<td>I.</td>
<td>1:2:4</td>
<td>2250</td>
<td>30</td>
<td>750</td>
<td>600</td>
<td>75</td>
<td>100</td>
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<tr>
<td>II.</td>
<td>1:1/2:5</td>
<td>2550</td>
<td>40</td>
<td>850</td>
<td>680</td>
<td>85</td>
<td>110</td>
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<tr>
<td>III</td>
<td>1:1:2</td>
<td>2925</td>
<td>50</td>
<td>950</td>
<td>780</td>
<td>95</td>
<td>120</td>
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DESIGN FACTORS (C = 18000 lbs/in.² )

<table>
<thead>
<tr>
<th>Mix</th>
<th>-neutral axis factors</th>
<th>Lever arm factor</th>
<th>Resisting moment factor</th>
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<tbody>
<tr>
<td>I.</td>
<td>.385</td>
<td>.87</td>
<td>126 Lbs/in.²</td>
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<tr>
<td>II.</td>
<td>.415</td>
<td>.86</td>
<td>152 Lbs/in.²</td>
</tr>
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<td>III</td>
<td>.45</td>
<td>.85</td>
<td>186 Lbs/in.²</td>
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### Table 12

<table>
<thead>
<tr>
<th>WATER/CEMENT RATIO BY WEIGHT</th>
<th>CUBE CRUSHING STRENGTH (lb/sq. inch)</th>
<th>MEDIUM WORKABILITY GRAPH GRADING</th>
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<tr>
<td></td>
<td>7 DAYS</td>
<td>2.8 DAYS</td>
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<tr>
<td>0.35</td>
<td>5700</td>
<td>7500</td>
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<td>5600</td>
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<td>4300</td>
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<td>3600</td>
<td>5300</td>
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<td>4600</td>
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<td>3500</td>
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<td>0.70</td>
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<td>3100</td>
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### Curves of Four Grading of 3/4 Aggregate