the enlarged column where $\frac{r}{r_o} = 1$,

$$\text{Max. } M_2 = wr_o^2\,(0.2 + C_1 + C_2) \qquad (52)$$

$$\text{Max. } M_b = q\,r_o\,(C_a + C_b) \qquad (53)$$

The circumferential moments at this point are a minimum and smaller than M_2, and M_b hence these latter only need be computed for maximum stress and the circumferential moments, M_1 and M_a may be disregarded.

If q is in pounds per foot of length, w in pounds per square foot and r_0 in feet, the moments are in foot pounds per foot, or inch pounds per inch.

The formulas are readily solved by using the table of constants given on page 518. The table is made out for four values of g. It may be assumed that the fiber stresses caused by any moment do not weaken the concrete in the direction perpendicular to those stresses.

The thickness and reinforcement of the slab are found in the usual way by equating the actual bending moment, as determined above, to the moment of resistance of the steel and concrete. The limiting thickness of slab is usually determined by the thickness near the column required to resist the negative bending moment there. It is advisable, then, to make the thickness of the slab near the support as thin as possible by using a rich concrete and a larger amount of steel and by placing some steel in the bottom of the slab for compression. In this way the thickness near the support may be reduced nearly to the economical thickness in the center of the span.

The slab between the circular plates may be considered as supported on all edges. From Fig. 152 it is evident that the largest deflection and the largest positive bending moment occur in the middle of the panel, and may be safely taken as those of a square plate supported on all edges, the side of which is the diagonal distance between the circles of inflection. This distance between circles of inflection may thus be taken as the span, and the thickness and reinforcement at the middle computed very conservatively by the formula $M = \frac{w\,l^2}{16}$.

A value for compression in concrete, f_c, higher than in beam construction is permissible, and a lower value of n, because of the rich concrete mixture and because of the fact that the maximum stresses occur near the support, where the concrete bears on a larger area, and for this reason is able to stand, say, 15 per cent higher stresses than in the middle of the beam. It is advisable, however, to fix a maximum stress of 800 pounds per square inch even with a very rich concrete of proportions say $1:1\frac{1}{2}:3$.

EXAMPLE OF FLAT SLAB DESIGN

Example 14: Design a flat slab to support a live load of 250 pounds per square foot, the spacing of columns being 17 by 17 feet and the diameter of their head 48 inches. The working stresses in steel, $f_s = 16\,000$ pounds per square inch, in concrete, $f_c = 700$ pounds per square inch and Poisson's ratio, $g = 0.1$, allowing for a rather rich concrete, will be accepted.

Solution: The slab will be considered as a flat circular plate fixed to the column and supporting at its circumference the rest of the floor, as outlined on page 483. The inner radius of the plate is the radius of the column head and its outer radius will be accepted as the average distance of the points of inflection of the slab from center of column. The distance of points of inflection from column center is thus taken as one-fifth of the clear span plus the radius of the column. The minimum distance is $\frac{17 - 4.0}{5} + 2.00 = 4.6$ ft., the maximum is $\frac{24 - 4}{5} + 2.00 = 6.0$ ft., and the average distance $\frac{4.6 + 6.00}{2} = 5.3$ feet, or $r_o = 2$ feet, $r_1 = 5.3$ feet.

Live load	=	250 pounds per square foot.
Assumed dead load	=	120 " " " "
Total unit load, w,	=	370 " " " "

Area of slab is $17 \times 17 = 289$ square feet and area of circular plate $5.3^2 \times 3.14 = 88$ square feet; hence the difference of the two areas is 201 square feet, which is the area of slab outside of the assumed circular plate. The loading of this area is supported along the circumference of the flat plate, and equals $201 \times 370 = 74\,370$ pounds. Dividing this value by the circumference of the outer circle of the plate, the circumferential unit loading, $q = 2200$ pounds per foot, will be obtained. The ratio of radii

$$\frac{r_1}{r_o} = \frac{5.3}{2} = 2.65.$$

Finding from the table on page 000 the corresponding constants, the maximum moments which occur at the support, that is at the circumference of the column head, are

$M_2 = 370 \times 2^2\,(0.2 + 1.75 + 0.30) = 3330$ inch pounds.
$M_b = 2200 \times 2\,(2.01 + 1.13) = 13\,800$ inch pounds.

Total moment, $M = 17\,130$ inch pounds per inch of circumference of column head.

This is a negative moment, the top of slab being in tension and the bottom in compression, as in any fixed or continuous member, at the support.

The thickness of the slab may be found as explained on page 421. If steel is used only in the top of the slab, the depth and reinforcement may be assumed from the ordinary slab formula, page 421, using the total M given above. If compression as well as tension steel is used, requiring steel in both top and bottom, formulas (18) and (20), page 428, will determine the required depth and reinforcement. In the present case $1\frac{1}{2}\%$ of steel will be placed at the top, and the same amount at the bottom; hence, using formula (18), page 428, and table on page 516, with ratio, $a = 0.1$

$d = \sqrt{\frac{17\,130}{800 \times 0.32}} = 8.18$ inches, requiring a slab thickness of about $9\frac{1}{2}$ in.

The stress in concrete over the support was allowed at $700 + 15\%$, or 800 pounds per square inch. The same slab with $1\frac{1}{2}\%$ of steel at the top only would require a depth

$= \sqrt{\frac{2\,M}{f_c\,b\,j\,k}} = 10.6$ inches and a total thickness of about 12 inches.

Several trials should be made to determine the most economical relation of the amount of steel and concrete. It should be borne in mind that the increase of reinforcement for a short length over the support decreases the thickness of the entire slab, reducing the amount of material and at the same time the dead load and the moment. Hence, a larger percentage than used in beam and slab design and the introducing of steel at the top may prove economical.

The diagonal distance between the circles of inflection is $24 - 10.6 = 13.4$ feet, and the bending moment in the middle of the slab (see p. 486)

$$M = \frac{370 \times 13.4^2 \times 12}{16} = 49\,900 \text{ inch pounds per foot width.}$$

The effective thickness of the slab as determined by the necessary thickness over the support is $10 - 1 = 9$ inches.

Then

$$C = \sqrt{\frac{bd^2}{M}} = 0.139$$

In Table 11 (*p.* 520.) $p = 0.0036$ corresponds to $C = 0.139$, hence 0.36% of steel in each diagonal direction will be necessary.

CONCRETE COLUMNS

Columns of short length, essentially piers, the length of which is not more than six times the least lateral dimension, may be built of plain concrete with no reinforcement, provided the loading is central. Columns longer than this should be reinforced for safety in construction and also to guard against the possibility of eccentric loading and the danger of sudden failure. It is desirable to further limit the use of reinforced columns to a length of 15 diameters.

Although concrete is especially adapted for sustaining compression, its compressive strength is so much lower than that of steel that in a building it is frequently difficult to keep the columns in the lower stories within the limits required by the uses for which the building is constructed.

To reduce the size of the column, four distinct methods are used either separately or in combination:

(1) Rich proportions of concrete.

(2) Vertical steel bars designed to assist in taking the compression.

(3) Hooping or banding.

(4) Structural steel shapes in combination with the concrete.

These will be considered in the order given.

While as a general proposition concrete in compression is always cheaper than steel, the limits of size of column frequently make steel reinforcement necessary not only to resist bending caused by eccentric loading or lateral pressure, but to take a part of the vertical compression load.

Whatever the type of construction, the effective area to use in figuring the compression should usually be less than the total area to allow a certain thickness on the surface for fire protection. The Joint Committee recommend that the protective covering shall be taken to a depth of 1½ inch on all surfaces, since in a severe fire the concrete to this depth may be affected by the heat and its strength destroyed. A less thickness than this should be sufficient where the contents of a building are not especially inflammable, a decrease in the total diameter or width of a column of 1 to 2 inches being frequently a fair allowance when computing the effective area.

The steel, however, should in all cases be imbedded at least 1½ to 2 inches, and when circular hooping is used to add strength and ductility the effective area must be taken as that within the hooping.

Rich Proportions of Concrete. The compressive strength of concrete is approximately proportional to the amount of cement which it contains (see Chap. XX), so that increasing the proportion of cement is an effective means of strengthening the column to permit smaller section. A rich concrete also has a higher modulus of elasticity and there is consequently less deformation under load. Besides this, a rich concrete works smoother in placing and makes it easier to produce a homogeneous column, provided the aggregates are properly graded. The strength of concrete for different mixtures is indicated on page (360), and working stresses are suggested on page (527). Before permitting the use of high column stresses in a structure, actual compressive tests should be made upon cylinders 8 inches diameter by 16 inches high composed of the same materials to be used and mixed in the required proportions with the same wet consistency.

Vertical Steel Bar Reinforcement. Tests of long columns made at the Watertown Arsenal,* the Massachusetts Institute of Technology,† and the University of Illinois,‡ indicate conclusively that vertical steel bars imbedded in concrete may be counted upon to take their portion of the loading. As a column takes its load, it is shortened in height, the concrete and steel, shortening equally because they are bonded together. The concrete, however, has so much lower strength that it receives its allowable load before the steel can reach its full working strength. Consequently, the working load upon the steel must be figured at a low value, which is determined by the amount of shortening it has undergone up to the point where the concrete is shortened so as to reach its working strength. Since, with a given load, the shortening or deformation is proportional to its

* Tests of Metals, U. S. A., 1904, 1905, 1906, 1907.
† Transactions American Society of Civil Engineers, Vol. L, p. 487.
‡ University of Illinois Bulletin 20, December 25, 1907.

modulus of elasticity (see p. 529), the working stress in the steel must be the working stress in the concrete times the ratio of the moduli of elasticity of steel to concrete, as indicated below.

Although tests indicate that if vertical steel is placed at least 2 inches from the surface of the column, the elastic limit of the steel may be reached without danger or buckling, it is nevertheless advisable in almost all cases to place occasional horizontal loops around the steel spaced at distances apart not greater than the width of the column as an additional precaution against the buckling of the rods, and also for the purpose of keeping the bars in place during the pouring of the concrete. The size and location of such loops are discussed in connection with column design on page 466.

Joints in the vertical steel when small diameter rods are used, say up to 1¼ inch, may be provided for by lapping as indicated on page 464. Large diameter rods should have their ends planed true and butted with a sleeve around the joint, or should have some other positive connection. In footings where the length of imbedment is not sufficient to take all the stress, a horizontal bearing plate must be provided.

Since the relative loading upon the steel and the concrete at any period is theoretically in direct proportion to the ratio of their moduli of elasticity at that period, and since full size column tests have borne out this assumption, the allowable loading, that is, the allowable unit pressure, is readily obtained as follows:*

* From mechanics

$$\frac{\text{stress per square inch}}{\text{modulus of elasticity}} = \text{deformation}$$

hence $\frac{f'_s}{E_s}$ = deformation of steel and $\frac{f_c}{E_c}$ = deformation of concrete.

Since with perfect adhesion between concrete and steel all parts of the column must undergo the same deformation,

$$\frac{f'_s}{E_s} = \frac{f_c}{E_c} \text{ or } f'_s = f_c n$$

The allowable stress in steel is therefore the allowable stress in the concrete times the ratio of elasticity. For practical purposes the total loading must be introduced. Since the total pressure in the column must be the sum of the pressure in the concrete plus the pressure in the steel,

$$fA = f_c A_c + f'_s A_s \quad \text{or} \quad fA = f_c A_c + f_c n A_s$$

and since $A_c = A - A_s$ we have

$$f = f_c \left[\left(\frac{A - A_s}{A} \right) + n \frac{A_s}{A} \right]$$

or since $p = \frac{A_s}{A}$ we reach the result

$$f = f_c [(1 - p) + np]$$

Let

f = allowable unit pressure upon the reinforced column, equal to the total load divided by the effective area.

f_c = allowable unit pressure upon the concrete of the column.

f'_s = allowable unit pressure upon the vertical steel in the column.

$n = \frac{E_s}{E_c}$ = ratio of modulus of elasticity of steel to modulus of elasticity of concrete.

P = load to be sustained by the column.

A = area of total effective* cross-section of column.

A_c = area of concrete in cross-section.

A_s = area of steel in cross-section.

$p = \frac{A_s}{A}$ = ratio of cross-section of steel to total cross-section of column.

For determining the total allowable unit compression, f (which is the total load, P, divided by the effective area A) with fixed area of concrete and steel, we have

$$f = \frac{f_c A_c + f_c n A_s}{A} \tag{59}$$

In terms of the percentage of steel,

$$f = f_c[1 + (n - 1) p] \tag{60}$$

The percentage of steel to use to obtain total unit stresses when the compression on the concrete is limited to f_c is

$$p = \frac{f - f_c}{f_c (n - 1)} \tag{61}$$

and the effective cross-section of column is

$$A = \frac{P}{f_c[1 + (n - 1) p]} \quad (62) \qquad \text{or } A = \frac{P}{f} \quad (63)$$

To this area must be added the protective covering as indicated above.

* See page 497.

The table below gives values of f for different stresses and different moduli of elasticity.

Working Loads on Concrete Columns Reinforced With Longitudinal Rods. (*See p.* 492)

RATIO OF STEEL	ALLOWABLE UNIT LOAD ON COLUMNS IN LB. PER SQ. IN.											
p	Ratio of Moduli, n = 10				Ratio of Moduli, n = 15				Ratio of Moduli, n = 20			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
	fc = 450	fc = 550	fc = 650	fc = 750	fc = 450	fc = 550	fc = 650	fc = 750	fc = 450	fc = 550	fc = 650	fc = 750
0.01	490	599	708	817	513	627	741	855	535	654	773	892
0.02	531	649	767	885	576	704	832	960	621	759	897	1035
0.03	571	698	825	952	639	781	923	1065	706	863	1020	1177
0.04	612	748	884	1020	702	858	1014	1170	792	968	1144	1320

NOTE—Use column (6) ordinarily for first class 1 : 2 : 4 concrete.

Examples on page 498 illustrate the use of these formulas.

The table on p. 493 from tests by Mr. James E. Howard gives the relation of actual tests to theoretical computations based on a ratio of elasticity of 15. It is noticeable that the actual strength is almost always more than the theoretical, and this is especially the case with the leaner mixtures because the modulus of elasticity of the leaner concrete is lower, and therefore the ratio of 15 is very conservative.

An excellent analytical treatment of columns reinforced with vertical steel is given by Professor Talbot in one of his University Bulletins.* The problem is discussed briefly by one of the authors in a paper before the Boston Society of Civil Engineers.†

The analysis of the action of combined compression and bending, such as is produced in columns loaded eccentrically, and the method of computing the reinforcement in such cases is treated in pages 560 to 574.

Hooped or Banded Columns. Mr. A. Considère in France was the first to apply to reinforced concrete the principle that if a material is confined laterally, it will deform or shorten less under vertical loading, and therefore can sustain a heavier load before it crushes. This is the principle involved in the hooped or banded column. It is carried out in practice by placing steel bands or spiral hooping within the column designed to resist the lateral deformation.

* University of Illinois, Bulletin No. 12, Feb. 1, 1907.

† Sanford E. Thompson in Journal Association Engineering Societies, June 1907, p. 316.

Tests at the Watertown Arsenal,* the University of Illinois† and the University of Wisconsin,‡ 1906-1907, prove that while hooping or banding increases the crushing strength of the column, the deformation, that is, the shortening of the column, is so great at a comparatively early period in the loading that the safe strength cannot be based directly upon the breaking strength.

A perfect fluid like water will transmit pressure equally in all directions. Concrete, on the other hand, under ordinary loading expands laterally a very small percentage of its vertical deformation or shortening (see p. 484); so that, even from a theoretical standpoint, the hoops should not come into play until the concrete has shortened so much that its elastic limit, or the period corresponding to this, has been passed.§

Strength of Plain vs. Vertically Reinforced Concrete and Mortar Columns. Columns 12″ × 12″. *Height* 8 *feet. Age of Mortar and Concrete* 6 *months Watertown Arsenal* (*see p.* 492).

PROPORTIONS			Plain Concrete or Mortar Columns Actual Strength lb. per sq. in.	REINFORCED COLUMNS				REFERENCE TO "TESTS OF METALS" U. S. A.
				Reinforcement.			Computed Strength based on col. (4) and a ratio of $n = 15$ lb. p. sq. in.	
Cement.	Sand.	Stone.		Description.	Ratio Area Steel to Area Column.	Actual Strength lb. per sq. in.		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	2	0	3070	8-¾″ round bars	0.029	4200	4290	1905 p. 377
1	3	0	2380	8-¾″ round bars	0.029	3840	3320	1905 p. 377
1	4	0	1520	8-¾″ round bars	0.029	3380	2120	1905 p. 377
1	5	0	1080	8-¾″ round bars	0.029	2810	1510	1905 p. 377
1	5	0	1080	13-¾″ round bars	0.046	3900	1780	1905 p. 377
1	1	2*	1720	4-¾″ twisted bars	0.014	2890	2060	1904 p. 386
1	2	3*	1769	4-¾″ twisted bars	0.014	2010	2100	1904 p. 386
1	2	4	1413	4-0″ 0.74″ × 0.74″ trussed bars	0.014	1900	1689	1906 p. 538
1	2	4*	1710	4-¾″ twisted bars	0.014	1990	2050	1904 p. 386
1	2	4†	2400	8-¾″ twisted bars	0.029	3700	3360	1907 p. 242
1	3	6	1450	8-⅝″ corr. bars	0.019	2290	1840	1904 p. 379 1906 p. 535

* ½″ to 1½″ pebbles.

†Age 17 months 22 days.

The action of the hooped column as established by tests on long columns is discussed by one of the authors as follows:‖

* Tests of Metals, U. S. A., 1906.

†University of Illinois. Bulletin No. 20, Dec. 15, 1907.

‡Transactions American Society for Testing Material, Vol. IX 1909.

§ See discussion by Sanford E. Thompson in Journal Association Engineering Societies, July, 1907, p. 320. The effect of lateral expansion based on the action of plain columns is here treated before the publication of the tests of hooped column which established the principle.

‖ Sanford E. Thompson in Transactions American Society of Civil Engineers, Vol. LXI, 1908, p. 47.

When a load is placed upon the top of any column, it causes vertical compression or deformation, with, at the same time, a lateral expansion. The lateral expansion in concrete columns, as shown by tests upon plain and upon reinforced columns by Mr. J. E. Howard at the Watertown Arsenal* and by A. N. Talbot, M. Am. Soc. C. E., at the University of Illinois,† is at first very small. Any stress produced in the steel hooping must be proportional to its deformation or stretching; hence, with small lateral expansion of the concrete, there can be but slight stress in the hoops. For this reason, and also because of the initial shrinkage of the concrete, which the lateral expansion must first overcome, scarcely any stress or pull comes upon the hoops until the concrete within them has reached a loading equal to the breaking load in plain concrete. As this load is approached, the modulus of elasticity of the concrete becomes very much lower, and consequently both the vertical and lateral deformations become much greater. Then, and not until then, does the lateral pressure begin to act appreciably upon the hoops. In other words, up to the very crushing strength of plain concrete, the value of the hooping is actually negligible. From then on, the reinforcement takes practically all the load, and a high ultimate strength may be attained, although coincident with great shortening of the column.

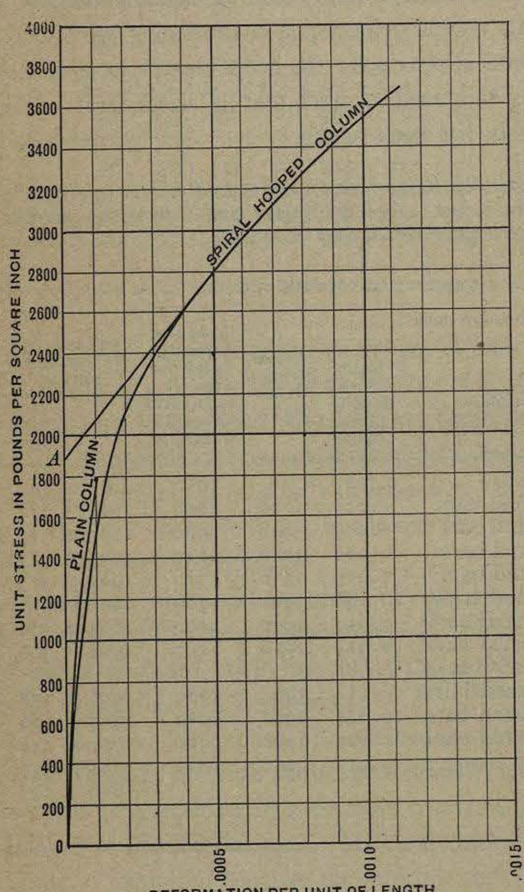

FIG. 153. Deformation of a Hooped and of a Plain Column.‡ (See p. 494.)

Even with the concrete restrained within the hoops, the shell of concrete outside of them, which is necessary for fireproofing and for the protection

* Tests of Metals, U. S. A., 1905, pp. 293–336.
† Proceedings American Society for Testing Materials, Vol. VII, 1907, p. 382.
‡ Columns 109 and 182 from Bulletin No. 20, University of Illinois, December 15, 1907.

of the steel, begins to crack and peel at about the same load as that which will cause complete failure in unreinforced concrete. Professor Talbot, in fact, states as a general proposition that: "Cracking and peeling of the concrete appear at loads corresponding to the ultimate strength of the concrete."

Tests also indicate that the shortening of the column is so great that the elastic limit of any vertical steel rods is passed at a load but slightly greater than that corresponding to the crushing strength of plain concrete.

The typical deformation of a column reinforced with spiral hooping as compared with a column having no reinforcement is shown by the curves Fig. 153. Although the ultimate strength of the hooped column shown is 3700 pounds per square inch, it will be seen that at a load of 1800 pounds per square inch, the crushing strength of the plain column, the curve drops off very rapidly and the line produced back to the axis of ordinates at A agrees very closely with the crushing strength of the plain column. At 2000 pounds per square inch the deformation per unit of length is 0.0017. At this deformation vertical steel in such a column would be stressed to 51 000 pounds per square inch. In other words, at a load only 10% higher than that to be expected of a plain column, even steel of a high elastic limit would have reached its yield point.

The entire subject is treated very fully by Professor Talbot in the description of his tests in the Bulletin from which the diagram is taken.

Quoting again from Mr. Thompson's Discussion before the American Society of Civil Engineers:

Tentative conclusions with regard to hooped column design at the present stage of tests may be summarized as follows:

(1) Hooping, if properly applied, increases the ultimate breaking strength under a single loading to double or treble the breaking strength of a plain column.

(2) The surface of concrete outside of the hooping will begin to crack at a loading corresponding to the breaking load of an unhooped column.

(3) Hooping, if not continuous or rigid, will peel off with surface concrete so that the effective strength of the column will be no greater than a similar one of plain concrete.

(4) The total vertical deformation of a hooped column is so great at the period of first external crack that any vertical steel, unless designed to carry the entire load, is stressed beyond its safe strength.

(5) The ultimate breaking strength of a hooped column is no measure of its safe strength, and formulas based on the ultimate strength are useless.

Notwithstanding these conclusions it must not be inferred that hooping is of no value. It does increase the crushing strength, and thus adds ductility to the column and permits of a somewhat higher unit stress upon the concrete. The hoops also appear practically to affect the shearing stress so that the column acts more like a cube than like a long prism, with consequently higher strength. The Joint Committee conclude:

> The general effect of bands or hoops is to increase greatly the "toughness" of the column and its ultimate strength, but hooping has little effect upon its behavior within the limit of elasticity. It thus renders the concrete a safer and more reliable material, and should permit the use of a somewhat higher working stress. The beneficial effects of "toughening" are adequately provided by a moderate amount of hooping, a larger amount serving mainly to increase the ultimate strength and the possible deformation before ultimate failure.

The loadings suggested for use by the Joint Committee are referred to on page 527.

A type of formula suggested by Considère for determining the ultimate strength of hooped columns is as follows:

Let

f = ultimate unit pressure upon the reinforced column, equal to the total load divided by the effective area in pounds per square inch.

f_c = ultimate unit pressure upon the concrete of the column in pounds per square inch.

p = ratio of sectional area of longitudinal reinforcement to the area of concrete core.

p'' = ratio of volume of steel hooping in a given height of column to the volume of the concrete core in this height.

Then

$$f = 1.5 f_c + 2\,400\,p + 510\,p'' \tag{64}$$

Professor Talbot suggests the following formulas for ultimate crushing strength:

$$f = f_c + 65\,000\,p'' \tag{65}$$

for columns reinforced with bands, and for those reinforced with spirals

$$f = f_c + 100\,000\,p. \tag{66}$$

The above formulas cannot be safely used, however, for computing the working strength of hooped columns.

The Joint Committee suggest with reference to hooping:

> The effective area of the column shall be taken as the area within the protective covering (see page 489); or, in the case of hooped columns or columns reinforced with structural shapes, it shall be taken as the area within the hooping or structural shapes.

The Joint Committee also specify that the hoops or bands should not be counted upon directly as adding to the strength of the column. They suggest:

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> Where bands or hoops are used, the total amount of such reinforcement shall be not less than 1% of the volume of the column disclosed. The clear spacing of such bands or hoops shall not be greater than one-fourth the diameter of the enclosed column. Adequate means must be provided to hold bands or hoops in place so as to form a column, the core of which shall be straight and well centered.

Hooping then may be considered not as adding to the working strength in proportion to the amount of steel in the hoops, but rather as increasing the ductility of the column and reducing the danger of sudden failure, so that a lower factor of safety is permissible. In practice, to gain the benefit of this, a higher working stress may be permitted in hooped columns when reinforced with steel bands or hoops the total volume of which in a given length of column is at least 1 per cent of the volume of concrete within the hooping.

Adopting the Joint Committee recommendations:

Columns with reinforcement of not less than 1 per cent in bands or hoops may be given a working stress 20 per cent higher than for plain concrete columns. If working stress in plain concrete is taken as 450 pounds per square inch, the hooped concrete may be thus given 540 pounds per square inch.

Columns reinforced with not less than 1 per cent and not more than 4 per cent of longitudinal bars and with not less than 1 per cent in bands or hoops may be given a working stress 45 per cent higher than plain concrete columns. If the working stress in plain columns is taken as 450 pounds per square inch, the hooped and vertically reinforced column may be thus given 650 pounds per square inch plus the working value of the longitudinal rods as indicated on page 492.

STRUCTURAL STEEL REINFORCEMENT

If the structural steel is designed to take all the load and then is simply fireproofed with a concrete covering, it is not reinforced concrete. When the structural steel is designed so that it takes a load in combination with