THE PROPERTIES AND DESIGN

OF

REINFORCED CONCRETE

INSTRUCTIONS, AUTHORISED METHODS OF CALCULATION, EXPERIMENTAL RESULTS AND REPORTS BY THE FRENCH GOVERNMENT COMMISSIONS ON REINFORCED CONCRETE

TRANSLATED AND ABRIDGED

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AUTHOR'S PREFACE

In setting out to study the properties and the applications of such a comparatively new material as Reinforced Concrete, one is led to consider the possible sources of information and their nature. The fact should be kept in mind that, from certain points of view, the study of much successful practice is frequently not so fruitful as the study of some examples where failure took place or of full-sized structures tested to destruction.

A considerable amount of experimental work has been done and a vast number of reinforced concrete structures successfully carried out in America, of which the records are in English. In Germany, Italy, and Austria a considerable amount of both experimental and practical work has been done. In Great Britain the volume of reinforced concrete construction is steadily growing, but very little experimental research on the properties of the material has been made in this country. It is undoubtedly to France and to French literature that a student must turn for the most concise and authoritative information on the subject.

Reinforced Concrete had its origin in France. French constructors were accumulating experience of its properties, and research was being carried on by French engineers very many years before the material came to be regarded as a practical proposition in other countries. Consequently, the Commission appointed in December, 1900, by the French Minister of Public Works embraced a group of engineers whose experience in this material was unrivalled. The work of this Commission, extending over the succeeding six years, included a series of experiments, simple in detail and comprehensive in range, and directed not to the solution of academical minutiæ, but to the obtaining of results immediately applicable to practice. The report of the Commission contains the results of the tests on experimental structures and of the tests to destruction of several of the structures of the Paris Exhibition of 1900. It is unique in the literature of Reinforced Concrete, containing as it does all the necessary scientific data, based on first-hand observations, for the design of reinforced concrete structures, with the observations thereon of a group of engineers of the widest and most mature experience obtainable. The instructions are characteristically French in their clearness and boldness—a boldness derived from intimate knowledge, and entirely justified by results.

The report has been much quoted and extracts have appeared from time to time in various English books and periodicals, but a complete survey of the work of the Commission was to be obtained only from the French edition. The translator has thought that a useful purpose might be served by an abridged English edition which would enable professional men with limited time to acquaint themselves with the scope of the work of the Commission and to have the results at hand in readily accessible form. It will also guide the research student to the detailed records of methods and results to be found in the French edition, and at the same time form one of the easiest avenues of approach for those who wish to make a systematic study of the properties and applications of Reinforced Concrete.

NATHANIEL MARTIN.

Glasgow,
December, 1911.

PREFACE

The rapidity of the development of reinforced concrete structures, the principle of which was indicated by Monier in 1877, is well known. Thanks to the initiative of some bold and skilful constructors, a considerable number of applications of Reinforced Concrete had already been made when on December 19, 1900, the French Minister of Public Works instituted a Commission to study the question from the point of view of his Administration.

Notwithstanding the already numerous experiments, the properties of the new material were still imperfectly known, and the methods of calculation followed by different constructors presented essential differences and even absolute contradictions.

The elementary properties of Reinforced Concrete were incompletely investigated, and it was unknown in what measure these properties permitted application to the new material of the principles and the results which had been laboriously acquired for metallic structures, and which constituted the classic science of the Resistance of Materials.

The Commission decided that they ought first to study the elementary properties of Reinforced Concrete, reserving for later research, in the light of these properties, the interpretation of the complex phenomena arising in structures.

The following are some of the facts established and results obtained by the Commission :—

Experiments of several months' duration have shown the importance of the contraction, denied by certain constructors, which occurs in concrete, not only at the end of the period of setting but also during a long period of hardening, and which influences the distribution of the stresses between the concrete and the metal. The constructor ought to make the necessary arrangements to avoid the undesirable consequences this contraction may produce.

The study of elasticity is the basis of that of stress. The Commission made an important contribution to the former, and verified for the first time the exactitude of the law announced by one of its members concerning concrete under tension. The modulus of elasticity of concrete under tension varies according to a straight line law up to a certain limit, and afterwards becomes almost rigorously constant till rupture. The stronger the reinforcement with bars well distributed in the tension areas, the longer is rupture postponed.

The laws of elasticity of concrete in compression had already been the object of numerous experiments. The Commission made additional experiments without revealing any new facts.

Attention has been drawn to the fact that in compression members, the longitudinal bars necessarily produce resistances proportional to the shortenings which the concrete with which they are associated supports without crushing. Thus arises the idea of the importance of the ductility of the concrete.

By varied experiments, which have confirmed the results announced by one of its members, the Commission has definitely proved that for equal weights transverse reinforcements and especially spirals increase the resistance to compression of the concrete much more than longitudinal bars of the same weight. In preparing the regulations for the employment of transverse reinforcements the French Minister of Public Works may, at first sight, have appeared rather daring, but the expediency of his initiative is now demonstrated by the almost identical regulations on this matter which have been issued in Germany, and particularly in Austria, after the repetition of the experiments inaugurated in France.

The tests by the Commission have given useful but too scanty information on the resistance of Reinforced Concrete to shear and to torsion.

The study of flexion is of prime importance. The Commission has given to it a rational basis by proving by numerous experiments that the conservation of plane sections, which is the foundation of the classic theory of bending, is realised almost as exactly in Reinforced Concrete as in metallic members.

The application to Reinforced Concrete of the exact ideas of the science of flexure, and particularly of those of the neutral axis and of the moment of inertia, has thus been sanctioned. Together with the laws of elasticity, they permit of the determination of the stresses which are developed in statically indeterminate structures. These stresses depend not only on the laws of statics, as in members statically determinate, but also on the deformations.

Light has been thrown by some interesting experiments on the question of the extent to which, from the point of view of resistance to compression, slabs assist the ribs with which they are continuous and form part.

Structures in Reinforced Concrete—slab, floor, footbridge and retaining wall—which were erected for the Exhibition of 1900, were tested to destruction. Made with care and method these tests have given useful information.

Finally, the theoretical studies of the Commission following on its experiments cleared up several questions until then obscure, and have thus given a new impulse to the researches of engineers.

A. CONSIDÈRE.

PARIS,
June 11, 1912.

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INTRODUCTION

The Commission on Reinforced Concrete was instituted by a Ministerial order dated December 19, 1900, and was charged "to study the questions relative to the employment of Reinforced Concrete and to proceed to the necessary researches to determine as far as possible the Regulations which might be framed for the employment in Public Works of this mode of Construction."

The following gentlemen constituted the Commission :—Monsieur Lorieux, President; Monsieur Considère, Reporter; MM. Bechmann, Harel de la Noë, Rabut, Résal, Mesnager, Hartmann, Boitel, Hermant, Gautier, Coignet, Hennebique, Candlot.

At its first meeting on February 16, 1901, the Commission appointed three Sub-Commissions to undertake preparatory work.

The first Sub-Commission, presided over by M. Rabut, undertook the testing to destruction of certain of the reinforced concrete structures of the Paris Exhibition of 1900.

The second Sub-Commission, presided over by M. Considère, was appointed to study the following questions :—

1. Safe limits for tensile and compressive stresses in the concrete.
2. Information to be produced in draft schemes to demonstrate that the different parts of the works are within these limits of security.
3. Time which must elapse between completion of work and application of tests, conditions and duration of tests and the nature of results to be obtained.

This Commission resorted to experimental methods in order to give its propositions an unassailable foundation. It established the programme of the numerous tests carried out under the direction of M. Mesnager at the Laboratory of the School of Bridges and Roads.

The third Sub-Commission, presided over by M. Bechmann, studied questions relative to

1. The production and qualities of the cement, sand and gravel, the preliminary tests intended to indicate the quality of the cement, the proportions of the materials, the quantity of water employed, the methods of mixing, etc.
2. The qualities of the reinforcement.
3. The practical limits of percentage.

The three Sub-Commissions had finished their labours by the commencement of 1905. On April 17, 1905, the Commission nominated M. Considère reporter, and undertook the discussion of the results obtained by the Sub-Commissions. Finally, on January 19, 1906, the Commission presented the following propositions to the Minister of Public Works :—

1. Draft circular to accompany the regulations for structures in Reinforced Concrete.
2. Draft Regulations for structures in Reinforced Concrete.
3. Report in support of these Regulations, followed by complementary notes by M. Considère.

REINFORCED CONCRETE

CHAPTER I

INSTRUCTIONS RELATIVE TO THE USE OF REINFORCED CONCRETE

PART I

DATA FOR DESIGN

A. Imposed Loads.

Article 1.—Bridges in reinforced concrete shall be designed to support the vertical externally applied loads and the wind loads stipulated by the regulations of August 29, 1891.[1]

Article 2.—Superstructures in reinforced concrete shall, unless exception can be justified, be submitted to the imposed loads stipulated in the regulations of February 17, 1903.[1]

Article 3.—The floors and other parts of buildings, retaining walls, conduits under pressure and all other works in which the public safety is involved shall be designed in view of the greatest imposed loads to which they will be exposed in service.

B. Limits of Stress.

Article 4.—The compressive stress allowed in the calculations for reinforced concrete ought not to exceed 0·28 of the crushing resistance acquired by non-reinforced concrete of the same composition after ninety days' setting. The estimated value of this resistance, as measured on cubes of 7·88 inch side, shall be stated in the calculations for each project.

Article 5.—When the concrete is spiralled, or when transverse or oblique reinforcements are introduced and so arranged as to resist more or less efficiently the transverse swelling of the concrete under the influence of compression, the limit of the compressive stress set forth in the preceding article will be increased in a greater or less degree depending on the volume and the efficiency of the transverse reinforcements, provided that the new limit, whatever the percentage of metal employed, shall not exceed 0·60 of the resistance to crushing of the non-reinforced concrete as it is defined in Article 4.

Article 6.—The limit of stress in shear, in the longitudinal slipping of the concrete on itself and in the adhesion of the metal of the reinforcements to the concrete will be 0·10 of the limit of compressive stress defined in Article 4.

[1] See Appendix, p. 109.

Article 7.—The limit of the compressive and tensile stresses in the metal of the reinforcements shall be 0·50 of the apparent limit of elasticity, as defined in the calculations for each project.

For members exposed to shocks or submitted to stresses alternating in sense, such as in some floors, this limit will be reduced to 0·40 of the said apparent limit of elasticity.

Article 8.—In members submitted to very variable stresses, the limits of stress defined above shall be reduced to an extent depending on the variation of stress. This diminution need not exceed in any case 25 per cent. of the values above stated.

The limits of stress shall also be reduced for members exposed to causes of fatigue or enfeeblement, of which the calculations of resistance do not take account, *e.g.*, members such as rail bearers exposed to violent dynamic action.

Part II

Calculations of Resistance

Article 9.—In the calculations of the resistance of works in reinforced concrete there shall not only be taken into account the greatest external forces, including the action of the wind and snow, that the works will have to resist, but also the thermal effects and the effects of the contraction of the concrete in those cases where the concrete is prevented from contracting or expanding freely in the theoretical sense of the word, or where experience does not show that they may be regarded as approximately such.

Article 10.—The calculations of resistance shall be made according to scientific methods resting on experimental data, and not by empirical processes. They shall be deduced either from the principles of resistance of materials or from principles offering at least the same guarantees of exactitude.

Article 11.—The resistance of the concrete to tension will be taken into account in the calculation of the deformations. To determine the stress, however, in any section this resistance shall be regarded as zero in that section.

Article 12.—It must be ensured that compression members are not exposed to buckling. That precaution may be dispensed with when the ratio of the height to the least transverse dimension is less than 20, and where the stress does not exceed the limit specified in Article 4.

Article 13.—The description must indicate the qualities and the proportions of the materials entering into the composition of the concrete. The proportion of water employed in gauging ought to be supervised with care, and should be only sufficient to give the concrete the necessary plasticity for the proper covering of the reinforcements and the filling of all the voids.

Part III

Execution of Works

Article 14.—The framing and the setting of the reinforcements must present sufficient rigidity to resist without sensible deformation the loads and the shocks to which they will be exposed during the execution of the work up till the removal of the framing.

Article 15.—Unless in exceptional cases where the mixture would be poured slow-setting cement shall be used. The concrete shall be rammed with the greatest care in beds of which the thickness will be in proportion to the dimensions of the aggregate used and the spacing of the reinforcements, and shall not exceed 2 inches after ramming, unless stones are used as aggregate.

Article 16.—The distances between the reinforcements themselves and to the faces of the frames shall be such that they permit the perfect ramming of the concrete and the complete surrounding of the reinforcements by the latter. The distance between the reinforcements and the framing, even when mortar without gravel or pebbles is used, ought always to be at least from 0·6 inch to 0·8 inch in order to protect the reinforcement from the weather.

Article 17.—When specially shaped sections are employed for reinforcements instead of round bars, special precautions shall be taken to ensure the complete covering of the reinforcements on all their perimeter, and particularly on any re-entrant angles.

Article 18.—When the concreting of a reinforced concrete member has been interrupted, which should be avoided as much as possible, the old concrete shall be cleaned to the solid and thoroughly moistened in order that it may be completely saturated before fresh concrete is put in contact with it.

Article 19.—In time of frost work shall be suspended if efficient arrangements cannot be made to obviate harmful effects. On restarting the work all parts of the concrete injured by the frost shall be removed, and the procedure will then be as described in the preceding article.

Article 20.—For fifteen days at least after its execution there shall be maintained in the concrete sufficient humidity to ensure setting under good conditions.

The removal of the framing shall be done without shock by purely static forces and only after the concrete has acquired the necessary resistance to support, without damage, the forces to which it will be exposed.

PART IV

TESTS OF WORKS

Article 21.—Works in reinforced concrete which concern the public safety will be tested after completion. The conditions of the tests as well as the interval which must elapse before the works are put into service shall be inserted in the general conditions of the contract. The maximum deflections in the various parts of the works which ought not to be exceeded shall also as far as possible be stated in the general conditions.

The age that the concrete ought to have at the moment of the tests will also be fixed by the general conditions. It will be at least ninety days for large works, forty-five days for works of average importance and thirty days for floors.

Article 22.—The engineers shall record at the time of the tests not only the measurements of deformation and such other measurements as are necessary for the verification of the general conditions of the contract, but also as far as possible any other measurements which might be of service to engineering science.

For works of importance registering apparatuses shall be employed.

Article 23.—Bridges in reinforced concrete shall be tested in the manner prescribed for metallic bridges by the regulation of August 29, 1891.[1]

[1] See Appendix, p. 111.

If it appears desirable to obtain any amendments to the prescription of this regulation, these amendments ought to be justified and inserted in the general conditions.

Article 24.—Superstructures shall be tested in the manner prescribed by the regulation of February 17, 1903, unless amendments are justified.

Article 25.—Floors shall be submitted to a test consisting of the application of the loads and the superloads intended, either to the whole of the floor or at least to an entire span.

The test loads ought to remain in place for at least twenty-four hours, and the deflections ought not to increase at the end of fifteen hours.

LIST OF SYMBOLS

CHAPTER II

A Circular issued by the French Ministry of Public Works in explanation of the Instructions, giving Detailed Methods of Calculating Resistances and Checking Designs (abridged)

List of the Symbols used in Chapters II and III

V	the volume of the concrete in unit length of prism.
V'	the volume of the transverse or oblique reinforcements in unit length of prism.
m'	coefficient varying with the spacing of the transverse or oblique reinforcements and the efficiency of the transverse connection established between the longitudinal bars.
E_s	the modulus of elasticity of steel. (Young's modulus.)
E_c	the modulus of elasticity of concrete in compression.
m	the ratio $\frac{E_s}{E_c}$.
Δ	the equivalent homogeneous section replacing the heterogeneous section composed of concrete and steel.
Δ_c	the sectional area of the concrete,
Δ_s	the sectional area of the longitudinal reinforcements. As Δ_s is generally small in comparison with Δ_c, $\Delta_c + \Delta_s$, the total area, is frequently taken instead of Δ_c.
N	the total compression normal to the section applied at the centre of gravity and consequently uniformly distributed, or the value of the compression applied halfway between the extremities of a section if it varies across the section.
R_c	the unital stress in the concrete.
R_s	the unital stress in the reinforcement on the compression side.
R'_s	the unital stress in the reinforcement on the tension side.
G	the centre of gravity of the equivalent homogeneous section, the position of which is to be found.
G_s	the known centre of gravity of the longitudinal reinforcements.
G_c	the known centre of gravity of the concrete section.
C	the centre of pressure of the resultant force on the cross section.
$Y = GK$	the ordinate of G from an axis xx' chosen at will. (See Fig. 1.)
$Y_s = G_sK$	the ordinate of G_s from the axis xx'. (See Fig. 1.)
$Y_c = G_cK$	the ordinate of G_c from the axis xx'. (See Fig. 1.)
I_s, I_c	the second moments of area (the moments of inertia) of the geometrical sections of the steel and concrete respectively in the cross section of the member, about the axis XGX'.
I	the second moment of area of the equivalent homogeneous section about XGX'.
T	the shear on any section, *i.e.*, the component, tangential to the section, of all the exterior forces, including the support reactions which lie on one side of the section.

LIST OF THE SYMBOLS USED IN CHAPTERS II AND III—*continued.*

M	the bending moment applied to the cross section, *i.e.*, the sum of the moments of the exterior forces relatively to G, the centre of gravity of the equivalent section.
n_c	the unital stress on the concrete distant x from the axis XGX'.
n_s	the unital stress on the steel distant x from the axis XGX'.
x_0	the co-ordinate of the centre of pressure (or point of application of N) relative to the axis XX'.
x_c, x_s	the distance from the extreme edge of the concrete and from the reinforcement respectively to the axis XGX', reckoned negative when measured to the side of XX' on which the bending moment produces tension.
x'	the co-ordinate of the neutral axis relatively to the axis XX'.
r	the radius of gyration of the equivalent section relatively to XX'.
r_1	the minimum radius of gyration. In a symmetrical section either about the axis of symmetry or an axis normal to the latter.
p	the imposed load per unit length of span.
s	the effective span (centre to centre of bearings).
h	the overall depth of slab and rib.
b	the width of the slab.
b'	the width of the rib.
t	the thickness of the slab.
w	gross section of the compression reinforcement.
d	its mean distance to the compression face.
w'	net section of the tension reinforcement.
d'	its mean distance to the tension face.
y_1	the distance of the neutral axis from the compressed face.
y_2	the distance of the neutral axis from the centre of pressure C.
c	the distance of the point of application of the resultant of the exterior forces from the compressed face.
L	the spacing centre to centre of the ribs.
P	the perimeter of the longitudinal reinforcements.
l	the length of a column.
k	a numerical coefficient depending on the end conditions of a column.
K	the "angular coefficient" of the line $A'B'$ in Fig. 3, numerically equal to $\frac{M}{I}$ or $\frac{R_c}{y_1}$.

NOTE.—The quantities must be expressed in the same units throughout. The most generally convenient units are the inch and the lb., with their derivatives, the inch-lb. and the lb. per square inch. These units are applicable without conversion to the formulæ throughout the volume.

Imposed Loads and Working Stresses.

Article 3.—At first sight this article seems simply to be the statement of the very evident necessity of designing a reinforced concrete structure so that the elastic resistances called into play in the various members by the action of the external forces only attain a determined fraction of those which would endanger the stability or the life of the structure. This pre-determined fraction is called the coefficient of security or factor of safety.

The article is really directed, however, against a method in use by certain specialists. This consists, not in determining the elastic resistances called into play to balance the actual loads, but in attempting to determine the proportion in which it would be necessary to imagine the external loads multiplied in order to produce rupture. This coefficient of amplification is called by them the factor of safety.

This procedure will be disallowed in preparing designs, as it is held not necessarily to offer a sufficient guarantee of security. No work has ever perished by the proportional amplification of the loads, but always by accidental cause or by some internal flaw the development of which proved fatal.

Article 4.—The following table provides a comparison of working stresses in compression allowed by foreign regulations with those permitted by the French instructions:—

TABLE No. 1.

Composition of Concrete. (For conversion 1 cubic foot of loose cement was taken as weighing 85 lbs.)	Average crushing resistance in lbs. per square inch estimated by the Commission.		Working stresses allowed by Foreign Regulations, 25 per cent. of (a), lbs. per square inch.	Working stresses allowed by French Instructions, 28 per cent. of (b), lbs. per square inch.
	(a) After 28 days.	(b) After 90 days.		
6 cwts. cement, 28·7 cubic feet of gravel, 14·4 cubic feet of sand or 1 of cement, 3·5 of gravel, and 1·75 of sand by measure.	1520	2275	380	637
7 cwts. cement, 28·7 cubic feet of gravel, 14·4 cubic feet of sand or 1 of cement, 3·12 of gravel, and 1·56 of sand by measure.	1706	2560	426	717
8 cwts. cement, 28·7 cubic feet of gravel, 14·4 cubic feet of sand or 1 of cement, 2·75 of gravel, and 1·37 of sand by measure.	1890	2846	470	798

The French instructions thus permit a working compressive stress of very much higher value than that allowed by foreign regulations. It is pointed out that these regulations are of earlier date, and when they are modified the change will, without doubt, be in the direction of the French figures.

Article 5.—When concrete is prevented from swelling laterally under a longitudinal compression by transverse or oblique reinforcements, or by spiralling, the crushing resistance is augmented.

Experiments made by the Commission indicate that transverse reinforcements and spirals multiply the crushing resistance by the quantity

$$1 + m' \frac{V'}{V}.$$

m' varies with the degree of efficiency of the transverse connections established between the longitudinal bars.

When the transverse reinforcement consists of ligatures forming rectangles on a transverse section of the prism, the coefficient might vary from 8 to 15, the minimum value being applied when the spacing of the transverse reinforcements attains a value equal to the least transverse dimension of the member considered, the maximum value being applied when the spacing of the transverse reinforcements is reduced to at least one-third of that dimension.

When the transverse reinforcement consists of a spiral tie, the coefficient might vary from 15 to 32. The minimum should be applied when the pitch of the spirals is two-fifths of the least transverse dimension of the piece considered, and the maximum when this spacing is reduced to

one-fifth of that dimension for a compression of 711 lbs. per square inch,
one-eighth of that dimension for a compression of 1,422 lbs. per square inch.

Whatever the value of the quantity $1 + \frac{m'V'}{V}$ or the percentage of metal, the compressive stress must not exceed 60 per cent. of the resistance of non-reinforced concrete as defined by Article 4. This ensures that the stress does not exceed half the value at which superficial cracking of the concrete commences, which, according to the experiments of the Commission, varies from 25 per cent. to 60 per cent., depending on the case, of that which produces crushing of the reinforced concrete.

Calculations of Resistance.

Article 10.—This article sets aside all purely empirical processes of calculation. The principles of resistance of the materials employed afford here, as in ordinary structures, more sure solutions.

The experiments described in Chapter IV. show that the principle of Navier relative to the plane deformation of transverse sections is applicable within the limits of the experiments. This, combined with the application of the principle of the proportionality of stresses to the deformation suffices, in the case of compression members.

Each heterogeneous section may be replaced by an equivalent section having the same mass as the real heterogeneous section by attributing to the parts of the section formed by the concrete a density 1 and to the parts formed by the longitudinal reinforcements[1] a density m.

Hypothetically

$$m = \frac{E_s}{E_c} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

This ratio within the limits of the loads defined by Article 4 is about 10. It grows with the stress in the concrete and might be doubled or tripled at the moment of rupture, if failure took place by the crushing of the concrete. On the other hand, it would diminish if failure took place by excess of load on the reinforcements.

This fact alone shows how uncertain would be the calculations of resistance based on the imaginary increase up to rupture of the actual loads,—a method previously referred to.

It is preferable to regard the coefficient m as a figure derived from experience and not as representing exactly the ratio of the moduli of elasticity of the metal and of the concrete separately found. In every case the experiments to determine the modulus of elasticity E_c were carried out in non-reinforced concrete, and it is

[1] The transverse reinforcements do not figure here. Their essential *rôle* has already been considered and allowed for by the increase (Article 5) permitted in the limit of crushing stress of the concrete. It is in the augmentation of the resistance to crushing due to the opposition to lateral swelling in which their efficacy resides.

doubtful if the value thus found is the true value of the modulus of the concrete in reinforced members owing to the difficulty experienced in ramming the concrete between the reinforcements and the moulds.

It may be taken that this coefficient might vary from 8 to 15. The minimum value will be applied when the longitudinal bars have a diameter equal to one-tenth of the smallest dimension of the member, the transverse interties being spaced at this latter dimension and the longitudinal bars being slightly shorter than the column. The maximum will be applied when the diameter of the longitudinal bars is one-twentieth of the smallest dimension of the member and the transverse interties spaced at one-third of this same dimension.

The greater number of authors allow for "m" a fixed value, which is often taken equal to 15. There is thus attributed in many cases, without doubt, to the metal a greater part of the resistance than it actually takes, and to the concrete a less. Trouble might thus arise owing to the fact that the stress in the concrete is greater than the calculation assumes, and consequently the coefficient of security is lower than was intended. By varying "m" as above described, a more accurate determination of the stresses is made, which is necessary in view of the high values allowed in Article 4.

Once the coefficient m has been chosen, the formulæ to be applied may easily be put into the classical form which applies to a homogeneous solid.

Simple Compression.

$$\Delta = \Delta_c + m\,\Delta_s \quad . \quad . \quad . \quad . \quad . \quad (2)$$

$$R_c = \frac{N}{\Delta} \qquad R_s = m\,\frac{N}{\Delta} \quad . \quad . \quad . \quad . \quad (3)$$

If R_c be given, Δ can be calculated from it. Consequently by the help of (2), and knowing the actual total section of the member, Δ_s the total section of the reinforcements, or $\frac{\Delta_s}{\Delta_c}$ the percentage reinforcement, may be found.

Compression with Bending.

When the total compression N is not uniformly distributed, it is necessary to consider, besides the area Δ of the equivalent section, the position of its centre of gravity and its moment of inertia (more properly its second moment of area) relative to the axis passing through its centre of gravity and normal to the plane of bending.

Fig. 1 is a sketch of a section, supposed to be symmetrical about an axis $y'y$ passing through its centre of gravity.

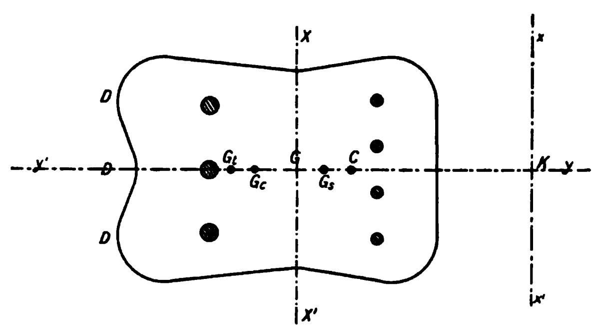


FIG. 1.

The centres of gravity G_s and G_c of the reinforcement and of the concrete are known, as are also Y_s and Y_c, their distances from an axis xx' chosen at will, the abscissæ Y_s and Y_c, etc., being reckoned positive on one side xx', negative on the other.

Δ, the area of the equivalent section, is given by (2). Y and the position of the axis XGX' is then determined from the following equation :—

$$\Delta Y = \Delta_c Y_c + m \Delta_s Y_s \quad . \quad . \quad . \quad . \quad . \quad (4)$$

I_s and I_c are then calculated, and I is then determined by equation (5) :

$$I = I_c + m I_s \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

As previously pointed out, it is more convenient in practice to deal with Δ_t, the total area of the member, than with Δ_c, the area of the geometrical section of concrete, and the error thereby introduced is, almost without exception, negligible:

$$\Delta_t = \Delta_c + \Delta_s.$$

Consequently we also deal with G_t and I respectively, the centre of gravity of the total area of the cross section and the second moment of that area relative to an axis parallel to XGX' passing through G_t.

Formulæ (2), (4) and (5) are then rewritten

$$\Delta = \Delta_t + (m - 1) \Delta_s \quad . \quad . \quad . \quad . \quad . \quad (2')$$

$$\Delta Y = \Delta_t Y_t + (m - 1) \Delta_s Y_s . \quad . \quad . \quad . \quad (4')$$

$$I = I_t + \Delta_t (Y - Y_t)^2 + (m - 1) I_s \quad . \quad (5')$$

Now if in addition to N, there is applied a bending moment M,

$$n_c = \frac{N}{\Delta} + \frac{M}{I} x \quad . \quad . \quad . \quad . \quad . \quad . \quad (5a)$$

and if at the point considered there is found a longitudinal reinforcement, the compression it would carry would be

$$n_s = m\, n_c \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

In $(5a)$ x must be reckoned positive when it represents the distance to points lying to that side of XGX' on which the bending moment produces compression. For example, when the bending moment produces compression between G and K, and tension between G and D, then x must be reckoned positive for all points between G and K and negative for all points between G and D.

The maximum compression in the concrete will then be given by (7) :

$$n_c = \frac{N}{\Delta} + \frac{M}{I} x_c \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

Its least compression will be

$$n_c = \frac{N}{\Delta} - \frac{M}{I} x_c \quad . \quad . \quad . \quad . \quad . \quad . \quad (7a)$$

The compressions in the reinforcements will be obtained by substituting the appropriate values of x_s in (7) :

$$n_s = m \left(\frac{N}{\Delta} + \frac{M}{I} x_s\right) . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

$$n_s = m \left(\frac{N}{\Delta} - \frac{M}{I} x_s\right) . \quad . \quad . \quad . \quad . \quad . \quad (8a)$$

It is an essential condition for the correct application of these formulæ that compression exists throughout the cross section, or that in $(7a)$ and $(8a)$ n_c and n_s are positive. If in these formulæ the resultant stress was negative or a tension, the formulæ would no longer apply, since the laws of tension in concrete differ widely from those expressing the phenomena of its compression. In that case the procedure will be as explained later.

If the centre of pressure of the cross section considered were known, by definition

$$M = N x_o \qquad (9)$$

and

$$I = \Delta r^2 \qquad (10)$$

we have

$$n_c = \frac{N}{\Delta}\left(1 + \frac{x_o x}{r^2}\right) \qquad (11)$$

The neutral axis would by definition be obtained when $n_c = o$, that is when

$$\left(1 + \frac{x_o x'}{r^2}\right) = o \qquad (12)$$

x' being the coordinate defining the position of that axis.

Formula (7*a*) becomes with these notations

$$n_c = \frac{N}{\Delta}\left(1 - \frac{x_o x_c}{r^2}\right) \qquad (13)$$

Comparison of (12) and (13) indicates that there is only compression throughout the cross section when $- x' > x_\omega$ that is when the neutral axis falls outside the section.

The preceding formulæ suppose that N and M are known for each cross section of the member. That is the case for a column carrying a central load—that is, where the load is applied to the centre of gravity G of the equivalent section, and consequently $M = o$, or when the load is eccentric and $M = Nx_o$. This is the case of a dam where the curve of pressures gives N and x_o for each section.

When, however, these values are not directly furnished by statics, the procedure will be as indicated in the much more general case where members provide a resistance composed simultaneously of compression and tension. That is the case which really justifies the employment of reinforcements.

Article 11.—When the ordinary principles of statics furnish the normal and tangential components of the external forces acting on each cross section of the member considered, and also the bending moment at that section, the calculations necessary for the design of a reinforced concrete member may be proceeded with, without any reference to the elastic deformations of the member.

But in the case of continuous beams, or beams wholly or partially built in or arched ribs working in tension, the provisions of Article 11 are applicable.

The Administration are prepared to accept the method of calculation in common use, although it is not at all accurate. It consists in attributing the same coefficient of elasticity in tension as in compression. Once this hypothesis is allowed the formulæ established above, under the restriction that there is only compression on any cross section, become general.

Now, by virtue of the intervention of the equivalent section Δ, these formulæ bring back the resistance of a reinforced concrete member, that is to say, of a heterogeneous member, to that of the resistance of an equivalent homogeneous piece. All the general and classical results which apply in the case of the latter are in consequence capable of application to the former. Thus to determine the values of N, M and T in the case of an arch, or of M and T and the support reactions in the case of a straight beam loaded transversely (where $N = o$), it suffices to adopt the well-known values which apply to homogeneous members.

Thus for a reinforced concrete beam built in at both extremities, the greatest bending moment will be at the building in, and will have for its value

$$\frac{ps^2}{12} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (14)$$

The bending moment at the middle of the span, of sign contrary to the preceding, will be in absolute value

$$\frac{ps^2}{24} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (15)$$

For partial building in an intermediate value between the latter value and $\frac{pl^2}{8}$, the value of the bending moment in a simply supported beam must be chosen.

For continuous span bridges the well-known expressions are applicable to reinforced concrete structures.

In the case of a double-hinged arch the thrust is readily obtained from tables [1] relative to homogeneous arches, and for a built-in arch the tables recently published by M. Pigeaud [2] give similar information. If it is considered that there is only partial constraint, intermediate values between those furnished by the tables must be chosen.

Once the thrust is known, all the necessary data to determine M, N and T graphically or by calculation are obtained for each of the sections it is desired to consider.

A more correct interpretation of the regulation is obtained by taking into account the actual elastic behaviour of concrete in tension. As a result of different experiments it is known that the coefficient of elasticity of reinforced concrete in tension only conserves a value sensibly constant up to a limit of stress, which is the same as the limit of the resistance to extension of a similar concrete non-reinforced. Beyond this limit the concrete becomes in effect plastic, that is, it extends without its resistance to tension being modified, owing to its connection with the reinforcements. There is no difficulty theoretically in building up on this hypothesis, together with that of Navier relating to the plane deformation of plane sections, expressions for the resistance of materials, but the expressions become much more complex.

By one or other of the above methods the values of N, M and T will be found.

From these the unital stresses, at least on the more highly stressed sections, must be determined. In this latter calculation Article 11 prescribes the abstraction of all the resistance to extension of the concrete. This is not in the least contradictory to that which prescribes the taking account of that resistance in the calculations of deformation, since in the latter the abstraction of all the resistance to extension of the concrete would lead to excessive values being put on the deformations and consequently on N, M and T, which depend on the calculated values obtained; on the other hand, the local stresses must be determined on a hypothesis representing the most unfavourable conditions which may possibly supervene in practice. In fact, the concrete cracks more or less from the tension side of each member. These microscopic cracks, or even a fissure at any one place, have very little effect on the deformation of the member as a whole, although the local stress in the reinforcement at the fissure will be materially increased owing to the complete annulling of the tension resistance of the concrete.

[1] Tables de Bresse, published by Gauthier Villars, 55, Quai des Grands Augustins, Paris (6e).
[2] "Annales des Ponts et Chaussées" (Deuxième trimestre 1905, et troisième trimestre 1906). Obtainable from H. Dunod and E. Pinat, 47, Quai des Grands Augustins, Paris (6e).

Example of the Application of the above Principles to a Slab of T Section and to a Beam of Rectangular Section.

The cross section of the member dealt with is shown in Fig. 2.

If there are no compression reinforcements w must be made equal to zero. In Fig. 3 the ordinates of the straight line $A'B'$ give the stresses in the concrete, and

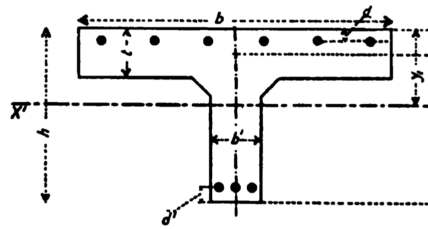


FIG. 2.

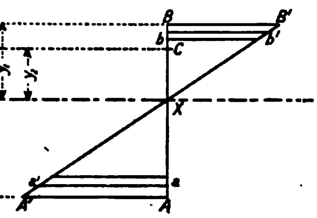


FIG. 3.

the appropriate ordinates multiplied by "m" give the values of the stresses in the reinforcements.

(a) Simple Bending.

$N = o$, the algebraic sum of the elastic forces normal to the cross section is consequently zero. From this equation the unknown distance y_1 to the neutral axis is determined. We may therefore write

$$o = \frac{b'y_1^2}{2} + (b - b')\,t\left(y_1 - \frac{t}{2}\right) + m\,w\,(y_1 - d) - m\,w'\,(h - d' - y_1) \quad (16)^1$$

in which the only unknown is y_1. Also the angular coefficient K is determined thus:

$$\frac{M}{K} = \frac{b'y_1^3}{6} + (b - b')\,t^2\left(\frac{y_1}{2} - \frac{t}{3}\right) + m\,w\,(y_1 - d)\,d - m\,w'\,(h - d' - y_1)\,(h - d') \quad . \quad . \quad . \quad . \quad (17)$$

where K is the only unknown value.

These formulæ implicitly suppose that the neutral axis falls in the rib. If it falls in the slab, it suffices to make $b' = b$ in the above formulæ, which become

$$o = \frac{b\,y_1^2}{2} + m\,w\,(y_1 - d) - m\,w'\,(h - d' - y_1) \quad . \quad . \quad . \quad (18)^1$$

$$\frac{M}{K} = \frac{b\,y_1^3}{6} + m\,w\,(y_1 - d)\,d - m\,w'\,(h - d' - y_1)\,(h - d') \quad . \quad (19)$$

[1] For a ribbed slab with both tension and compression reinforcements in which the neutral axis falls in the rib (see expression 18a, p. 14)—

$$y_1 = \sqrt{\left\{\frac{(b-b')t+m(w+w')}{b'}\right\}^2 + \frac{(b-b')t^2+2m\{wd+w'(h-d')\}}{b'}} - \frac{(b-b')t+m(w+w')}{b'} \quad (a)$$

When the neutral axis falls in the slab or for a member of rectangular section with both tension and compression reinforcements, $b = b'$ and

$$y_1 = \sqrt{\left\{\frac{m(w+w')}{b'}\right\}^2 + \frac{2m\{wd+w'(h-d')\}}{b'}} - \frac{m(w+w')}{b'} \quad . \quad . \quad (b)$$

In the latter case when there is only tension reinforcement in either the ribbed slab or member of rectangular section, $w = o$, and

$$y_1 = \sqrt{\left(\frac{mw'}{b'}\right)^2 + \frac{2mw'(h-d')}{b'}} - \frac{mw'}{b'} \quad . \quad . \quad . \quad . \quad . \quad (c)$$

To know where the neutral axis falls, and in consequence whether its position is determined by (16) or (18), it suffices to replace y_1 by t in the right-hand member of (18), which gives

$$\frac{b\,t^2}{2} + m\,w\,(t - d) - m\,w'\,(h - d' - t). \qquad (18a)$$

If this expression has a positive value, the neutral axis falls in the slab and is determined by (18). When its value is negative the neutral axis falls in the rib and is determined by (16).

Formulæ (18) and (19) apply also to a rectangular section of base b and height h.

When the two unknowns (18) and (19) are determined, the maximum values of the unital stresses are:

$$R_c = K\,y_1 \qquad (20)$$

$$\left.\begin{aligned} R_s &= m\,K\,(y_1 - d) \\ R'_s &= m\,K\,(h - d' - y_1) \end{aligned}\right\} \qquad (21)$$

(b) Composite Flexion (or Flexion Combined with Thrust).

In addition to M and T, as in the previous case, N is also known, as is the centre of pressure C, which is distant c from the compressed face, reckoned positive if it falls in the section and negative if it lies outside the latter. In the following calculations it is better to determine in the first place the position of the neutral axis by its distance XC (y_2) from C rather than by y_1, its distance from the compressed face (Fig. 3).

Since the resultant of the elastic forces coincides with N, the sum of the moments of the elastic forces relatively to C is zero, which gives the following equation to determine y_2:—

$$\frac{b' y_2^3}{6} - b\left[\frac{c^2}{2}\,y_2 + \frac{c^3}{3}\right] + (b - b')\left[\frac{(-c+t)^2}{2}\,y_2 - \frac{(-c+t)^3}{3}\right]$$
$$+ m\,w\,(y_2 + c - d)\,(-c + d) - m\,w'\,(h - d' - c - y_2)\,(h - d' - c) = o \qquad (22)$$

which is of the form

$$y_2^3 + p\,y_2 + q = o. \qquad (23)$$

where

$$\left.\begin{aligned} p &= -\frac{3\,b}{b'}\,c^2 + 3\left(\frac{b}{b'} - 1\right)(c - t)^2 - \frac{6\,m\,w}{b'}\,(c - d) + \frac{6\,m\,w'}{b'}\,(h - d' - c) \\ q &= -\frac{2\,b}{b'}\,c^3 + 2\left(\frac{b}{b'} - 1\right)(c - t)^3 - \frac{6\,m\,w}{b'}\,(c - d)^2 - \frac{6\,m\,w'}{b'}\,(h - d' - c)^2 \end{aligned}\right\} \qquad (24)$$

The term in y_2^2 is missing, which facilitates the solution and justifies the use of y_2 in preference to y_1.

When y_2 has been found, the unknown auxiliary K is obtained immediately by the equation

$$\frac{N}{K} = \frac{b' y_2^2}{2} + b\,c\left(y_2 + \frac{c}{2}\right) + (b - b')\left[(-c + t)\,y_2 - \frac{(-c+t)^2}{2}\right]$$
$$+ m\,w\,[y_2 + c - d] - m\,w'\,[h - d' - c - y_2]. \qquad (25)$$

where K is the only unknown.

Formulæ (22) to (25) inclusive assume that the neutral axis falls in the rib. If it falls in the slab, as also in the case of a beam of rectangular section, base b, height h, it suffices to make $b' = b$, which gives

$$p = -3\,c^2 - \frac{6\,m\,w}{b}\,(c - d) + \frac{6\,m\,w'}{b}\,(h - d' - c) \qquad (26)$$

$$q = -2c^3 - \frac{6\,m\,w}{b}(c-d)^2 - \frac{6\,m\,w'}{b}(h-d'-c)^2 \quad . \quad (27)$$

In the case of a ribbed slab, to know if the neutral axis falls in the rib or in the slab it is sufficient to determine whether or not the first member of the equation (23) has or has not contrary signs at the upper and lower extremities of the rib.

When the unknowns y_2 and K are determined, there is obtained from the former

$$y_1 = y_2 + c$$

for the distance of the neutral axis to the compressed face. The values of the unital stresses on the concrete and reinforcements are then determined, as previously, by formulæ (20) and (21).

Remarks on the Calculation of Slabs.

When a floor is formed of a slab with ribs (Fig. 4), a rib is considered with a part only of the slab $c\,c'$, $d\,d'$, of width $cd = b$, without taking into account the help derived from the neighbouring parts.

This width b should bear a relation to the span and spacing of the ribs and

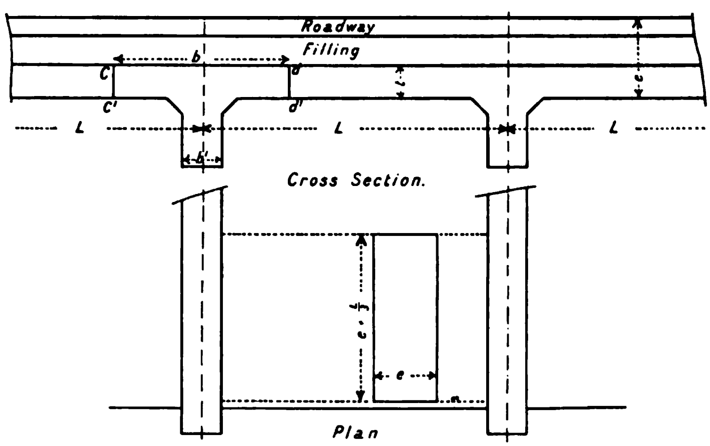


FIG. 4.

the thickness of the slab. It should never exceed one-third of the span s of the ribs, nor three-fourths of their spacing L.

If the floor has to support concentrated loads between the ribs, it ought to be provided with two series of reinforcements at right angles. There is generally given to the feebler reinforcement a total section per metre of width of slab at least equal to one-half of the stronger section per metre length of slab.

To calculate the thickness t of the slab, an isolated load might be replaced by a load uniformly distributed on a rectangle having this load as centre, its sides parallel to the ribs and at a distance apart e equal to the sum of the thicknesses of the slab itself, and of the filling and of the pavement which it carries, its sides, perpendicular to the ribs, having for spacing $e + \frac{L}{3}$. The load thus distributed is supposed to be carried by a band of the floor slab of width $e + \frac{L}{3}$ and of span L, supported on two consecutive ribs.

When a floor is supported by two sets of ribs at right angles, spaced at spans L and L' respectively, the bending moment in the span L may be obtained, in the absence of a better method, by calculating it as if the ribs spaced L apart alone existed, and by multiplying the figure so obtained by the coefficient of reduction:

$$\frac{1}{1 + 2\dfrac{L^4}{(L')^4}}.$$

The bending moment in the span L' is obtained by a similar process and by changing the letters in the coefficient of reduction.

Resistance to Slipping of Reinforcements (Adhesion).

If it has been found that in two neighbouring sections AB, $A'B'$ of a member (Fig. 5), spaced Δs apart, there are unital stresses of R_s' and R_s'', the total stresses on these sections will be $w'R_s'$ and $w'R_s''$ respectively, and the tendency of the reinforcement to slip in its sheath of concrete is measured by the difference

$$w'(R_s' - R_s'').$$

The tendency to slip, per unit of surface of the reinforcement, will then be $\dfrac{w'(R_s' - R_s'')}{P \cdot \Delta s}$, and it is this ratio which must not exceed the limit imposed by Article 6 of the instructions, viz., one-tenth of the maximum compressive stress, or 0·028 of the crushing strength of the concrete after ninety days' setting.

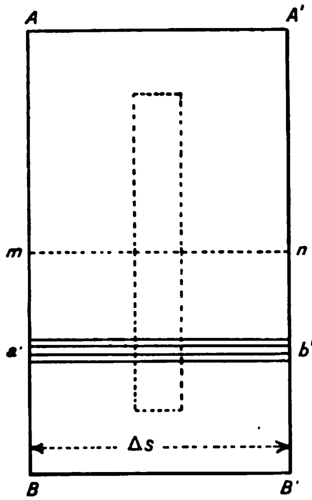


FIG. 5.

When stirrups or other transverse reinforcements are attached to the longitudinal reinforcement in such a manner that slipping of the latter cannot take place without shearing the former, then the shearing resistance F of the transverse pieces occurring on the length Δs of the longitudinal reinforcement considered as the product of the section in shear by the allowable shearing stress for the metal ought to be deducted from the slipping force $w'(R_s' - R_s'')$. It is then sufficient that the ratio

$$\frac{w'(R_s' - R_s'') - F}{P \cdot \Delta s}$$

does not exceed the limit allowed for adhesion.

When there are only simple ties between the transverse and longitudinal reinforcements, these are not sufficient to bring the shearing strength of the transverse pieces into action as a reinforcement lent to the resistance to slipping of the longitudinals. Consequently no account ought to be taken of the shear resistance of the transverse reinforcements. The ties, however, serve other purposes and ought to be provided.

Longitudinal Slipping of the Concrete on Itself and Shearing Resistance.

Consider a portion of the member lying between two transverse sections AB and $A'B'$, distant apart Δs, and having a longitudinal reinforcement $a'b'$ in the tension side of the member. Consider a horizontal section of the member in the stretched part of the concrete, that is, between the reinforcement and the neutral plane and parallel to the latter. Let w_c be the area of this section.

As the tensions in the concrete normally to mB and nB' are not taken account of, the portion $m\,n\,BB'$ of the member is in equilibrium under the influence of the tensions $w'R_s'$ and $w'R_s''$ of the longitudinal reinforcements and of the longitudinal shearing stress on the plane mn. The value of this longitudinal shearing stress per unit area is

$$\frac{w'\,(R_s' - R_s'')}{w_c} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (28)$$

and ought not in any case to exceed the stress allowed for the shearing of the concrete.

This stress (28) remains constant up to the neutral plane. Above that plane it diminishes by the effect of the compression, so that what has been taken account of here represents the maximum stress.

If transverse reinforcements are employed to resist efficiently the longitudinal slipping, they might be taken account of as described in the discussion of adhesion.

The vertical shearing stress at each point is besides, as is well known, the same in magnitude as the longitudinal slipping force just considered.

Buckling of Compressed Pieces.

Article 12.—To ensure that buckling of compressed pieces will not occur, the following inequality, which expresses Rankine's formula, must be satisfied:

$$\frac{N}{\Delta}\left(1 + \frac{k\,l^2}{10{,}000\,r_1^2}\right) < R_c \quad . \quad . \quad . \quad . \qquad (29)$$

The varying values of k to suit the different end conditions met with in practice are as follows:—

TABLE No. 2.

End conditions.	k	Remarks.
Built in at one end, free at the other	4	—
Jointed at both ends . . .	1	—
Jointed at one end, built in at the other.	$\frac{1}{2}$	If the building in is imperfect, a mean value between $\frac{1}{2}$ and 1 should be chosen.
Built in at both ends . . .	$\frac{1}{4}$	If the building in at one end is imperfect, a mean value between $\frac{1}{4}$ and $\frac{1}{2}$ should be taken. If it is imperfect at both ends, a mean value between $\frac{1}{4}$ and 1 should be chosen.

When the compression member is of great length, it happens that unity is negligible in comparison with the number $\frac{k\,l^2}{10{,}000\,r_1^2}$. The inequality which expresses the condition of stability might be simplified thus:

$$\frac{N}{\Delta} \cdot \frac{k\,l^2}{10{,}000\,r_1^2} < R_c,$$

or

$$N < \frac{10{,}000}{k} \cdot \frac{\Delta\,r_1^2}{l^2} \cdot R_c \quad . \quad . \quad . \quad . \quad . \qquad (30)$$

The average value of R_c is about 710 lbs. per square inch, and the coefficient of elasticity of concrete about on the average one-tenth that of steel, that is, $E_c = 2{,}844{,}000$ lbs. per square inch.

The product $10{,}000\ R_c = 7{,}100{,}000$ lbs. per square inch.

The product $\frac{\pi^2 E_c}{4} = 7{,}030{,}000$ lbs. per square inch.

These are sensibly equal, so that (30) may be rewritten :

$$N < \frac{\pi^2}{4k} \cdot \frac{\Delta r_1^2}{l^2} \cdot E_c \qquad . \quad . \quad . \quad . \quad . \quad (31)$$

which is Euler's formula with a coefficient of security of 4. It is thus seen that the indications furnished by this formula agree with those of the Rankine formula for very long pieces.

If in addition to the purely compressive stress on the member there is a bending moment, the effect of which cannot be considered negligible, *e.g.*, the case of an eccentrically loaded column, or of a long column exposed to wind pressure, the maximum compressive stress due to this bending moment must be introduced into the inequality (29) in order to completely state the conditions of stability.

The stress due to this bending moment is expressed by

$$N_c = \frac{M x}{I} \qquad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (31a)$$

or by

$$N_c = \frac{N x_o x}{\Delta r^2} \qquad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (31b)$$

Rankine's formula is then represented by one or the other of the following inequalities :—

$$\frac{N}{\Delta}\left(1 + \frac{k l^2}{10{,}000 r_1^2}\right) + \frac{M x}{I} < R_c \qquad . \quad . \quad . \quad . \quad (32)$$

$$\frac{N}{\Delta}\left(1 + \frac{k l^2}{10{,}000 r_1^2} + \frac{x_o x}{r^2}\right) < R_c \qquad . \quad . \quad . \quad . \quad (33)$$

Execution and Tests of Works.

The Commission point out that comment on the remaining articles of the Instructions is superfluous. They limit themselves to the remark regarding the execution of works that in reinforced concrete construction, perfection of execution is the essential condition of success. Accidents which have happened are in general due to the mediocre quality of the materials used or their improper employment. It is necessary to exercise a special supervision over the production and the purity of the materials used, their mixture, the quality and the quantity of the water used in the manufacture of the concrete, the ramming and placing of the concrete round the reinforcements, and the stability of the latter till the enveloping concrete is properly in position.

CHAPTER III

ANNEX TO THE EXPLANATORY CIRCULAR BY THE FRENCH MINISTRY OF PUBLIC WORKS, BEING A REPORT ON THE DRAFT REGULATIONS OF THE GOVERNMENT COMMISSION BY THE COMMISSION NOMINATED BY THE GENERAL COUNCIL OF BRIDGES AND ROADS (abridged).

As explained in the Introductory Note, this commission, consisting of M. Maurice Lévy as President and Reporter, and MM. de Préaudeau and Vétillart, reconsidered the whole question of the draft regulations presented to the Minister of Public Works by the larger Government Commission, and as a result of their labours the regulations took the form which appears in Chapter I.

They suggested the substitution of the word "Instructions" for the word "Regulations," as conveying a sense of less permanence and finality without losing any of the obligatory character of the latter.

They also reconsidered the propriety of authorising the high working stress proposed by the Government Commission. After much consideration and discussion, they came to the conclusion that, taking into account the methods of calculation by which the stresses would be arrived at, the limits defined in Articles 4 and 5 would be safe in practice.

The fundamental premise of all calculations in reinforced concrete is the ratio expressing the equivalence of equal sections of reinforcement and of concrete represented by the symbol "*m*." The Government Commission did not come to a unanimous decision on the value of *m*. Two members, MM. Rabut and Mesnager, were of opinion that this number should be taken equal to 10, whilst the majority of the commission decided that it should have a value ranging from 8 to 15. The German and Swiss regulations, as well as the majority of French and Belgian writers, adopt the value 15.

Hypothetically *m* is the ratio between the moduli of elasticity of the reinforcement and of the concrete. From the experiments of M. Mesnager, which agree closely with those of Professor Bach of Stuttgart, this ratio up to a stress of 850 lbs. per square inch on a concrete composed of 6 cwts. Portland cement to 14·3 cubic feet of sand to 28·7 cubic feet of gravel, has the value 10. With the value 15 there is often attributed to the metal a greater share of the load than it really takes, so that the concrete is more highly stressed than the calculations indicate. Danger thus arises from the use of a fixed value of *m*.

The following considerations concerning the closely interrelated quantities, "*m*," the working stress and the coefficient of security, lead the Commission nominated by the General Council of Bridges and Roads to agree to the figures appearing in the circular accompanying the instructions.

Consider a column of reinforced concrete in which the calculated stress is 710 lbs. per square inch. A test cube of the same concrete not reinforced broke after ninety days' setting under a load of 2,840 lbs. per square inch. The column would then be said to have a coefficient of security of 4. This is, of course, only a conventional value. The real value of the coefficient of security can only be determined by the testing to destruction of the actual structure.

This actual coefficient of security was determined in the case of five experimental columns on which very precise rupture experiments were carried out by Professor Bach of Stuttgart. The breaking loads experimentally determined were compared with the calculated stresses resulting

1. From the employment of the working stresses allowed by foreign regulations and m constant and equal to 15.

2. From the employment of the working stresses allowed by the instructions, taking advantage of the coefficient of increase $\left(1 + m' \frac{V'}{V}\right)$ and making m vary from 8 to 15.

The columns tested had the cross section shown on Fig. 6 and a length of 39·37 inches. The area of each column was thus 96·9 square inches. Each was reinforced with four rods varying from 0·59 to 1·18 inches in diameter and spaced at 7·09-inch centres. These longitudinal rods were united in pairs by rods of 0·276 inch diameter, forming double transverse ligatures, forming the four sides of a square and spaced longitudinally from 2·46 inches to 9·84 inches apart. V', the volume of each set of four ligatures, was 3·83 cubic inches.

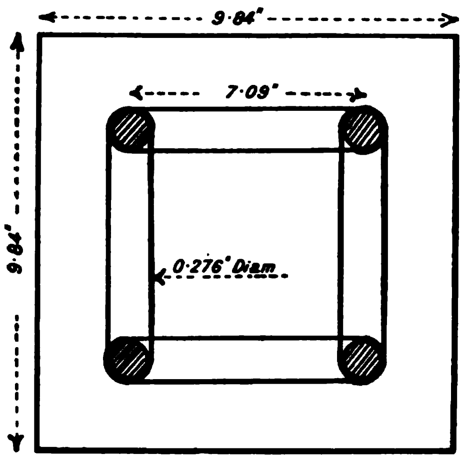


FIG. 6.

Table No. 3 is a *résumé* of the five series of experiments.

TABLE No. 3.

1.	2.	3.	4.	5.
No. of Experiment.	Diameter of Longitudinal Reinforcements.	Spacing of Transverse Reinforcements.	Mean Value of the Breaking Load.	Total Cross Section of the Longitudinal Reinforcements.
	Inches.	Inches.	Lbs. per square inch.	Square Inches.
1	0·59	9·84	2,390	1·10
2	0·59	4·92	2,520	1·10
3	0·59	2·46	2,915	1·10
4	0·79	9·84	2,420	1·95
5	1·18	9·84	2,700	4·39

The breaking stress of a non-reinforced concrete column was found to be 2,020 lbs. per square inch and that of a cube of this concrete 2,500 lbs. per square inch.

Supposing $m = 15$ and taking $R_c = 500$ lbs. per square inch, which would conform to the German regulations,

$$N = 500\,(96{\cdot}90 + 15\,w).$$

TABLE No. 4.

No. of Experiment.	$15\,w$.	$(96{\cdot}90 + 15\,w)$.	N.	$\frac{N}{96{\cdot}9}$.	Breaking Loads.	Effective Co-efficient of Security.
	(Inches).^{2}	(Inches.)^2	Lbs.	Lbs. per inch.^{2}	Lbs. per inch.^{2}	
1	16·4	113·3	56,400	582	2,390	4·1
2	16·4	113·3	56,400	582	2,520	4·3
3	16·4	113·3	56,400	582	2,915	5·0
4	29·3	126·2	62,800	648	2,420	3·5
5	65·7	162·6	81,000	834	2,700	3·2

It is seen that the effective coefficient of security is a very variable quantity. It varies between 5 and 3·2, which indicates that the hypothesis of $m = 15$ might lead to serious miscalculations.

Now in terms of Article 4 of the instructions, allowing a stress of 710 lbs. per square inch instead of 500 lbs. per square inch, as in the above calculations, and in virtue of Article 5 we increase this stress according to the coefficient of increase

$$1 + m'\frac{V'}{V},$$

which leads to

$$R_c = 710\left(1 + m'\frac{V'}{V}\right).$$

The safe working loads N for the various columns are given by the formula

$$N = R_c\,(96{\cdot}9 + m\,.\,w).$$

The values of m' and m indicated in the circular are adopted and are given in Table No. 5.

TABLE No. 5.

1.	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.
Nos.	m.	$m\,w$.	$96{\cdot}90 + m\,w$.	Spacing of Transverse Reinforcements.	m'.	$\frac{V'}{V}$.	$R_c = 710\left(1 + m'\frac{V'}{V}\right)$.	N.	$\frac{N}{96{\cdot}9}$.	Effective Coefficient of Security.
		(Inches.)^2	(Inches.)^2	(Inches).				Lbs.	Lbs. per inch.^{2}	
1	10	11·01	107·91	9·84	8	0·00401	734	79,200	818	2·9
2	12	13·18	110·08	4·92	12	0·00802	779	85,800	886	2·8
3	15	16·43	113·33	2·46	15	0·01604	882	121,800	1,258	2·8
4	9	17·52	114·42	9·84	8	0·00401	734	105,900	1,094	2·8
5	8	35·03	131·93	9·84	8	0·00400	734	96,850	1,000	2·7

The figures in column 11 display a remarkable constancy and admit of confidence in the values of "m" and "m'" set forth in the Instructions, and also justify the use of the high-working stress.

CHAPTER IV

SHORT DESCRIPTION, WITH RÉSUMÉ OF THE RESULTS, OF THE EXPERIMENTS CARRIED OUT AT THE LABORATORY OF THE NATIONAL SCHOOL OF BRIDGES AND ROADS UNDER THE PROGRAMME OF THE SECOND SUB-COMMITTEE.

1. Measurement of Contraction during Setting.

To measure the contraction during the setting of cement concrete, four prisms, each 3·28 feet long and of approximately 6-inch side, were moulded vertically, two reinforced and two without reinforcement. One of the non-reinforced prisms was kept in the Court of the Laboratory covered with sacks and watered from time to time, the others were kept dry on supports in a closed shed.

The results are discussed in Chapter V., p. 70.

2. Measurement of Elasticity of Concrete without Reinforcement.

Two prisms of concrete, containing 6 cwts. Portland cement, 28·7 cubic feet of gravel and 14·4 cubic feet of sand mixed to a plastic consistency, each 3·28 feet long and of square section of 1·64 feet side, were subjected to compression in a hydraulic press after setting for about $10\frac{1}{2}$ months.

Table No. 6 gives the results obtained.

TABLE NO. 6.

Compressive Stress.	Modulus of Elasticity.			
	Load Applied Normal to Planes of Ramming.		Load Applied Parallel to Planes of Ramming.	
Lbs. per square inch.	Lbs. per square inch.	Tons per square inch.	Lbs. per square inch.	Tons per square inch.
From 64 to 580	$4{\cdot}20 \times 10^6$	1,875	$3{\cdot}83 \times 10^6$	1,710
From 64 to 970	$4{\cdot}01 \times 10^6$	1,790	$3{\cdot}77 \times 10^6$	1,683
From 64 to 1,360	$3{\cdot}87 \times 10^6$	1,728	$3{\cdot}60 \times 10^6$	1,608

3. Determination of the Resistance to Crushing of Cement Concrete prepared in Different Degrees of Plasticity.

Three prisms each 7·87 inches square and 19·68 inches long were cast vertically in moulds, the concrete consisting of 12 cwts. of Portland cement to 28·7 cubic feet of gravel and 14·4 cubic feet of sand.

Prism No. 1.—The concrete was mixed with 11·6 per cent. of water by weight of dry mixture and was sufficiently wet to pour into the mould, which was completely filled in that way.

[1] *Prism No.* 2.—The mould was filled for three-quarters of its height with material similar to that in Prism No. 1; the remainder of the mould was filled with successive layers of dry mixture strongly rammed.

Prism No. 3.—The mould was filled with ordinary plastic concrete rammed.

The tests in each case were made fifty-four days after moulding.

TABLE No. 7.

—	Cross Sectional Area of Prisms in square inches.	Weight in Lbs. per cubic foot of Concrete of Prisms.			Resistance to Rupture by Crushing.
		After 7 days.	After 26 days.	After 54 days.	Lbs. per square inch.
Prism No. 1 .	63·24	140·8	138·4	137·7	2,092
Prism No. 2 .	63·23	143·2	142·2	141·8	> 2,510 [2]
Prism No. 3 .	62·46	141·6	140·0	139·5	2,440

4. Influence of the Percentage of Reinforcement on the Stresses developed during Setting.[3]

Nine pairs of cylinders were manufactured, each 19·7 inches long and 3·9 inches in diameter, each pair containing a similar reinforcing rod, the area of which varied throughout the series from a percentage of 0·23 to 9·0. One cylinder of each pair consisted of 6 cwts. of Portland cement, 28·7 cubic feet of gravel and 14·4 cubic feet of sand, the other of 10 cwts. of Portland cement to the same quantities of sand and gravel. The cylinders of the former mixture were kept in air, those of the latter in water.

No very definite results were obtained; but at the end of seven months it was found that the shortening of the reinforcements of the test cylinders kept in air was 0·018 per cent. when the percentage reinforcement was 0·23, and 0·010 per cent. when the percentage reinforcement was 9. The corresponding percentage elongations of the reinforcements of the cylinders kept in water were 0·010 per cent. and 0·009 per cent. when the percentage reinforcements were 0·23 and 9·0 respectively.

The shortenings and elongations were measured on a length of 1·64 feet.

5. Tension Tests.[4]

Four prisms of square section 3·94-inch sides and 78·8 inches long, reinforced with four round rods, each 0·24 inch diameter, giving a percentage reinforcement of 1·13, were made of concrete consisting of 6 cwts. of Portland cement, 28·7 cubic feet of gravel and 14·4 cubic feet of sand, and gauged with 8·8 per cent. of water by weight of dry mixture.

[1] See note on p. 74.
[2] This stress only produced fissures without leading to total rupture.
[3] For notes on these tests by M. Considère, see pp. 105, 106.
[4] For notes on these tests by M. Considère, see Chapter VII., 1.

Prism No. 1 was tested by a gradually increasing load until cracking was observed. Under a load of 550 lbs. per square inch, when the elongation reached 0·135 per cent. of the original length (measured on one metre), fissures were observed.

Prism No. 2 was tested by repeatedly removing the load and reapplying it. Twenty-five repetitions were made. The last load applied was 293 lbs. per square inch, which produced an elongation of 0·061 per cent. of the original length without producing any visible cracks. The permanent deformation under this load was 0·0265 per cent. of the original length.

The reinforcing rods were carefully cut out and the concrete of the prism subjected to bending tests. The breaking load in the first of these bending tests corresponded to a tension of 133 lbs. per square inch, and in the second to a tension of 126 lbs. per square inch.

Prism No. 3.—This prism was tested similarly to No. 2. After the tenth loading the permanent deformation found on unloading was observed not to increase, and after the twentieth loading the load was allowed to remain on for fifteen hours. No appreciable increase in the extension took place. The load applied in each case was 255 lbs. per square inch, and the extension due to the twenty-fifth application was 0·0165 per cent. of the original length. The total extension from the commencement of the experiment was 0·0435 per cent. of the original length without any appearance of fissures.

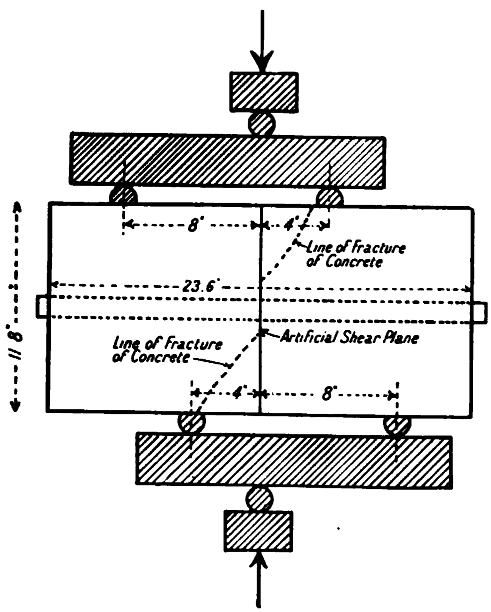


Fig. 7.

Prism No. 4.—This prism was tested as was Prism No. 1 by a gradually increasing load. When the load had reached 540 lbs. per square inch and the elongation 0·13 per cent. of the original length, fissures appeared uniformly over the whole length of the prism at intervals of from 3 to 6 inches.

From a portion of the concrete of Prism No. 1 which had been loaded to 550 lbs. per square inch, the coefficient of elasticity of concrete in compression was determined and the following values were obtained:—

For stresses between

70 and 700 lbs. per square inch, $2{\cdot}105 \times 10^6$ lbs. per inch^2 or 940 tons per inch.^{2}
700 and 1,140 lbs. per square inch, $2{\cdot}180 \times 10^6$ lbs. per inch^2 or 973 tons per inch.^{2}
70 and 1,140 lbs. per square inch, $2{\cdot}130 \times 10^6$ lbs. per inch^2 or 951 tons per inch.^{2}

The steel wire used in the manufacture of the prisms had an apparent limit of elasticity of 19·8 tons per square inch, a breaking strength of 27 tons per square inch, with an elongation of 20·5 per cent. on 7·87 inches, and a modulus of elasticity of 13,000 tons per square inch.

6. Shearing Tests.

Six rectangular prisms, each 11·8 inches × 11·8 inches × 23·6 inches long, were made in three series, each consisting of concrete containing 6 cwts. of Portland cement, 28·7 cubic feet of gravel and 14·4 cubic feet of sand.

The prisms were moulded in wooden boxes in two halves, with the reinforcements vertical. As soon as the first half of each prism was set it was turned in the mould and the other half formed, a sheet of oiled paper being placed between the halves to secure a division plane.

Fig. 7 shows the prism with division plane and the method of applying the shear, and Table No. 8 gives a *résumé* of the results obtained.

The planes along which fissuring took place are indicated by dotted lines on the diagram.

1st series of prisms were reinforced with six sheet-iron straps 1·575 inches × 0·084 inch, giving a percentage reinforcement of the cross section of 0·57.

2nd series of prisms were reinforced with two round rods each 0·716 inch diameter, giving a percentage reinforcement of 0·58.

3rd series of prisms were reinforced with six rods of 0·394 inch diameter, giving a percentage reinforcement of 0·52.

TABLE NO. 8.

Shearing Effort Applied to the Division Plane. Total Stress. Lbs.	Shearing Effort Applied to the Division Plane. Unital Stress per square inch of Reinforcement. Tons.	Mean Relative Displacement of the two halves. Inches.	Remarks.
		SERIES I.—*1st Prism.*	
29,730	16·57	0·50	Maximum load attained.
		2nd Prism.	
7,750	4·32	—	Stress beyond which a movement imperceptible to the eye was indicated.
18,640	10·41	—	Relative displacement became visible.
25,600	14·29	—	Sudden displacement.
28,600	14·94	0·50	Maximum load. Concrete fissured.
		SERIES II.—*1st Prism.*	
809	0·45	0·0005	—
6,590	3·68	0·0044	—
13,550	7·56	0·0163	—
14,690	8·13	0·0263	—
20,700	11·53	0·70	Prism fissured.

TABLE No. 8—(*continued*).

Shearing Effort Applied to the Division Plane. Total Stress. Lbs.	Shearing Effort Applied to the Division Plane. Unital Stress per square inch of Reinforcement. Tons.	Mean Relative Displacement of the two halves. Inches.	Remarks.
			SERIES II.—2nd *Prism*.
6,590	3·68	—	Stress beyond which a movement imperceptible to the eye was indicated.
18,850	10·42	—	Visible displacement took place.
23,280	12·89	0·36	Concrete fissured.
			SERIES III.—1st *Prism*.
13,550	8·27	—	Relative displacement became visible.
25,300	15·50	0·33	Concrete fissured.
			2nd *Prism*.
17,020	10·43	—	Relative displacement became visible.
24,870	15·27	0·43	Concrete fissured.

The steel reinforcing the prisms used in the shearing tests had the properties indicated by the following figures :—

TABLE No. 9.

—	Practical Elastic Limit.	Breaking Load on Original Section.	Elongation per cent. on 8 inches.	Modulus of Elasticity.
	Tons per inch.[2]	Tons per inch.[2]		Tons per inch.[2]
Sheet steel, 1·57 inches × 0·084 inch.	20·7	27·5	21·5	13,000
Round rod, 0·394 inch diameter.	18·8	26·5	25·7	15,500
Round rod, 0·717 inch diameter.	15·8	24·0	33	14,800

7. Torsion Tests.

Two cylinders, each 39·37 inches long and 4·21 inches diameter, were reinforced longitudinally by four steel wires placed symmetrically in the section about 0·47 inch from the surface. The concrete was of the same proportions as that described on p. 25. The moulds were placed vertically and filled in three parts, one immediately after the other.

As soon as the cylinders were removed from the moulds, square heads for the purpose of applying the torsion were moulded on each extremity.

In the application of the test the cylinders were kept horizontal and the torsion applied by means of a lever. The variations in length of the cylinders during the test were measured on one of the reinforcing rods, the ends of which projected beyond the concrete.

The results obtained were the following :—

Table No. 10.

Moment of Torsion. Foot-lbs.	Total Elongations per cent. of Original Length. Measured on 39·37 inches.	Remarks.
	1. *Cylinder with Reinforcements* 0·213 *inch diameter.*	
36	0·0	—
108	0·002	—
253	0·010	—
290	0·013	Rupture took place when torsion was increased beyond this value.
	2. *Cylinder with Reinforcements* 0·300 *inch diameter.*	
36	0·0	—
108	0·001	—
253	0·005	—
325	0·010	A crack appeared at this load.
360	0·016	Rupture took place immediately after this torque was applied.

The direction of the principal fissures with the generator of the cylinder was 61° for the first cylinder and 45° for the second.

8. Resistance to Slipping of Reinforcements.

1. Four test pieces, consisting of mild steel cylinders 4·18 inches diameter, rough from the rolls, were enveloped for a length 7·87 inches in a square block of concrete of 12·20-inch side, otherwise not reinforced. The composition of the concrete was 6 cwts. Portland cement 28·7 cubic feet gravel, 14·4 cubic feet sand, and it was filled into the mould by ramming.

In the case of two of the test pieces tension was applied to the steel cylinder and in the other two a thrust was applied.

The age of the concrete at the time of the test was seventy-two days, and failure took place by the splitting of the concrete block, in the case of the first test piece, when a load was applied corresponding to a stress of 97 lbs. per square inch of area of contact of the metal and concrete. In the case of the second, failure took place at 190 lbs. per square inch, and in the case of the third and fourth test pieces to which a thrust was applied at stresses of 215 lbs. per square inch and 235 lbs. per square inch respectively. The low stress obtained in the first case is accounted for by a defect in the testing apparatus.

2. Four test pieces similar in all respects to the above, but having the concrete surrounding the steel cylinder reinforced with two rings of sheet steel 1·18 inches × 0·23 inch, and with three longitudinal wires 0·98 inch diameter, were similarly tested. After six months' setting slipping took place when stresses of 347 and 360 lbs. per square inch of area of contact between the concrete and the steel had been applied by tension and when stresses of 356 and 464 lbs. per square inch had been applied by thrust. In each case no cracking of the enveloping cylinder of concrete occurred when the first slipping of the cylinder through its concrete envelope took place, which slipping amounted to about 0·08 inch. The cylinder was able after this initial slipping had occurred to resist a stress tending to cause further slipping amounting on the average over the four specimens to 303 lbs. per square inch.

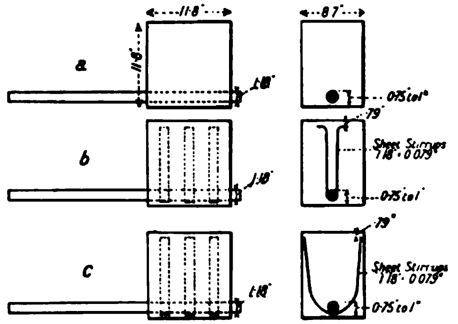


FIG. 8.

3. Test pieces representing parts of beams and formed as indicated in Fig. 8 were prepared. They were divided into three series according to the proportions of cement employed in the concrete:—

Series I.	3 cwts. cement,	28·7 cubic feet gravel, and	14·4 cubic feet sand.	
Series II.	6 „ „	28·7 „ „	14·4 „ „	
Series III.	10 „ „	28·7 „ „	14·4 „ „	

They were tested by tension applied to the projecting end of the reinforcement.

The test pieces were non-reinforced as at "*a*," reinforced with ordinary (Hennebique) stirrups as at "*b*" or reinforced with open stirrups as at "*c*."

The results obtained are given in Table No. 11, p. 29.

In the case of the test pieces without stirrups a crack occurred in each case along the middle of the lower face following the reinforcing bar at the moment of slipping. In none of the others were fissures observed.

4. *Additional Tests on the Slipping of Reinforcements in Concrete made with French Quick-setting Cements.*—Six test pieces were made of the form and dimensions shown in Fig. 8, of type "*a*," without transverse reinforcements of any kind. The concrete in each case was of the proportions of Series II.

With a cement setting of which commenced in nine minutes and was complete in thirteen minutes, the average resistance to slipping per square inch of the surface of contact of reinforcement and concrete was 60 lbs. per square inch after twenty-four hours, and 232 lbs. per square inch after twenty-eight days.

With a cement which commenced setting in air in one hour and was set in two hours, the resistance was 31 lbs. per square inch after twenty-four hours and 91 lbs. per square inch after twenty-eight days.

5. Tests were made of the resistance to slipping of the wires in an old reinforced concrete sleeper. The diameter of the wires was 0·197 inch.

TABLE No. 11.

—	Stress which produced the First Slipping of the Reinforcement.	Maximum Stress under which Slipping ceases after the First Slipping.	Average Stress under which the Reinforcement leaves its Bed, when the Displacement reaches 0·2 inch.	Age of Test Piece.
	Lbs. per square inch.	Lbs. per square inch.	Lbs. per square inch.	Months.
SERIES I.				
(a) Without stirrups . .	317	102	60	3
(b) With ordinary stirrups .	267	236	182	3
(c) With open stirrups .	304	182	142	3
SERIES II.				
(a) Without stirrups . .	400	155	115	3
	422	161	141	3
	230	158	119	3
	102	91	115	6
	283	232	202	6
(b) With ordinary stirrups.	411	311	284	3
	300	245	200	3
	404	300	272	3
	284	245	245	6
	240	215	182	6
(c) With open stirrups .	305	262	226	3
	472	353	312	3
	556	357	313	3
	365	298	259	6
	424	383	302	6
SERIES III.				
(a) Without stirrups . .	433	146	148	3
(b) With ordinary stirrups.	579	483	417	3
(c) With open stirrups .	454	375	326	3

Longitudinally the wires were not straight, but presented considerable undulations. Slipping of the reinforcement took place generally about 1,150 lbs. per square inch of the surface of contact, although in one case slipping took place at

820 lbs. per square inch. In three cases the wire broke outside the concrete before slipping had taken place when the stress had reached 1,050, 1,310 and 590 lbs. per square inch respectively of the surface of contact between the wire and the concrete.

9. Bending of Beams.

Twenty-three specially manufactured beams were tested, and a *résumé* of the results obtained is given in Table No. 12, pp. 32-38.

Fig. 9 shows in outline the method of applying the test loads to the beams *A* to *G* inclusive. By this arrangement there is obtained a uniform bending moment between the points of application of the load, and a uniform shear between the latter and the points of support of the beam.

The sand and gravel used were carefully measured by weight, allowance being made for moisture in the sand and for sand and moisture in the gravel. When

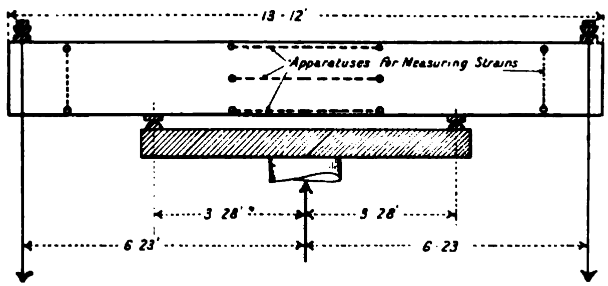


FIG. 9.

beam *A* was prepared the sand contained 4 per cent. of its weight of water and the gravel 2·7 per cent. The results of this method, however, were sensibly the same as those obtained by the volumetric methods used in practice.

The granulometric composition of the materials used was as follows :—

Sand.

Weight per cubic foot = 99·7 lbs.

Granulometric composition, weights per cent.

0·197 inches diameter to 0·079 inches diameter . . .	25·6
0·079 ,, ,, 0·0197 ,, ,, . . .	48·6
0·0197 ,, ,, 0·0 ,, ,, . . .	25·8
	100

Gravel.

Weight per cubic foot = 93·4 lbs.

Granulometric composition, weights per cent.

0·984 inches diameter to 0·787 inches diameter	3·5
0·787 ,, ,, 0·394 ,, ,, ...	71·7
0·394 ,, ,, 0·197 ,, ,, ...	24·8
	100

For the tests of beams *J* to *U* inclusive an alteration in the spacing of the points of support and of application of the loads was made, giving the ends of the beam a greater overhang. The effect of the projection of reinforcements beyond the points of support can thus be studied. The arrangement is outlined in Fig. 10.

In beam *H*, the whole load was applied as a concentrated load midway between the points of support, whilst in beams *I*, *V* and *W* the points of application of the load, were 3·28 feet apart.

Manet-Rabut apparatuses for measuring strains were fixed to the beams in the positions indicated in Figs. 9 and 10 by rods built into the concrete. The strains in the reinforcements were measured by similar apparatuses fixed directly to the latter, which were exposed for a short length by careful chiselling.

The relative slipping of the concrete and the reinforcement was obtained in either of three ways :—

1. By the use of an apparatus fixed to the concrete at the end of the beam, operated by a rod from the end of the reinforcement, bared for the purpose.

2. By means of an apparatus fixed to the concrete, and measuring the relative movement of the reinforcement in a vertical plane normal to the beam 1½ inches distant.

3. By means of a microscope fixed to the concrete, and reading on a reference mark on the reinforcement in the plane of the microscope.

The deflections were measured by means of a Rabut recording apparatus

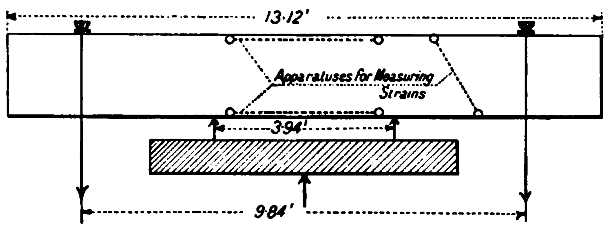


FIG. 10.

fixed to each face of the beam midway between the tie rods, whilst the extension of the latter and the deflections of the loading system were also recorded, so that the actual values of the deflections of the beam were obtained.

All the beams had a length over-all of 13·12 feet, with the exception of beams *U* and *V*, which had an over-all length of 9·84 feet.

The ends of the longitudinal reinforcements of beams *A*, *B*, *C*, *D*, *F* and *G* were fish-tailed according to the Hennebique practice, whilst in beam *E* they projected 2 inches at each end for measurement purposes. In beams *H* to *T* inclusive, with the exception of *I*, the longitudinal reinforcements were cut to a uniform length, 1½ inches shorter than the beam, and left simply with square-cut ends, whilst in beams *I*, *U*, *V* and *W* each end of the longitudinal reinforcements was returned square upwards for a length of 2¾ inches.

Beams *A* to *G* inclusive had measuring apparatuses attached to the vertical faces of the beams near the upper and lower faces and also at the neutral axis for the purpose of verifying Navier's hypothesis, viz.—that plane sections remain plane during bending. The result obtained in the case of beam *D* is graphically represented in Fig. 11, p. 39.

Beams *H* to *M* inclusive give a comparison between flat and round stirrups of equal sectional area.

Beams *M*, *N*, *O* allow of the study of the effect of the splitting up to the transverse reinforcement into a greater or less number of stirrups.

TABLE No. 12.

1.	2.	3.	4.		5.		6.	7.	8.	9	10.	11.
Reference Letter.	Span.	Depth × Breadth.	Longitudinal Reinforcement.		Transverse Reinforcement.		Composition of Concrete.	Age at Test.	Results of Test.			
			—	Per cent.	—	Total Section per Foot of Length.			Total Loads Applied.	Bending Moment.	Deflection in Length of 6·56 Feet at Middle.	Remarks.
	Feet.	Inches.				Sq. Inches.		Days.	Tons.	Foot-tons.	Inches.	
A.	12·47	15·75 × 7·87	Four round bars, each 0·874 ins. diameter, 0·80 in. clear from tension face.	1·94	For 3·28 feet at each end, with stirrups 1·57 ins. × 0·049 ins. at 9·84 ins. centres on each longitudinal.	0·76	Six cwts. Portland cement; 14·3 cub. ft. of sand; 28·7 cub. ft. of gravel; water 8·8 per cent. of weight of dry material.	90	10·08 19·38 23·10	14·85 28·60 34·10	0·06 0·107 —	— An oblique fissure occurred near one end. Maximum load attained.
B.	Do.	Do.	Do.	Do.	Do.	Do.	Do.	146	7·29 11·02 12·25 0	10·66 16·26 18·08 0	0·033 0·037 0·057 0·014	— — Fissure on tension face. Permanent deformation.
C.	Do.	Do.	Two round bars, each 1·575 ins. diameter, 0·80 in. clear from tension face.	3·14	For 3·28 feet at each end stirrups as above at 11-inch centres.	0·756	Do.	107	9·76 13·49 15·96 23·40	14·42 19·93 23·60 34·58	0·039 0·051 0·053 0·096	— Fissure formed in beam began to open. New cracks appeared. Slipping of reinforcements commenced.

D.	Do.	Do.	Do.	Do.	Do.	Do.	Do.	129	7·29 11·01 12·25 20·93	10·66 16·26 18·08 30·90	0·0197 0·0236 0·0295 0·0650	— — Small cracks on tension face. All cracks widening.
E².	Do.	Do.	Round bar, 1·26 ins. diameter; 0·80 ins. clear from tension face, and round bar, 0·874 ins. diam., 0·80 in. clear from compression face.	1·00 0·48	Stirrups as above placed closely at ends and more openly at centre.	Ends. 0·57 Centre. 0·29	Do.	180	—	—	—	The cracks observed during the hardening under load were only visible to the magnifying glass after unloading. During the loading tests a maximum load of 7·6 tons or a moment of 11·22 foot-tons produced new fissures and extended the old ones into the beam.
F.	Do.	Do.	Two round bars, 0·878 ins. diameter, 0·80 in. clear from tension face.	0·98	Sheet iron stirrups of 1·575 ins. × 0·085 ins. for 3·28 ft. at each end.	0·975	Do.	210	—	—	—	Under a total load of 7·6 tons or a bending moment of 11·22 foot-tons a fissure on the tension side of the beam appeared, and under a total load of 14·73 tons or a bending moment of 21·75 foot-tons the beam failed by cracking all over, and by slipping of the reinforcement at one end.
G.	Do.	Do.	Two round bars, 0·630 ins. diameter.	0·50	Do.	Do.	Do.	210	—	—	—	Under a total load of 5·12 tons or a bending moment of 7·55 foot-tons two cracks on the tension side appeared, and under a load of 10·08 tons or a bending moment of 14·88 foot-tons the beam failed by cracking all over.

TABLE No. 12.—(*continued*).

1.	2.	3.	4.		5.		6.	7.	8.	9.	10.	11.
Reference Letter.	Span.	Depth × Breadth.	Longitudinal Reinforcement.		Transverse Reinforcement.		Composition of Concrete.	Age at Test.	Results of Test.			
			—	Per cent.	—]	Total Section per foot of Length.			Total Loads Applied.	Bending Moment.	Deflection in Length of 6·56 Feet at Middle.	Remarks.
H.	Feet. 9·84	Inches. 15·75 × 7·87	Tension reinforcement, four round bars, 0·906 ins. diameter, placed two over two with 0·60 in. clear between rods and 0·80 in. clear between rods and tension face.	2·07	Sheet stirrups 0·787 ins. × 0·47 ins., 3·94 ins. centre to centre.	0·48%	Six cwts. Portland cement; 14·35 cub. ft. of sand; 28·70 cub. ft. of gravel; water 8·2 per cent. of weight of dry material.	Days. 95	Tons. 5·25 6·82 16·14	Foot-Tons. 12·92 16·79 39·68	Inches. 0·0390 0·0618 > 0·8110	— Vertical crack occurred. Concrete was crushed near points of application of load. Beam broke by bending without production of oblique fissures.
I.	Do.	Do.	Do.	Do.	Rods 0·197 ins. diameter, 3·937 ins. centre to centre.	0·39%	Do.	96	8·22 10·38 23·70	13·48 17·05 38·90	0·0531 0·0748 0·2815	— Vertical crack appeared. Perished by flexion. The concrete in compression crushed and splintered off. There were a number of vertical and oblique cracks.

	Do.	Do.	In addition to tension reinforcements as in I. two rods, 0·906 ins. diameter, were placed 0·80 in. clear of compression face of the beam.	2·07 1·04	As in Beam H.	0·48%	Do.	97	8·22 9·76 26·82	12·15 14·43 39·64	0·0381 0·0512 0·3083	— Vertical crack appeared. Failed by crushing of concrete.
K^3.	Do.	Do.	Do.	Do.	As in Beam I.	0·39%	Do.	98	8·22 9·76 28·40 5·12 6·67 15·35	12·15 14·43 41·86 7·55 9·83 22·68	0·0370 0·0512 > 0·60 0·0689 0·0972 > 0·60	— Vertical crack appeared at tension face at point of application of load. Beam failed by flexion. — Vertical crack on tension face near middle of span. Failed by flexion and crushing of concrete.
L.	Do.	Do.	Do.	Do.	As in Beam H.	0·48%	Do.	99	3·56 9·76 11·32 26·82	5·26 14·43 16·71 39·65	0·0157 0·5350 0·0681 > 0·750	— — Vertical crack near middle of span on tension face. Beam perished by crushing and crumbling of concrete.
M.	Do.	Do.	Do.	Do.	As in Beam I.	0·39%	Do.	99	5·12 8·22 26·82	7·55 12·15 39·65	0·0232 0·0433 0·2539	— Vertical fissure from tension face near centre of span. Beam perished by crushing of concrete.

TABLE No. 12.—(*continued*).

1.	2.	3.	4.		5.		6.	7.	8.	9.	10.	11.
			Longitudinal Reinforcement.		Transverse Reinforcement.				Results of Test.			
Reference Letter.	Span.	Depth × Breadth.	—	Per cent.	—	Total Section per foot of Length.	Composition of Concrete.	Age at Test.	Total Loads Applied.	Bending Moment.	Deflection in Length of 6·56 Feet at Middle.	Remarks.
N.	Feet. 9·84	Inches. 15·75 × 7·87	In addition to tension reinforcement as in I. two rods, 0·906 ins. diameter, were placed 0·80 ins. clear of compression face of the beam.	2·07 1·04	Rods 0·157 ins. diameter at 2·52 ins. centres.	0·39%	Six cwts. Portland cement; 14·35 cub. ft. of sand; 28·70 cub. ft. of gravel; water 8·2 per cent. of weight of dry material.	Days. 104	Tons. 6·67 9·76 25·25	Foot-Tons. 9·84 14·43 37·35	Inches. 0·0389 0·0606 0·2736	— Vertical fissure, tension face, at point of application of load. Beam failed by shearing of concrete between support and point of application of load.
O.	Do.	Do.	Do.	Do.	Rods 0·236 ins. diameter, 5·51 ins. centres.	0·40%	Do.	103	8·21 9·76 29·90	12·14 14·43 44·13	0·039 0·0492 0·5276	— Vertical crack on tension side near middle. Failed by crushing of concrete.
P.	Do.	Do.	Do.	Do.	Without vertical reinforcement.	0%	Do.	99	8·21 9·76 28·35	12·14 14·43 41·90	0·0413 0·0523 0·5158	— Vertical cracks from tension side. Ruptured suddenly at this load by shearing of the concrete.

Q.	Do.	Do.	Do.	Do.	Rods 0·276 ins. diameter, 3·15 ins. centres.	0·96%	Do	109	6·67 8·21 28·08	9·84 12·14 41·46	0·0374 0·0484 > 0·897	— Vertical crack from tension side at middle of span. Perished by crushing of concrete; oblique fissures very small.
R.	Do.	Do.	Do.	Do.	Rods 0·197 ins. diameter, 10·24 ins. centres horizontally, inclined 45% to horizontal.	0·21%	Do.	103	8·21 9·76 29·90	12·14 14·43 44·20	0·0385 0·0480 > 0·52	— Vertical fissure from tension face near middle of span. Perished by crushing of concrete at middle of span.
S.	Do.	Do.	Do.	Do.	Rods 0·276 ins. diameter, inclined 45° and spaced 8·66 ins. apart centres horizontally.	0·49%	Do.	104	8·21 9·76 29·60	12·14 14·43 43·70	0·0433 0·0551 > 0·8270	— Slightly inclined crack from tension face near point of application of load. Perished by crushing of concrete at middle of span.
T.	Do.	Do.	Do.	Do.	Rods 0·276 ins. diameter, inclined 45°, and spaced 6·3 ins. apart centres horizontally.	0·68%	Do.	105	9·76 11·32 31·45	14·43 16·72 46·50	0·0496 0·0633 > 0·80	— Vertical crack from tension face near point of application of load. Perished by crushing of concrete at middle of span. The oblique cracks were capillary.
U.	Do.	Do.	As in Beam H.	2·07	As in Beam I.	0·33%	Do.	108	5·11 6·67 26·82	7·55 9·84 39·63	— — —	— Vertical crack from tension face near middle of span. Perished by crushing of concrete.

TABLE No. 12.—(*continued*).

1.	2.	3.	4.		5.		6.	7.	8.	9.	10.	11.
Reference Letter.	Span.	Depth × Breadth.	Longitudinal Reinforcement.		Transverse Reinforcement.		Composition of Concrete.	Age at Test.	Results of Test.			
			—	Per cent.	—	Total Section per Foot of Length.			Total Loads Applied.	Bending Moment.	Deflection in Length of 6·56 Feet at Middle.	Remarks.
	Feet.	Inches.				Inches.[2]		Days.	Tons.	Foot-Tons.	Inches.	
V.	9·51	7·87 × 5·906	Three rods, 0·472 ins. diameter, and two rods, 0·63 ins. diameter in a row at 0·63 in. between centres and tension face.	2·47	Nil.	—	8 cwts. Portland cement; 14·3 cub. ft. of calcareous sand; 28·7 cub. ft. of gravel; water 9·6 per cent. of weight of dry materials.	178	—	—	—	The beam hardened in air. At the end of 178 days it was subjected to a total load of 4·06 tons or a bending moment of 4·6 foot-tons without signs of cracking. Slabs sawn from the tension side of the beam and subjected to bending gave an average resistance to tension, calculated by the formula $R = \frac{3Pl}{2bh^2}$, of 436 lbs. per in.[2] The resistance to tension of a slab cut from the compression side of the beam was 513 lbs. per square inch.
W.	Do.	Do.	Do.	Do.	Do.	—	Do.	188	—	—	—	This beam hardened under water. At the end of 188 days it was subjected to a total load of 5·42 tons or a bending moment of 6·67 foot-tons, when partial failure at one end owing to slipping of reinforcements and to cracking of concrete took place. Slabs sawn from the tension and compression sides of the beam gave resistances to tension, calculated as above from bending experiments, of 315 and 411 lbs. per square inch respectively.

1. Beam C had an artificial crack or division plane formed on its tension side extending up to the neutral axis by two thin copper plates left in the mould.
2. Beam E.—After ten days' setting a uniformly distributed load of 1·53 tons was applied over the length of 11·8 feet, which was increased to 2·66 tons after twenty days' setting and allowed to remain for six months.
3. Beam K was tested with the four rods in tension till failure by flexion practically took place. The beam was then turned upside down and again tested.

The influence of the percentage of vertical reinforcement may be studied in beams *M*, *P* and *Q*.

The comparison of vertical stirrups with reinforcements inclined at 45 degrees may be made in beams *Q*, *R*, *S* and *T*.

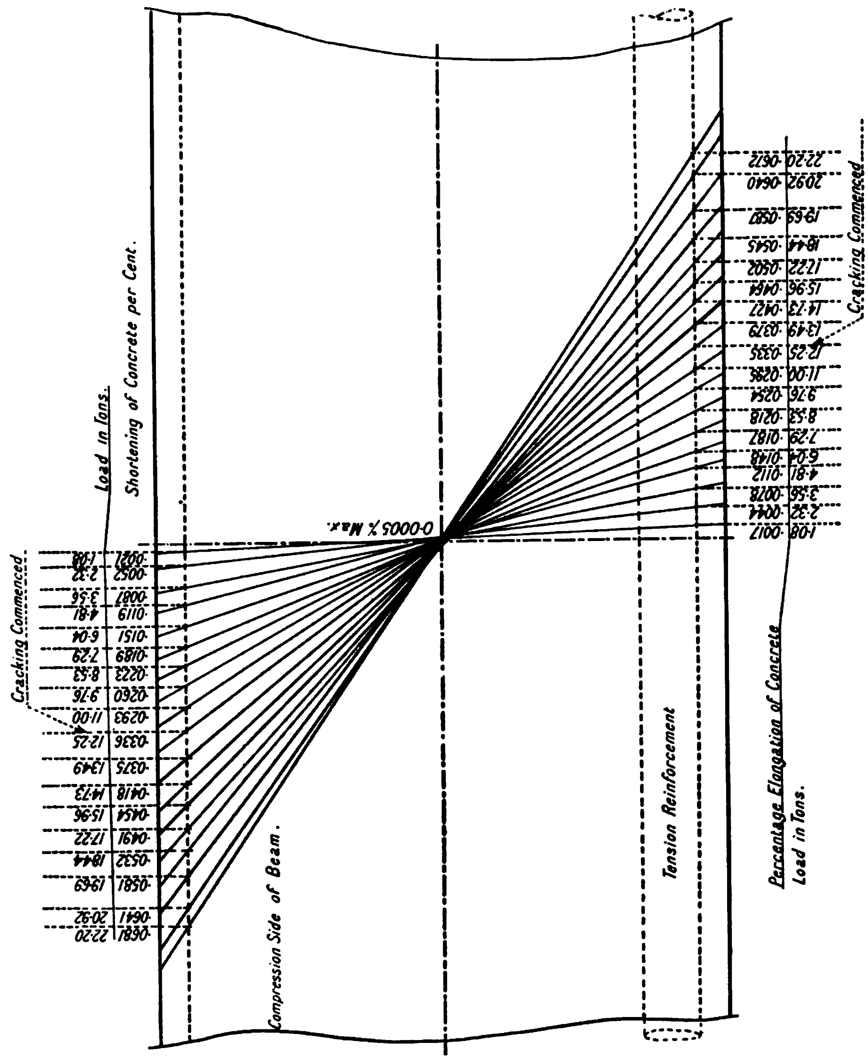


FIG. 11.

The utility of returning the reinforcements at the extremities was studied in beams *I*, *U*, *V* and *W*.

Beams *V* and *W* were also intended to show the condition of the concrete in a reinforced concrete beam after it had been stressed.

Table No. 13 gives an indication of the quality of the cement used in the various beams. In every case a good result was obtained in the soundness tests.

TABLE No. 13.

—	Time of Setting in Humid Air.		Average Tensile Strength of Neat Cement (lbs. per square inch).		
	Commencement.	Termination.	7 days.	28 days.	84 days.
Beams *A*—*G* . . .	20 minutes	4½ hours	370	518	687
Beams *H*—*U* . .	3 hours	5¼ hours	549	653	646
Beams *V* and *W* . .	1¾ hours	6½ hours	469	550	654

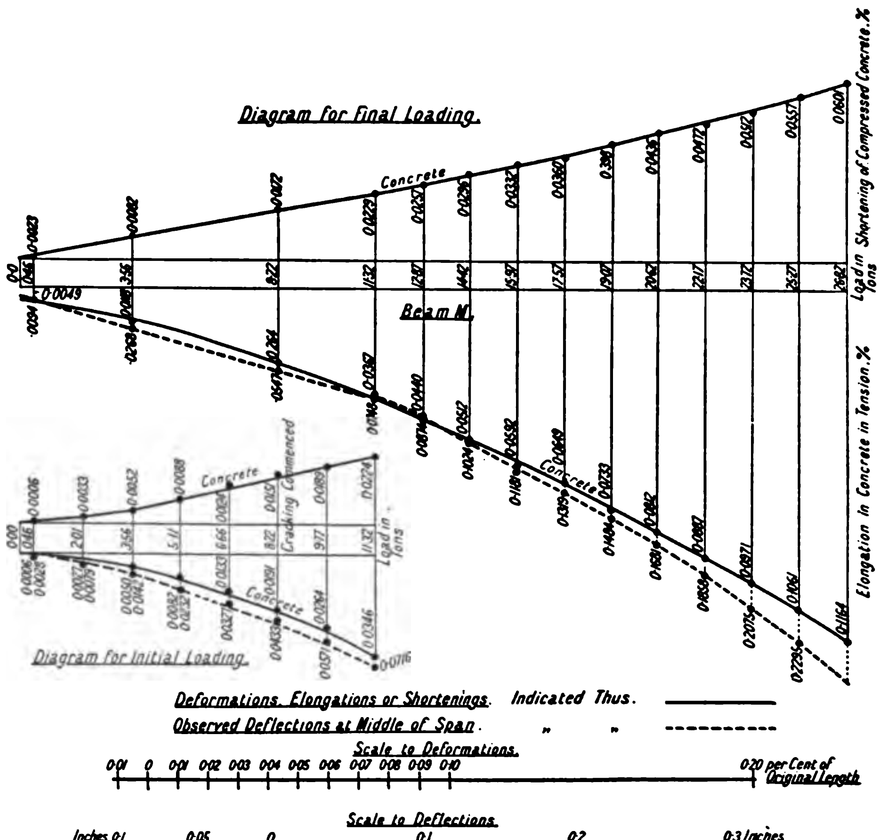


FIG. 12.

The steel used was mild steel, having a practical limit of elasticity varying with the diameter of the rod of from 15 to 20 tons per square inch, a breaking strength of from 23 to 28 tons per square inch, and an elongation of from 17 per cent. to 34 per cent. on a length of 7·87 inches.

In addition to the information given in Table No. 12, the actual strains on the tension and compression faces and at the neutral line were measured and also the variations in length due to the setting of the concrete in many of the beams. These are fully recorded in Chapter IV. of the French edition of the Report.

In the case of beam *M*, the extension and shortening of the concrete on the tension and compression faces respectively at the level of the reinforcements

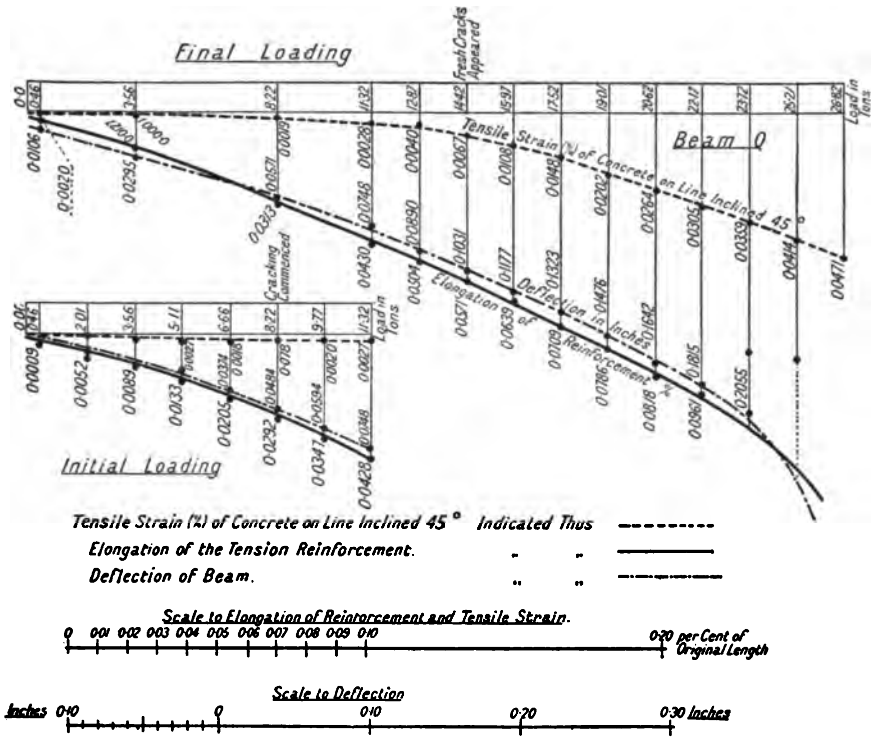


FIG. 13.

measured on a length of 3·28 feet, where the bending moment was uniform, together with the observed deflections at the middle of the span, are given in Fig. 12.

In the case of beam *Q*, the deflection at mid span, the elongation of the tension reinforcements measured on a length of 19·68 inches, and the elongation on a line inclined at 45° to the length of the beam are graphically represented in Fig. 13.

The elongations on a line inclined at 45° were measured on a length of 19·68 inches and are stated as percentage alterations of original length, and are plotted in the curve in Fig. 13 marked "Tensile Strain (%) of Concrete on Line inclined 45°." [1]

[1] In the French edition this curve was marked simply "Shear Strain." Monsieur Mesnager has pointed out to me that this is an error and that the actual shear strains are double the values given by the curve. That is clearly so, since in an elastic material subject to a pure shear the shear strains are each very approximately double the tensile strains measured on a line at 45° to the directions of the shears. Consequently the shear strain, measured in radians, has values represented by the figures given divided by 50.—N.M.

10. Bending of Flat Slabs.

Thirteen slabs, each 3·94 inches deep and 19·68 inches wide, reinforced with four rods at 4·72-inch centres and placed in the slab with 0·79 inches clear between the rods and the lower face of the slab, were tested to destruction in order to compare the resistances obtained under the various conditions described in Table No. 14, in which a *résumé* of the results is also given.

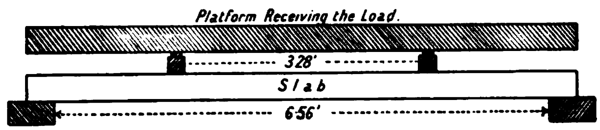


FIG. 14.

The sand used weighed 99 lbs. per cubic foot, of which

27·4	per cent.	was from	0·197	inches diameter	to	0·079 inches diameter,
59·6	,,	,,	0·079	,,	,,	0·0197 ,, ,,
13·0	,,	,,	0·0197	,,	,,	0·0 ,, ,,

and contained 6·2 per cent. by weight of moisture.

The gravel used weighed 89·1 lbs. per cubic foot, and

10·4	per cent.	was from	0·984	inches	to	0·787 inches diameter.
75·6	,,	,,	0·787	,,		0·394 ,, ,,
14·0	,,	,,	0·394	,,		0·197 ,, ,,

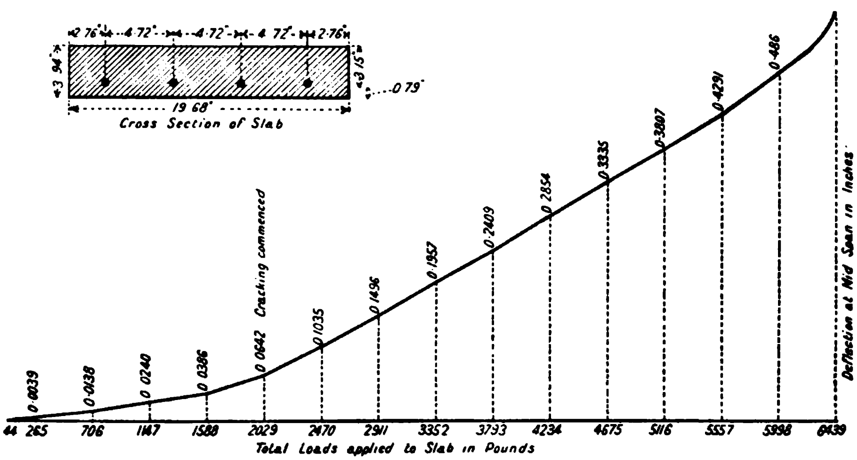


FIG. 15.

The gravel contained 6·8 per cent. of its weight of sand and 3·8 per cent. of its weight of water.

The cement used was of medium setting time, setting in 4¼ hours, and giving a resistence to tension of 480 lbs. per square inch after seven days and 580 lbs. per square inch after twenty-eight days. Slab No. 14 was made of a variety of lime setting in eighteen hours and giving a resistance to tension of 140 lbs. per square inch after seven days, 207 lbs. per square inch after fourteen days, and 304 lbs. per square inch after eighty-four days.

TABLE No. 14.

1.	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.	12.
	Composition of the Concrete.				Reinforcement.		Results of Tests.				
Slab Number.	Aggregate.	Sand.	Gravel.	Water per cent. Weight of Dry Materials.	Diameter of Bars.	Percentage.	External Applied Load.	Corresponding Bending Moment.	Observed Deflection at Middle of Span.	Age at Test.	Remarks.
No. 1.	Cwts. 6	Cubic feet. 14·35	Cubic feet. 28·70	8·2°/o	Inches. 0·394	0·63°/o	Tons. 0·906 1·102 2·875 3·071	Foot-Tons. 0·743 0·904 2·360 2·520	Inches. 0·0433 0·0709 0·4450 —	2 months.	— Cracking commenced. — Rupture took place immediately on the application of this load.
No. 2.	Do.	Do.	Do.	10·8°/o	Do.	Do.	0·512 0·709 2·482 2·680	0·420 0·582 2·040 2·200	0·0315 0·0594 0·4134 —	Do.	— Cracking commenced. — Slow yielding, leading to rupture 15 minutes after application of this load.
No. 3.	Do.	Do.	Do.	8·2°/o	Do.	Do.	0·906 1·102 2·678 2·875	0·743 0·904 2·198 2·360	0·0492 0·0807 0·4330 —	Do.	— Cracking commenced. — Slow yielding, leading to rupture 20 minutes after the application of this load.
No. 4.	Do.	7·18	35·87	8·1°/o	Do.	Do.	0·906 1·102 2·678 2·875	0·743 0·904 2·198 2·360	0·0460 0·0830 0·4570 —	Do.	— Cracking commenced. — Ruptured immediately on the application of the load.

TABLE No. 14.—(*continued*).

1.	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.	12.
	Composition of the Concrete.				Reinforcement.		Results of Tests.				
Slab Number.	Aggregate.	Sand.	Gravel.	Water per cent. Weight of Dry Materials.	Diameter of Bars.	Percentage.	External Applied Load.	Corresponding Bending Moment.	Observed Deflection at Middle of Span.	Age at Test.	Remarks.
No. 5.	Cwts. 6	Cubic feet. 21·53	Cubic feet. 21·53	8·3%	Inches. 0·394	0·63%	Tons. 0·709 0·906 2·678 2·875	Foot-Tons. 0·582 0·743 2·198 2·360	Inches. 0·0385 0·0641 0·4843 —	Do.	— Cracking commenced. — Ruptured a few minutes after the application of the load.
No. 6.	Do.	14·35 and 0·215 cubic feet of powdered clay.	28·70	10%	Do.	Do.	0·512 0·709 2·482 2·875	0·420 0·582 2·040 2·360	0·0464 0·0838 0·4900 > 2·75	Do.	— Cracking commenced. — Ruptured immediately after the application of the load.
No. 7.	8	14·35	Do.	9%	Do.	Do.	1·102 1·300 2·678 2·875	0·904 1·066 2·198 2·360	0·0444 0·0885 0·4134 —	Do.	— Cracking commenced. — Ruptured 1 minute after the application of the load.
No. 8.	5	Do.	Do.	8·2%	Do.	Do.	0·709 0·906 2·482 2·678	0·582 0·743 2·040 2·198	0·0390 0·0689 0·4016 > 0·7	Do.	— Cracking commenced. — Ruptured after about 10 minutes application of the load.

No. 9.	6	Do.	Do.	Do.	0·591	1·41%	0·906 1·102 5·435 5·825	0·743 0·904 4·460 4·788	0·0381 0·0610 0·6457 > 1·20	Do.	— Cracking commenced. — Ruptured after 10 minutes' application of the load.
No. 10.	Do.	Do.	Do.	Do.	0·197	0·16%	0·512 0·709 0·906	0·420 0·582 0·743	0·0248 0·0972 —	Do.	— Cracking commenced. Failed immediately on application of load.
No. 11.	Do.	Do.	Do.	Do.	0·394	0·63%	0·807 0·906 2·482 2·678	0·662 0·743 2·040 2·198	0·0709 0·0952 0·0472 —	15 days.	— Cracking commenced. — Ruptured immediately on loading.
No. 12.	Do.	Do	Do.	Do.	Do.	Do.	0·709 0·906 2·482 2·678	0·582 0·743 2·040 2·198	0·0570 0·0944 0·6044 —	8 days.	— Cracking commenced. — Ruptured immediately on loading.
No. 13.	6 cwts. hydraulic lime.	Do.	Do.	10·5%	Do.	Do.	0·512 0·709 2·284 2·482	0·420 0·582 1·875 2·040	0·0378 0·0846 0·5330 —	2 months.	— Cracking commenced. — Slow rupture after 6 minutes' action of the load.

NOTES.—The concrete in Slab No. 2 was simply poured into the mould. In the other cases it was rammed in beds.

Rupture occurred in each case owing to failure of the reinforcements. No slipping of the reinforcements or shearing at the ends took place. The ends of the reinforcements were simply cut square, with the exception of those in Slab No. 3, which were split and turned slightly out.

The reinforcements of 0·197 inches diameter were of iron having a limit of elasticity of 17 tons per square inch, a breaking strength of 25 tons per square inch, and an elongation of 14 per cent. on 7·87 inches.

The reinforcements of 0·394 inches and 0·591 inches diameter were of mild steel having a limit of elasticity of 17 tons per square inch, a breaking strength of 25½ tons per square inch and an elongation of 26 per cent., with a modulus of elasticity of 14,200 tons per square inch.

The slabs were left in the open air from the date of manufacture till the date of the test; and the loads were applied as indicated in Fig. 14.

The load in the case of slab No. 11 was applied in twenty minutes; in the other cases the time varied from fifty-five minutes to 2½ hours.

From zero load up to the first load given in the table, the deflection in each case follows a straight-line law, and is directly proportional to the load applied. For loads beyond the first load given, the deflection continues generally to obey a straight-line law, but the rate of increase of the deflection is much greater than up to that load. The results from all the slabs are very similar, and Fig. 15 gives the deflection diagram for slab No. 5, which is typical of the others.

11. Experiments on Ribbed Slabs.

Two floors were constructed each of span about 9 feet 6 inches, one 4 feet wide, the other 6 feet 6 inches, and each stiffened by one central rib. The floors were 2·36 inches deep and the ribs had 7·50 inches of additional depth, and were 6 inches wide.

In the case of the floor 6 feet 6 inches wide the rib had tension reinforcement only consisting of four rods, placed two vertically above two, each of 0·87 inch diameter with 0·5 inch clear between the rods and 0·8 inch clear between the rod and the lower face of beam, and having sheet-iron stirrups of 1·575 inches × 0·085 inch at about 4½ inches centres extending nearly to the upper surface of the floor. The floor was reinforced in the middle of its depth by rods of 0·394 inch diameter spaced at about 4½ inches centres; the ribs, very slightly increased in depth, were returned along the ends of the slabs to form a continuous support for the floor. The test load was applied along a width of 9 inches centrally over the rib, and was uniformly distributed in the direction of the length except for two short gaps at the centre, where strain-recording instruments were placed. A maximum load of 20·2 tons was placed on the slab a little less than six months after manufacture. At this load a noticeable crack extended across the tension side of slab and rib. Measurements of the deflection were not made, but the actual strains in the structure were carefully measured. These are fully recorded in the French edition, p. 240.

In the case of the floor 4 feet wide, the rib had tension reinforcement consisting of four rods, each 0·787 inch diameter, placed two vertically above two, spaced and with sheet stirrups as in the other floor. The method of reinforcing and supporting the floor during the test was exactly as in the floor previously described, but the load in this case was uniformly spread over the whole floor. A total load of 15·26 tons was applied, corresponding to a load of 7·87 cwts. per square foot of total surface, and remained in position for twenty-three hours. After eighteen hours' application of the load no further straining was observed and no cracks of any kind were observed, and on removing the load the strains almost entirely disappeared.

The measured strains are given in the French edition, p. 238, and a diagram graphically representing these strains is reproduced in Fig. 16.

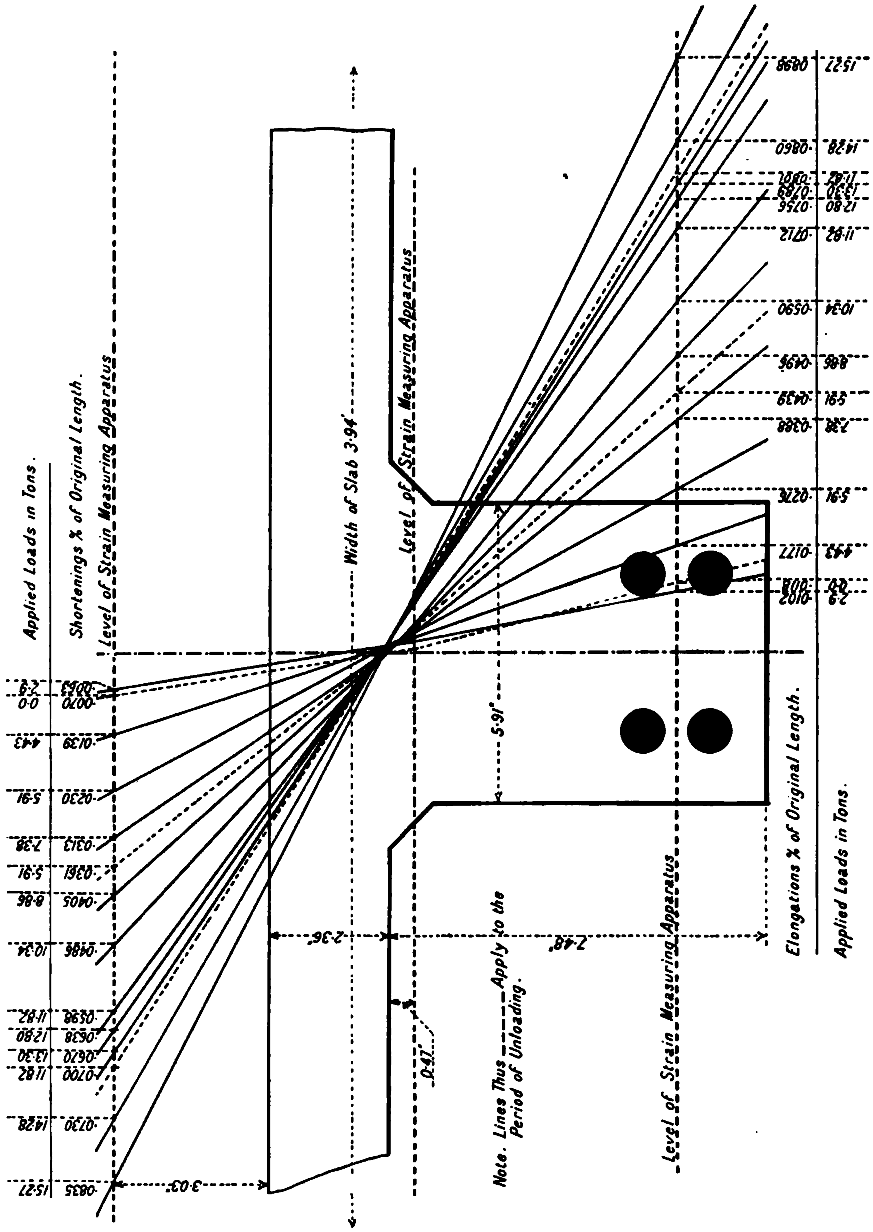


FIG. 16.

The materials employed were similar in character to those used in the manufacture of the experimental beams.

12. Tests of Columns and Prisms in Compression.

1. 9 *Columns, each* 16·4 *feet long.*

1. Nine columns, each 16·4 feet long, seven 15·75 inches square and two 9·84 inches square, were tested to destruction. Fig. 16A indicates the typical cross section of the columns, particulars of which are given in Table No. 16 (pp. 50—51) and Table No. 17 (p. 49) together with a *résumé* of the results obtained.

Each column was reinforced with four round bars placed one at each corner, so as to leave about 0·9 inch clear between each face of the column and the surface of the reinforcement. The reinforcing rods were united by sheet interties 0·118 inch thick, placed 19·68 inches apart in the column and 9·84 inches from either end.

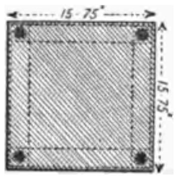


Fig. 16A.

The columns were manufactured under conditions pertaining in practice. Moulding was effected on the flat.

The concrete used consisted of 6 cwts. of Portland cement to 14·35 cubic feet of siliceous Seine sand passing through holes 0·197 inch in diameter, and 28·7 cubic feet of siliceous Seine gravel passing through holes 0·984 inch in diameter and retained on holes 0·197 inch in diameter. These materials were mixed with water, 8·2 per cent. of the weight of the mixture of dry materials. The quantities used were obtained by weighing, taking account of the humidity of the sand, about 5 per cent. by weight, and of the gravel, about 3·5 per cent. by weight as well as the quantity of sand contained in the gravel, 9 per cent. by weight of the latter. The granulometric composition of the sand and gravel was very similar to that of the materials described on p. 30.

The cement used commenced setting in air in four hours and setting was completed in 6¾ hours, and it stood the soundness tests satisfactorily. The average resistance to tension of briquettes of neat cement was 528 lbs. per square inch after seven days and 576 lbs. per square inch after twenty-eight days.

The quality of the reinforcement is indicated in Table No. 15.

Table No. 15.

—	—	Apparent Limit of Elasticity.	Breaking Load per square inch of Initial Section.	Elongation per cent. after Rupture ; on 7·87 inches.	Modulus of Elasticity.
		Tons per square inch.	Tons.		Tons per square inch.
Rod of 1·77 inches diameter.	Mild steel	14·5	22·8	32%	13,650
Rods 0·47 inch to 1·26 inches diameter.	Do.	17·9	25·5	28%	14,500
Sheets 0·118 inch thick.	Do.	16·9	22·6	26·5%	—
Do. . . .	Iron	16·5	21·6	15%	—

TABLE NO. 17.

No.	Longitudinal Reinforcements.		Transverse Interties.	Results of Tests.				
	Description.	Percentage of Total Section.		Applied Loads. Tons per square inch of Total Section.	Mean Shortening on Compression Face, as Percentage of Original Length.	Mean Elongation on Tension Face, as Percentage of Original Length.	Deflection of Column due to Buckling.	Remarks.
							Inches.	
8. 15·75 ins. × 15·75 ins.	4 rods each, 1·772 ins. diameter.	3·97	Sheet interties, 2·756 ins. × 0·118 in.	0·180	0·0130	0·0015	0·0362	Age of concrete, 11½ months.
				0·585	0·0490	0·0065	0·1642	—
				0·855	0·0761	0·0110	0·2626	Cracks appeared in compression surface.
				0·967	—	—	—	Rupture by crushing on an oblique plane at one end.
9. Do.	4 rods each, 0·630 in. diameter.	0·50	Sheet interties, 1·575 ins. × 0·118 in.	0·180	0·0177	0·0025	0·0472	Age of concrete, 12 months.
				0·585	0·0648	0·0110	0·1764	—
				0·855	0·1145	0·0250	0·3150	Cracking commenced.
				0·959	—	—	—	Rupture took place by the cracking of the column at one end, and the splitting out of a part of the concrete.

NOTE.—Columns 8 and 9, instead of being axially loaded, were loaded excentrically, the centre of loading being 3·94 inches—*i.e.*, one-quarter of the width, from one face.

Table No. 16.

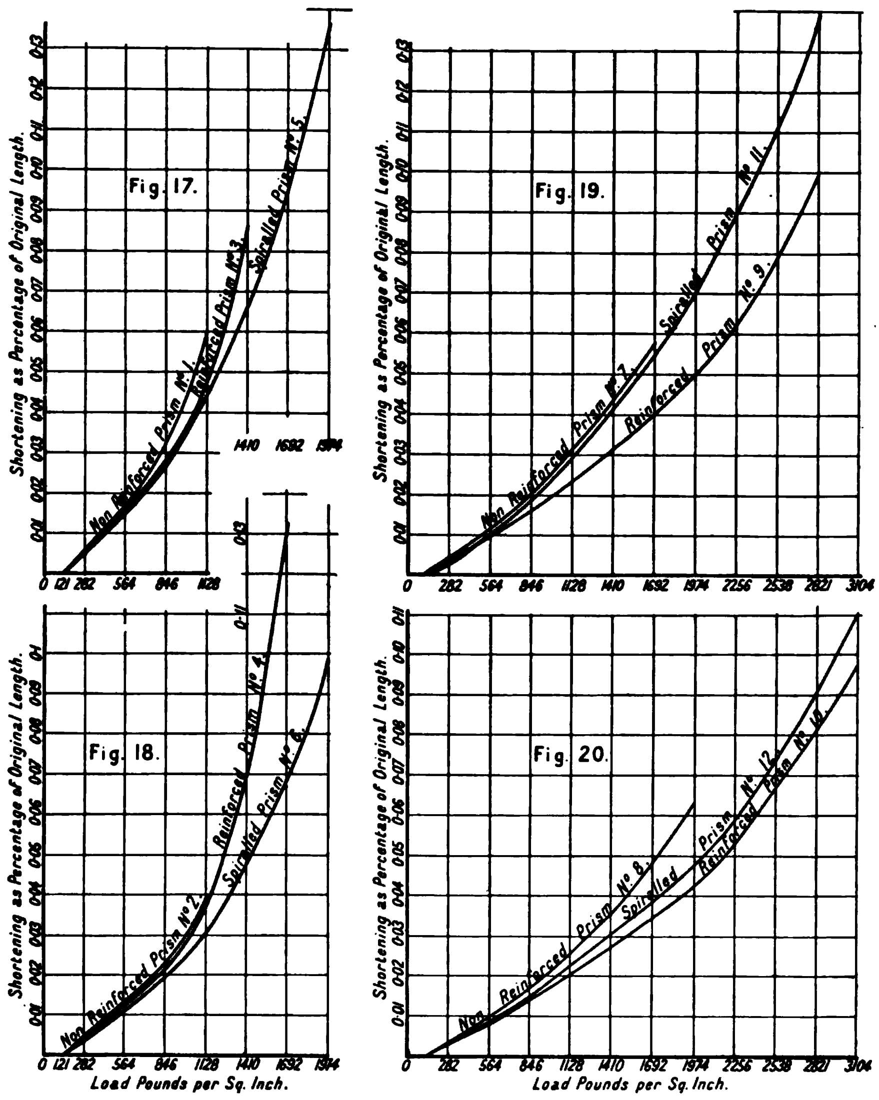
1.	2.	3.	4.	5.	6.	7.	8.
	Longitudinal Reinforcement.			Results of Tests.			
No	—	Percentage of Total Section.	Transverse Interties.	Applied Loads. Tons per inch² of Total Section.	Mean Shortening of Column per cent. of Original Length.	Quotient. Transverse Swelling between Two Interties ÷ Mean Shortening of Column.	Remarks.
1. 15·75 ins. × 15·75 ins.	4 rods, each 1·772 ins. diameter.	3·97	Sheet interties, 2·756 ins. × 0·118 in.	0·585	0·0220	0·37	Age at test, 10 months.
				0·989	0·0410	0·64	—
				1·257	0·0600	0·67	Longitudinal fissures appeared at one end and concrete commenced to flake off.
				1·588	—	—	Column ruptured sharply, following an oblique plane between two interties at one end immediately on application of load.
2. Do.	4 rods, each 1·26 ins. diameter.	2·01	Sheet interties, 2·165 ins. × 0·118 in.	0·675	0·0273	0·46	Age at test, 8 months.
				1·485	0·0825	0·69	Crushing of concrete at one end on face uppermost during manufacture.
				1·588	—	—	The above crushing accentuated no other damage to column.
3. Do.	4 rods, each 0·906 in. diameter.	1·04	Sheet interties, 1·772 ins. × 0·118 in.	0·585	0·0248	0·53	Age 8½ months.
				0·989	0·0454	0·98	—
				1·588	0·1028	0·92	—
				0·044	0·0267	1·48	The splitting of small pieces of concrete at the heads of the testing machine was the only observed damage.

4. Do.	4 rods, each 0·630 in. diameter.	0·50	Sheet interties, 1·575 ins. × 0·118 in.	0·585	0·0243	1·10	Age of concrete, 11½ months.
				0·989	0·0447	1·00	Small crack at one end in contact with the testing machine.
				0·044	0·0028	8·07	—
				0·989	0·0454	1·00	—
				1·588	—	—	Rupture 9 minutes after application of load owing to splitting along an oblique plane between two interties at one end.
5. Do.	4 rods, each 0·472 in. diameter.	0·28	Sheet interties, 1·378 ins. × 0·118 in.	0·585	0·0261	0·71	Age 10 months.
				0·854	—	—	Crack at one of the extremities which displaced the measuring apparatus.
				1·600	—	—	Rupture one minute after load was applied, following an oblique plane and breaking an intertie.
6. 9·84 ins. × 9·84 ins.	4 rods, each 1·102 ins. diameter.	3·94	Sheet interties, 1·969 ins. × 0·118 in.	0·585	0·0210	0·72	Age of the concrete, 10 months.
				1·128	0·0466	1·03	Cracks in the concrete at one extremity.
				1·256	0·0562	0·96	—
				1·482	—	—	Sudden rupture of the concrete following an oblique plane at one end.
7. Do.	4 rods, each 0·394 in. diameter.	0·50	Sheet interties, 1·378 ins. × 0·118 in.	0·585	0·0240	0·86	Age of the concrete, 11½ months.
				1·128	0·0524	1·13	—
				0·184	0·0100	2·51	—
				1·128	0·0536	1·13	—
				1·394	0·0692	1·19	—
				1·693	—	—	Sudden rupture following an oblique plane at one end.

NOTE.—All the above columns were placed horizontally, and the load uniformly distributed over the end of the column. The concrete covering the ends of the longitudinal reinforcements, which were about 0·3 inch shorter than the column, was removed so that stress was not directly communicated to them except in the case of Column No. 7, where the concrete was left in position.

2. 12 *prisms* 7·87″ × 7·87″ × 3·28 *feet long.*

These prisms were constructed of 6 cwts. Portland cement, 14·35 cubic feet Seine sand passing through 0·197 inch diameter holes, and 28·7 cubic feet Seine gravel passing through 0·984 inch diameter holes. In six of the prisms the concrete was



mixed with water amounting to 11·2 per cent. of the weight of the dry materials, and was consequently of soft consistency and was simply poured into the moulds. In the other six prisms 8·2 per cent. of water was used and the concrete rammed.

Four prisms were made without reinforcement; in two of these the concrete was simply poured, and in the other two rammed.

Four prisms were reinforced longitudinally, each by four rods 0·709 inch diameter, placed at 0·787 inch from each face and united by three sheet interties 1·575 inches × 0·118 inch thick, the extreme ties being placed about $6\frac{1}{2}$ inches from the ends. The longitudinal reinforcement amounted to 2·54 per cent. of the total area, and the interties, reduced to an equivalent rod of the same volume, 0·65 per cent., or together to a total of 3·19 per cent.

Four prisms were spiralled with a wire 0·236 inch diameter at 0·79 inch pitch, of a mean diameter of 7·24 inches. In addition there were six longitudinal rods, each 0·354 inch diameter, placed inside the spiralling. An equivalent rod of the same volume as the spiralling would have an area 2·04 per cent. of the total area of the prism, and adding 0·95 per cent. as the percentage of the longitudinal reinforcement, the total reinforcement is 2·99 per cent., roughly the same as for the longitudinal bars. The spiralling was applied in sections comprising seven to

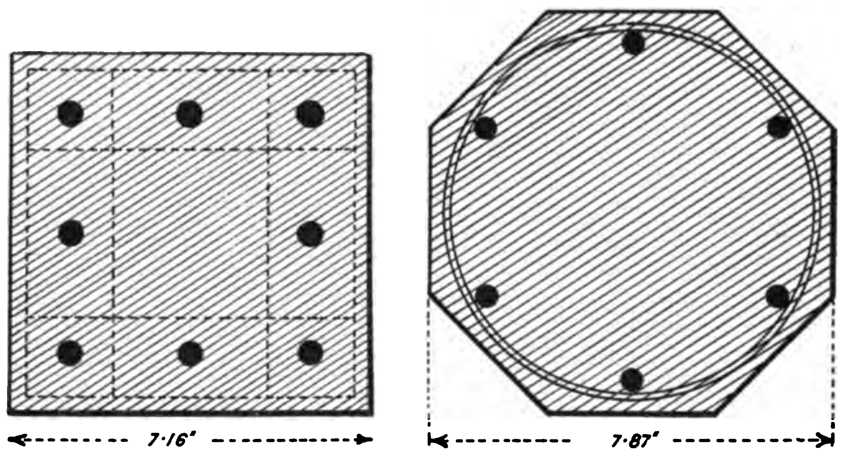


FIG. 21.

nine helices, the sections being simply superposed after having reduced the last spiral at each end to half pitch.

The reinforcement was of mild steel of ordinary quality. The rods of 0·354 inch diameter had an apparent limit of elasticity of 18 tons per inch^2, an ultimate strength of 26 tons per square inch, and a total elongation of 26 per cent. on 7·87 inches.

In each case the prisms were removed from the moulds forty-eight hours after manufacture, and kept moist by watering for three days. One of each type was then kept dry in the air, the other of that type was kept under a bed of sand frequently watered. A *résumé* of the results is given in Table No. 18, pp. 54—56.

In Fig. 17 the results of the tests of the prisms with the three types of reinforcement are exhibited graphically. The three prisms in this group were made from the same wet mixture, and similarly treated after moulding, viz., kept in a dry atmosphere till the date of the tests.

In Fig. 18 the results obtained are exhibited, when the prisms from a similar wet mixture are kept moist after moulding.

Figs. 19 and 20 represent graphically the corresponding results when a dry mixture is used and carefully rammed, and the prisms kept in the dry and kept moist respectively.

TABLE No. 18.

1.	2.	3.	4.	5.	6.	7.
			Results of Tests.			
No.	Total Weight of Prism 3 days after Manufacture.	Change in Weight in 28 days.	Load Applied to Total Section of Prism.	Load Applied to Section of Core.	Shortening per cent. of Original Length.	Remarks.
	Lbs.	Lbs.	Lbs. per inch².	Lbs. per inch².		
			Concrete Poured into Mould.			
1°. Non-reinforced Prism, kept dry.	206·2	Decrease 3·10	282	—	0·0068	Modulus of Elasticity: From 120 lbs. per inch² to 560 lbs. per inch²; 2·6 × 10^6 lbs. per inch².
			1,130	—	0·0596	From 120 lbs. per inch² to 850 lbs. per inch²; 2·3 × 10^6 lbs. per inch².
			1,295	—	—	Rupture in the body of prism 6 inches from one end.
2°. Do., but kept moist.	207·0	Increase 0·44	282	—	0·0054	Modulus of Elasticity: From 120 lbs. per inch² to 560 lbs. per inch²; 3·4 × 10^6 lbs. per inch².
			1,130	—	0·0379	From 120 lbs. per inch² to 850 lbs. per inch²; 3·3 × 10^6 lbs. per inch².
			1,382	—	—	Rupture in the body of the prism 9 inches from end.
3°. Prism, with longitudinal reinforcement, kept in the dry.	220·0	Decrease 3·75	282	—	0·0060	—
			1,411	—	0·0870	First fissures took place at 1,570 lbs. per square inch.
			1,733	—	—	Rupture took place at one-third of length.
4°. Prism, with longitudinal reinforcement, kept moist.	220·2	Increase 1·54	282	—	0·0046	—
			1,411	—	0·0703	Cracking commenced.
			1,794	—	—	Rupture at one-third of the length.

5°. Spiralled Prism, kept in the dry.	225·2	Decrease 4·85	282	—	0·0055	—
			1,693	—	0·0939	—
			1,975	—	0·1362	Cracking commenced, and before the last load given had been applied all the concrete covering the spirals had been burst off.
			3,630	5,455	—	Maximum load which the prism could carry. In carrying the load the deformation increased at the same time as the load somewhat diminished. The prism was withdrawn without rupture, when the shortening had reached 2 per cent.
6°. Do., kept moist.	226·3	Increase 0·88	282	—	0·0042	—
			1,693	—	0·0682	—
			1,975	—	0·0979	Cracks at the corners of the prism.
			3,870	5,820	—	Maximum load, in resisting which the deformation increased. Rupture of the prism by the breaking of two spirals took place at about one-third of the length when the stress was 4,690 lbs. per square inch of the section of the core and the shortening 2·9 per cent.

Concrete Rammed.

7°. Prism, not reinforced, kept in the dry.	213·8	Decrease 1·544	282	—	0·0042	Modulus of elasticity from 120 lbs. per inch^2 to 850 lbs. per inch^2 is $3·82 \times 10^6$ lbs. per inch^2.
			1,693	—	0·0572	—
			2,138	—	—	Rupture at the middle of length of the prism.
8°. Do., kept moist.	214·2	Increase 1·323	282	—	0·0032	Modulus of elasticity from 120 lbs. per inch^2 to 850 lbs. per inch^2 is $4·38 \times 10^6$ lbs. per inch^2.
			1,693	—	0·0468	—
			2,368	—	—	Rupture diagonally, near the extremity.
9°. Prism, reinforced longitudinally, kept in the dry.	226·3	Decrease 2·204	282	—	0·0038	—
			2,540	—	0·0766	Creaking noises, first cracks.
			3,307	—	—	Rupture towards one-third of length.

TABLE No. 18—(*continued*).

1.	2.	3.	4.	5.	6.	7.
			Results of Tests.			
No.	Total Weight of Prism 8 days after Manufacture.	Change in Weight in 28 days.	Load Applied to Total Section of Prism.	Load Applied to Section of Core.	Shortening per cent. of Original Length.	Remarks.
	Lbs.	Lbs.	Lbs. per inch.²	Lbs. per inch.²		
			Concrete Rammed—(continued)			
10°. Do., kept moist.		Increase 1·985	282	—	0·0028	—
			2,540	—	0·0649	Creaking noises, first cracks.
			3,365	—	—	Rupture about one-third of length.
11°. Spiralled Prism, kept in the dry.	226·8	Decrease 1·102	282	—	0·0034	—
			1,412	—	0·0414	—
			2,542	—	0·1096	First cracks.
			4,716	7,090	—	In maintaining this pressure the deformation increased whilst the pressure slightly diminished. The prism was withdrawn from the press when the shortening was 2 per cent.
12°. Spiralled Prism, kept moist.	230·1	Increase 1·742	282	—	0·0032	—
			1,412	—	0·0300	—
			3,105	—	0·1099	Creaking noises and first cracks.
			4,516	6,790	—	The core was completely bared. As in the preceding prism, it was withdrawn when the shortening was 2 per cent.

NOTE.—In each case the concrete was four weeks old when the test was made, and a sheet of cardboard was placed between the end of the column and the head of the press. For weight per cubic foot of the prisms at the end of 28 days see Table No. 19, p. 57.

3. 24 *columns from* 6·56 *feet long to* 13·12 *feet long.*

The columns manufactured had a section either square of 7·16 inches side or octagonal with the diameter of the inscribed circle 7·87 inches, as shown in Fig. 21 (p. 53).

The series of square columns comprised columns not reinforced, and columns reinforced longitudinally as shown on the cross section. The interties were placed at 13 inches centres and 6½ inches from the ends.

The series of octagonal columns included only columns spiralled with a metallic spiral rolled at varying pitches to an exterior diameter of 7·48 inches.

In each case the longitudinal reinforcements were about a quarter of an inch shorter than the columns, and they were not flush at either extremity.

All the columns were cast vertically and were manufactured by workmen under the conditions met with in actual works.

The columns were made in two series : (*a*) containing 7 cwts. Portland cement, 14·3 cubic feet Seine sand passing through a sieve with holes 0·197 inch diameter, and 28·7 cubic feet of Seine gravel passing through a hole of 0·984 inch diameter ; and (*b*) containing 10 cwts. Portland cement to those quantities of sand and gravel.

The water employed in gauging was 8·3 per cent. of the weight of the dry mixture in the former case, and 9 per cent. in the latter. The resulting concrete was plastic ; that is to say, that after being well worked up it moistened on striking with the flat of a spade. During the first fortnight of hardening the columns were watered slightly every two or three days.

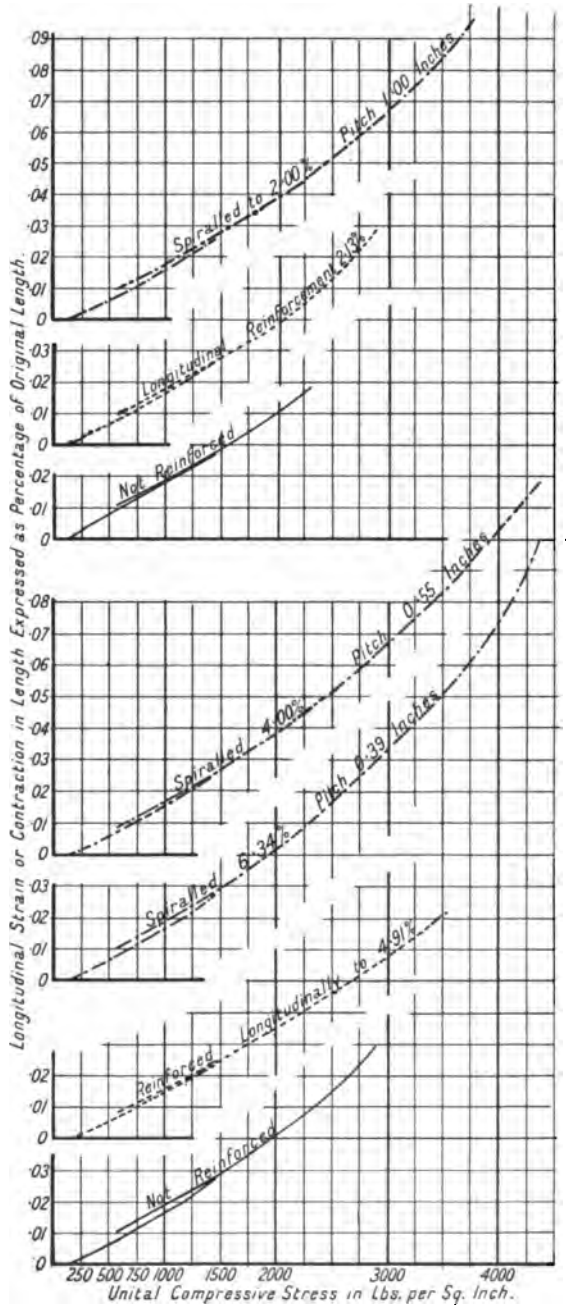
The columns were weighed in order to determine the approximate density of the concrete, and the weights per cubic foot are given in Table No. 19, and for purposes of comparison the weights per cubic foot of the prisms of Series 2 (p. 52) are added.

TABLE No. 19.

—	6 cwts. Portland Cement.				7 cwts. Portland Cement.	10 cwts. Portland Cement.
	Kept in Air.		Kept Moist.			
	Poured.	Rammed.	Poured.	Rammed.	Rammed and Kept in Air.	Rammed and Kept in Air.
Non-reinforced . .	143·5	150·2	146·9	152·5	149·0	148·0
Reinforced to total percentage of 2·13.	—	—	—	—	155·1	154·7
Reinforced to total percentage of 3·19.	153·0	158·6	156·9	162·2	—	—
Reinforced to total percentage of 4·94.	—	—	—	—	166·3	164·1

The extremities of each column were ground and rendered as exactly plane and parallel as possible, and the load was applied directly by the heads of the hydraulic press with the interposition of a sheet of cardboard 0·079 inch thick.

The reinforcements were of mild steel or of iron, of somewhat varying quality. Rods of mild steel of 0·304 inch diameter had an elastic limit of 14·6 tons per square inch, a breaking strength of 23·2 tons per square inch, and a percentage



Columns 13·12 Feet Long.
For Cross Sections see Fig. 21.

Columns 6·56 Feet Long.
For Cross Sections see Fig. 21.

7·0 cwts. Portland Cement, 14·3 C. Feet Sand, 28·7 C. Feet Gravel, Kept 5 Months in Air.

FIG. 22.

elongation on 7·87 inches of 25 per cent., whilst rods of 0·315 inch diameter had an elastic limit of 22·5 tons per square inch, and an ultimate strength of 29 tons per square inch, with an elongation of 21·5 per cent. in 7·87 inches, all the varieties of iron and steel used being between these values.

The cement used stood the soundness tests satisfactorily; setting commenced in 2 hours 40 minutes, and was complete in 5 hours 40 minutes. Briquettes of neat cement had a mean resistance to tension of 535 lbs. per inch^2 after seven days, and 578 lbs. per inch^2 after twenty-eight days.

Fig. 22 shows graphically the results obtained with columns 6·56 feet long and 13·12 feet long, manufactured in both cases with a concrete composed of 7 cwts. of Portland cement, 28·7 cubic feet of gravel, and 14·3 cubic feet of sand.

Fig. 23 shows the results obtained with columns of a similar length, and with a mixture of 10 cwts. of Portland cement to 28·7 cubic feet of gravel, and 14·3 cubic feet of sand.

Fig. 24 shows the results obtained with spiralled columns of various lengths manufactured with concretes containing 7 cwts. and 10 cwts. respectively of Portland cement, to 28·7 cubic feet of gravel and 14·3 cubic feet of sand.

The graphs in the above figures do not show the point at which the column actually collapsed, but simply the point at which the readings were discontinued owing to the virtual failure of the column.

The rupture of the columns having for the most part, and particularly for those of concrete with 10 cwts. of cement, occurred near the extremity corresponding to the upper end of the column during moulding, experiments were carried out to discover to what extent the resistance of the concrete might vary in columns moulded vertically without special precautions.

In the case of the columns 6·56 feet long, a prism 18·9 inches long was cut from the base and tested by crushing. In the case of the concrete containing 7 cwts. of cement, the prisms from the base showed an increase in crushing strength of 420 lbs. per square inch over the figure for the whole column. In the case of the concrete containing 10 cwts. cement the figure was 1,850 lbs. per square inch.

In the case of the columns 13·12 feet long, prisms were cut from the base and from a part of the column 3 feet from its upper end during moulding. The prisms from the base had 570 lbs. per inch^2 greater resistance to crushing than the upper prism, and about 1,360 lbs. per inch^2 greater resistance to crushing than the whole column for a concrete containing 7 cwts. Portland cement, whilst for a concrete containing 10 cwts. Portland cement these figures were 1,350 lbs. per inch^2 and 2,570 lbs. per inch^2 respectively.

13. Compression Tests of Spiralled Mortar and Concrete.

1. *Prisms in Cement Concrete, arranged by M. Considère.*

These prisms, of octagonal section of 29·45 square inches area and 1·64 feet long, were submitted to compression tests carried to rupture, the deformations being measured.

The results are given in Table No. 20 (p. 62).

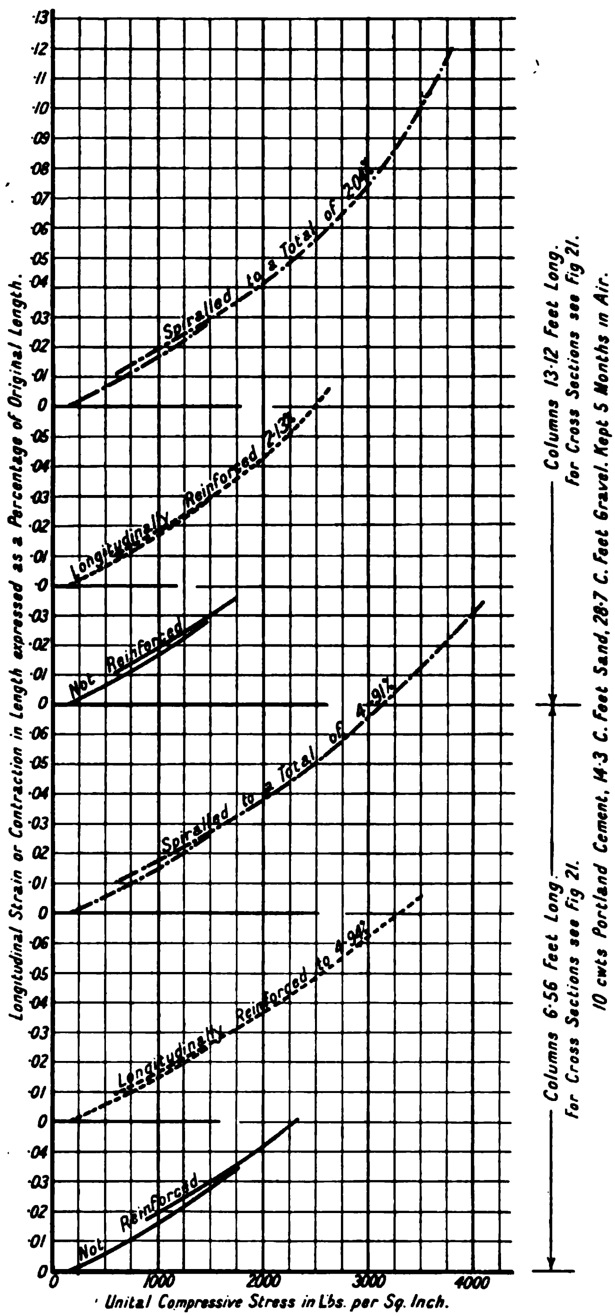

Longitudinal Strain or Contraction in Length expressed as a Percentage of Original Length.
Unital Compressive Stress in Lbs. per Sq. Inch.
0
1000
2000
3000
4000
Spiralled to a Total of 2·04%
Longitudinally Reinforced 2·13%
Not Reinforced
Spiralled to a Total of 4·91%
Longitudinally Reinforced to 4·94%
Not Reinforced
Columns 13·12 Feet Long.
For Cross Sections see Fig 21.
Columns 6·56 Feet Long.
For Cross Sections see Fig 21.
10 cwts Portland Cement, 14·3 C. Feet Sand, 28·7 C. Feet Gravel. Kept 5 Months in Air.

FIG. 23.

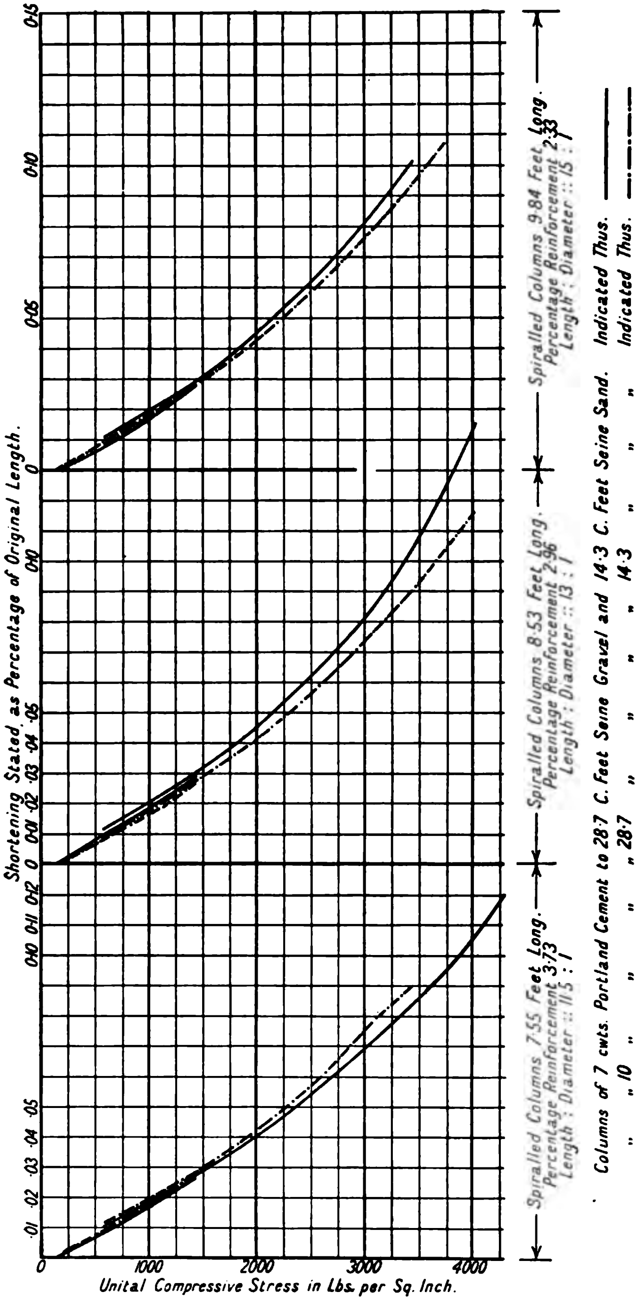

Shortening Stated as Percentage of Original Length.
Unital Compressive Stress in Lbs. per Sq. Inch.
0
1000
2000
3000
4000
Spiralled Columns 9·84 Feet Long.
Percentage Reinforcement 2·33
Length : Diameter :: 15 : 1
Spiralled Columns 8·53 Feet Long.
Percentage Reinforcement 2·96
Length : Diameter :: 13 : 1
Spiralled Columns 7·55 Feet Long.
Percentage Reinforcement 3·73
Length : Diameter :: 11·5 : 1
Columns of 7 cwts. Portland Cement to 28·7 C. Feet Seine Gravel and 14·3 C. Feet Seine Sand. Indicated Thus.
" " 10 " " " " 28·7 " " " " 14·3 " " " " Indicated Thus.

FIG. 24.

TABLE No. 20.

Description.	Load Applied to Spiralled Core, lbs. per inch².	Shortening per cent. of Original Length.	Remarks.
1. Prism not reinforced; age, 4 months. Stress reckoned on total section.	307	0·0128	Modulus of Elasticity: From 165 lbs. per inch² to 750 lbs. per inch² = $2{\cdot}18 \times 10^6$ lbs. per inch².
	1,050	0·0680	From 58 lbs. per inch² to 750 lbs. per inch² = $2{\cdot}11 \times 10^6$ lbs. per inch.²
	1,168	—	Rupture.
2. Prism reinforced with a rod 0·236 inch diameter, spiralled to a pitch of 1·18 inches, and to an exterior diameter of 5·51 inches. The sectional area of the reinforced core was 26·4 square inches.	344	0·0160	—
	1,502	0·0880	—
	65	0·0320	Permanent deformation.
	1,502	0·9400	—
	3,160	1·2740	Covering of spirals progressively detached.
	5,770	—	Prism completely deformed into S shape.
3. Prism reinforced with an iron rod 0·161 inch diameter, spiralled to a pitch of 0·59 inch, and to an exterior diameter of 5·51 inches. The sectional area of the reinforced core was 26·4 square inches; the pitch of the spirals was a little irregular, the pitch attaining ·67 inch in places.	344	0·016	—
	1,502	0·0740	—
	65	0·016	—
	1,502	0·0660	—
	3,610	0·6560	External covering of spiralling detached bit by bit.
	6,016	—	Maximum load of which the testing machine was capable. Column not broken.
4. Prism reinforced with a rod 0·236 inch diameter, spiralled to an external diameter of 5·51 inches, and to a pitch of 1·063 inches. Average section of core = 26·4 square inches.	344	0·0020	—
	1,502	0·0720	—
	65	0·0080	Permanent deformation.
	1,502	0·0768	—
	3,610	1·5640	External covering of spiralling detached bit by bit.
	5,635	—	Rupture.
5. Prism reinforced longitudinally with 8 rods, 0·315 inch diameter, giving sensibly the same percentage of metal as in the preceding spirally armoured prism. The stresses given are reckoned on the total section of 29·45 square inches.	1,530	—	Cracking commenced.
	1,835	—	Complete failure.
6. Non-reinforced prism of 29·45 square inches area. Stress reckoned on total area.	825	—	Complete failure.

NOTE.—In Prisms 2, 3 and 4 the transverse swelling of the spirals was measured, but no conclusive results were obtained.

2. *Six Prisms in Cement Concrete, as desired by M. Considère.*

Six octagonal prisms of about 15·5 square inches cross sectional area and 1·64 feet long were made.

Three of the prisms were of a mixture of 8 cwts. of Portland cement to 9·56 cubic feet of sand, screened to 0·197 inch diameter, and 28·7 cubic feet of gravel (0·197 inch to 0·984 inch diameter), and water amounting to 9·3 per cent. of the weight of the dry materials, was used as being necessary and sufficient to obtain a plastic consistency.

The other three were composed of a mixture of 10·7 cwts. of Portland cement to the above quantities of sand and gravel and 9·8 per cent. of water.

In the former series of three prisms, two were spiralled at a pitch of 0·55 inch for about one-quarter of the length from either end and 0·79 inch for the remaining portion in the middle. In the latter the pitch of the spirals for one-fifth of the length of the column at either end was 0·39 inch, whilst the middle portion was spiralled to 0·59 inch pitch. The spiralling was of iron wire 0·169 inch diameter, and the external diameter of the spirals was 3·94 inches. In addition, the prisms were reinforced each with eight rods of 0·169 inch diameter, placed in the interior of the spirals and in contact with them.

The two spiralled prisms, containing 8 cwts. Portland cement, were loaded by small increments to a stress of 1,300 lbs. per square inch of total section, and showing under that stress an average shortening of 0·0652 per cent. of their original length, with an average permanent deformation of 0·0244 per cent. of their original length when the load was removed. This permanent deformation increased slightly when the load was reapplied and removed.

From one of the prisms the crust of concrete covering the spiral reinforcement was removed down to the centre of the spiralling rods, and a compression was again applied and removed and the process repeated. At each repetition the stress was carried higher and the permanent deformation measured. At a stress of 2,366 lbs. per square inch of the core the shortening was 0·16 per cent. of the original length, whilst the permanent deformation was 0·074 per cent. At a stress of 9,270 lbs. per square inch the deformation was 2·4 per cent., and the permanent set amounted to 1·81 per cent. The maximum stress applied was 9,480 lbs. per square inch, but as the prism threatened to break, this load was removed before readings were taken. The spiralling was then stripped and the core tested. Crushing took place at 870 lbs. per square inch. The age of the concrete at test was about ten weeks.

In the other prism no further compression tests were made as above, but the spiralling was also removed and compression applied. Crushing took place at a stress of 1,670 lbs. per square inch.

The third prism of this mixture was made without reinforcement for purposes of comparison, and failed at 1,142 lbs. per square inch.

Two of the prisms of the mixture containing 10·7 cwts. Portland cement, one reinforced and the other not reinforced, were tested. The concrete was about four months old at test and had been immersed in fresh water for six weeks previous to it.

The reinforced prism at a stress of 6,370 lbs. per square inch of the spiralled core underwent a shortening of 0·582 per cent. of its original length, and on the load being removed showed a permanent deformation of 0·342 per cent. The prism ultimately perished by buckling under a stress of 12,700 lbs. per square inch of the section of the core.

The prism not reinforced failed at a stress of 2,235 lbs. per square inch of total section.

3. *Eight Cylinders of Mortar Spiralled for Compression Tests.*

These cylinders had a mean diameter of 3·3 inches and a length of 23·23 inches, but in consequence of the low modulus of elasticity of the mortar they showed a tendency to fail readily by buckling. Some were cut and tested in shorter lengths, but it has not been thought fit to give the detailed results owing to the impossibility of making comparisons.

The cylinders tested were spiralled, either with single spiral, with two concentric spirals or with three concentric spirals. When of the same length and when the columns failed by buckling the two concentric spirals only gave an increased ultimate strength of about 11 per cent. over the figure shown by the single spiral. When, however, the column failed more evenly by crushing, the two concentric spirals gave an increase of 24 per cent. of the crushing strength shown by the single spiral, and the three concentric spirals showed an increase of 49 per cent. over the crushing strength shown by the single spiral. The stress at which the external covering of concrete on the spirals began to flake off was not appreciably affected by the addition of the concentric spiralling. This stress for cylinders six months old was about 4,550 lbs. per square inch of the total section.

4. *Cylinders of Annular Section in Cement Mortar.*

Three cylinders, each 23·62 inches long, composed of mortar containing 6 cwts. of Portland cement, 11·8 cubic feet of Seine sand screened to 0·197 inch diameter and gauged with water amounting to 15·7 per cent. of the weight of the dry materials. The cylinders were kept in air for ten weeks after manufacture; they were then immersed in fresh water for six weeks, from which they were taken for testing purposes.

The particulars of the cylinders and the results of the tests are given in Table No. 21.

5. *Two Spiralled Prisms from the Experimental Bridge, built by M. Considère.*

These prisms were taken from the upper boom of the above bridge, which was of the bowstring type. They were of octagonal section, inscribed circle 9·84 inches diameter, giving a total cross section of 80·29 square inches and an area of spiralled core of 48·67 square inches.

The first prism tested was 41 inches long. Under a stress of 2,842 lbs. per square inch of the total section the shortening was 0·0856 per cent. of the original length, and under a stress of 5,684 lbs. per square inch of the total area or of 9,380 lbs. per square inch of the spiralled core was 0·2 per cent., whilst the outer covering of concrete on the spirals began to fall off. The maximum stress applied was 12,690 lbs. per square inch of the spiralled core, at which stress the bending of the prism rapidly increased. At the time of the test the prism was about two years old.

The other prism was 52 inches long and was tested by the application of a load at one-third of the diameter from one face. Up to a stress of 4,670 lbs. per square inch, reckoned as uniformly distributed over the whole section, no cracking was apparent, and the angle between the end faces of the column was 0·0087 radians. At a stress corresponding to 10,600 lbs. per square inch, if uniformly spread over the core, the angular deformation was 0·0113 radians, and the column failed by the shearing of the concrete of the core.

6. *Prism of Spiralled Concrete for Compression Test.*

This prism of square section of 10·24 inches side and 37 inches long was composed of fluid concrete poured without any ramming.

It was reinforced with a spiral of mild steel rod of 0·472 inch diameter

TABLE No. 21.

No.	External Diameter.	Internal Diameter.	Area.	Spiralling.	Results of Test. Load Applied, lbs. per square inch of Total Section.	Mean Shortening per cent. of Original Length.	Remarks.
	Inches.	Inches.	Inches.²				
1.	7·09	4·92	20·52	Iron wire 0·189 inch diameter in helical pitch of 0·787 inch. External diameter of spiralling 6·89 inches.	441 2,170 2,570	0·016 0·096 —	— The cylinder broke in consequence of the failure of the mortar inside the cylinder owing to want of support.
2.	7·09	4·53	23·34	Iron wire 0·189 inch diameter in helical pitch of 0·787 inch. External diameter of spiralling 6·69 inches.	2,300	—	Do.
3.	4·76	—	17·83	Not reinforced.	2,490	—	Maximum load supported.

wound to a mean diameter of 8·39 inches and to a pitch of 1·024 inches, giving a percentage of 4·3, and in addition in the longitudinal direction with seven bars of mild steel of 0·63 inch diameter arranged round the inside of the spiral and in contact with it, giving a percentage of about 2·1. The age of the concrete at the time of the test was about three months. The load was applied axially and the first fissures appeared at a stress of 4,040 lbs. per square inch of the total area, and the maximum stress supported was 10,200 lbs. per square inch of the spiralled core. At that stress the shortening was 2 per cent.

The spiralling was subsequently removed and the resistance to direct crushing of the non-reinforced concrete was found to be 1,124 lbs. per square inch, measured on a cylinder of 6·89 inches diameter and 18 inches long.

7. *Two Prisms Reinforced with Intertwining Spirals.*

These prisms were of the section shown in Fig. 25 and were 19·68 inches long.

The spirals were of mild steel wire 0·272 inch diameter, rolled on a cylinder of $5\frac{1}{2}$ inches diameter to a pitch of 1 inch. Alternate spirals were wound in opposite directions. Longitudinal reinforcements of mild steel 0·315 inch diameter were placed as shown.

The concrete consisted of 12 cwts. Portland cement to 14·35 cubic feet of sand sieved to 0·197 inch diameter and 28·7 cubic feet of gravel. In both cases the moulding was done on the flat. For prism No. 1 concrete of a plastic consistency well rammed was employed; for No. 2 a wetter concrete was used to fill the mould up to the level of the upper side of the spirals, of sufficient fluidity to fill the spirals without ramming between them, the mould was then filled with the mixture of materials quite dry and rammed till the moisture appeared on the surface. The prisms were kept in air for the eight months that elapsed between the time of moulding and the test.

In both cases at a stress of 4,550 lbs. per square inch of the total section of the prisms, or 7,110 lbs. per square inch of the section of the spiralled core, the concrete

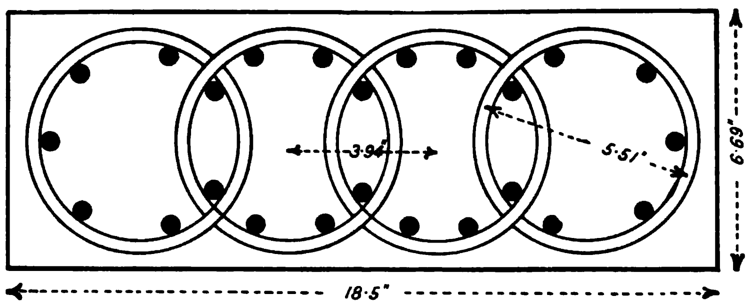


FIG. 25.

outside the spiralling began to flake off. The maximum load carried by prism No. 1 was 9,730 lbs. per square inch of the spiralled core, whilst the maximum stress supported by prism No. 2 was 10,930 lbs. per square inch of the spiralled core. In both cases the total shortening was 3·6 per cent. of the original length.

14. Tension Tests on Reinforced Members.

Two tension members were constructed of twenty-four wires of steel surrounded by a metallic spiral as indicated in Fig. 26, and encased for a length of 6·56 feet. One half of the length of each member was of neat cement, the other half of mortar.

In member No. 1 the diameter of the longitudinal wires was ·177 inch, giving a percentage reinforcement of 5·9. The wires were placed 0·47 inch centre to centre. The mild steel wire in the spiral had a diameter of 0·118 inch and a pitch of 1·18 inches. Ten wires were jointed in the member, the junction consisting of an overlap of 11·8 inches, the free ends being bound with soft iron wire. The junctions were spread over the length of the member. The reinforcing wires, which extended beyond the end of the concrete casing, were gathered together and fixed into a steel cage with spelter, there being a clear distance about 17 inches between the end of the concrete casing and the near face of the metallic anchor block. The concrete casing had a square section of 3·15 inches side.

Member No. 2 was of similar design, but was moulded under a tension of 8·9 tons per square inch of the area of the reinforcement, which was maintained up to the moment of the test. The longitudinal wires were 0·118 inch diameter, giving a percentage reinforcement of 3·4. The mild steel wire constituting the spiralling had a diameter of 0·079 inch and a pitch of about half an inch. Five of the wires were in two parts, the junctions being formed by means of a muff consisting of a slightly flattened steel tube 2 inches long, through which the ends of the wires were passed and allowed to project for lengths of 9 inches beyond the muff. The free ends of the wires were in this case moulded into a concrete head.

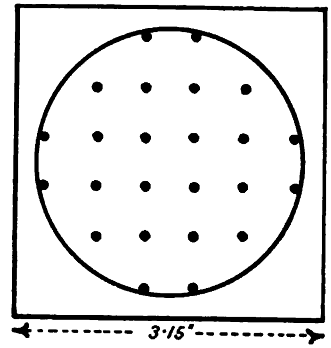


Fig. 26.

The longitudinal reinforcements consisted of a specially hard steel, which had a limit of elasticity in the neighbourhood of its breaking strength. The mean resistance to tension for wires of 0·177 inch diameter was 107 tons per square inch, and for wires of 0·118 inch diameter was 115 tons per square inch.

The cement was Portland cement of French manufacture. Its setting time was $4\frac{1}{2}$ hours. Its resistance to tension after seven days was 620 lbs. per square inch, and after twenty-eight days 734 lbs. per square inch. The mortar consisted of four parts of this cement to five parts by measure of fine siliceous sand passing through a sieve with holes 0·039 inch diameter and without much dust. The division between the mortar and the pure cement was maintained by a sheet-brass diaphragm 0·394 inch thick, with holes to keep the wires at the proper spacings.

Tension was applied to the member by means of the head of metal in one case and of concrete in the other.

Member No. 1 was tested after a hardening of one month. This member had several cracks in the part made of pure cement; none were visible in the part in mortar. In consequence of the difference of inclination of the reinforcing wires between the member and the fixed head the distribution of the load was necessarily very uneven, with the result that the wires began to break one after the other at a stress of 74·3 tons per square inch, calculated as being uniformly spread over the reinforcement. The first fissure in the mortar appeared at a stress of 10·7 tons per square inch of the reinforcement, assuming the load uniformly distributed.

Member No. 2 was tested after a hardening of three months. At the time of the test only two small cracks near the extremity in the pure cement were visible. The first crack in the pure cement occurred when a stress of 15·5 tons per square inch of the reinforcement had been applied, and the first fissure in the mortar when that stress was increased to 24 tons per square inch. The tension was not carried beyond 43 tons per square inch of the reinforcement. At that stress the mortar and cement were cracked at right angles to the reinforcement every 4 inches or so.

CHAPTER V

The Report and Draft Regulations Presented by the Commission

The Commission arranged its work in the following manner:—

1. It set out to define as precisely as possible the materials constituting reinforced concrete, the principal conditions they ought to fulfil, and the indispensable precautions to be taken in construction.
2. It studied the elementary properties of the materials, taken separately and also associated.
3. The physical and elastic properties thus determined were examined to find, if possible, a rational basis for and a scientific method of calculation.
4. The results obtained from these processes were compared with successful practice.

The Report passes in review the principal results obtained from the experimental work of the Commission.

Variations in Volume Resulting from the Setting of Cement.

It is known that, extending over some months, mortars and concretes exposed to air undergo contraction during setting, which is greater the higher the proportion of cement. This contraction leads to internal stresses, tension in the concrete and compression in the reinforcement. When this contraction is opposed by external connections, serious cracking frequently results.

The concrete employed in the tests was of the proportion most frequently employed in open-air work in France—viz., 6 cwts. Portland cement, 28·7 cubic feet of gravel and 14·4 cubic feet of sand, which, when rammed into the work, give about 36 cubic feet of concrete, or about $1\frac{1}{3}$ cubic yards.

Although the contraction varies somewhat with the length and section of the member, the Commission put it at 0·025 per cent. when it is exposed to the weather immediately after manufacture, and 0·020 per cent. when it is maintained humid for the first three weeks.

The Commission did not insist that the contraction should be taken account of, except in long members.

Thermal Variations of Volume.

No new experiments were made on this subject, but it appeared from the collected results of previous experiments that a coefficient of expansion per 100 degrees Centigrade of 0·0011, or 0·11 per cent. (0·00061 per 100 degrees Fahrenheit), might without sensible error be adopted, and that the coefficients of expansion of concrete and steel might be taken as equal.

From measurements made on an arched bridge at Chatelherault, it was demonstrated that reinforced concrete structures are only very slightly sensible to hourly variations of temperature, but that they assume the mean daily temperature.

Resistance and Deformation of Concrete Submitted to Tension.

When concrete, not reinforced, is submitted to simple tension, the tension increases proportionally to the elongation until rupture is produced for an elongation of at least, in general, 0·01 per cent.

When concrete, not reinforced, is submitted to bending, the part in tension has a certain ductility. From a limit which appears very near the maximum elongation when submitted to simple tension, the modulus of elasticity of the concrete in the tension part of the beam diminishes very markedly, and rupture is produced for an elongation which generally lies between 0·01 and 0·02 per cent.

Concrete properly prepared and reinforced becomes much more ductile still. From the experiments of the Commission it was established that the elongation of the concrete before rupture went up to 0·135 per cent., and it was observed that until an elongation between 0·01 and 0·02 per cent. was reached, the concrete had practically the same modulus of elasticity as concrete not reinforced. Beyond this limit the modulus of elasticity is sensibly zero; or, in other words, whilst the elongation increases, the tension of the concrete remains very nearly constant and in the neighbourhood of the resistance of concrete not reinforced.

Resistance to Crushing.

Table No. 22, pp. 70—73, contains a *résumé* of the results of the experiments made by the Commission and of those made by Professor Bach of Stuttgart.

In the first six divisions the particulars characterising each series of experiments are indicated. In division 7 is given the increase of resistance in lbs. per square inch of total section which the reinforcement has produced. The figure relative to each column was found by deducting from its resistance that of a test column not reinforced and made of the same concrete.

To permit of the comparison of the different types of reinforcement, a common measure has been devised for them as follows :—

Suppose the column to be reinforced with an equivalent bar having the same total volume as all the reinforcements of whatever kind in the column. Divide the total increase of resistance by the area of this equivalent bar. The quotient will represent the increase of resistance of the concrete produced by each square inch of reinforcement.

The figures given in Table No. 22 call for the following remarks :—

In Series A, in order to study the slipping of the reinforcements, the concrete covering the ends of the longitudinal bars was removed. As a result, the resistance to crushing has been almost exactly the same, although the percentage of metal varied from 0·28 to 3·97 per cent.

In Series B, prepared conformably to the programme of the Society of German Reinforced Concrete Engineers, the ends of the longitudinal bars were 2 inches from the ends of the columns and left buried in the concrete. The resistance per square inch of the equivalent bars in these cases varied from 6·67 to 12·33 tons per square inch, the former figure being that corresponding to the large longitudinal bars with the interties spaced widely apart, the latter to small bars with more closely spaced interties.

Experiments A and B having drawn attention to the importance of the end conditions of the longitudinal bars, there was realised in Series C the most perfect arrangement from this point of view. The bars were cut square, and adjusted to

TABLE No. 22.

1.	2.	3.	4.	5.	6.	7.	8.
Number of Experiments.	Section.	Reinforcements.		Position of Ends of Longitudinal Bars.	State of Concrete when Placed.	Increase of Resistance Due to Reinforcements.	
		Longitudinal.	Transverse.			Per square inch of Total Section.	Per square inch of Equivalent Reinforcing Bar.
	Inches.					Lbs.	Tons.
			I. Columns Reinforced Longitudinally.				
			A. First Experiments of the Commission (*Columns* 16·40 *feet long*).				
5.	Square, 9·84 × 9·84 or 15·75 × 15·75.	4 bars from 0·472 inch to 1·772 inches diameter.	Plate inerties of 2·756 × 0·118 inch, spaced 11·81 inches apart.	0·27 inch from the ends and bared.	Rammed with care.	0·0	0·0
			B. Stuttgart Experiments (*Columns* 3·28 *feet long*).				
3.	Square, 9·84 × 9·84	4 bars 0·591 inch diameter.	Round ties 0·276 inch diameter, spaced 9·84 inches, centres.	2 inches from ends and buried in the concrete.	Rammed.	384	10·04
3.	Do.	4 bars 0·591 inch diameter.	Round ties 0·276 inch diameter, spaced 2·46 inches, apart centres.	Do.	Do.	910	12·33
3.	Do.	4 bars 1·18 inches diameter.	Round ties of 0·276 inch diameter, spaced 9·84 inches, centres.	Do.	Do.	697	6·67
			C. Second Experiments of the Commission (*Columns* 3·28 *feet long*).				
2.	Square, 7·87 × 7·87.	4 bars 0·709 inch diameter.	Plate interties of 1·575 × 0·118 inch, spaced 13 inches apart.	Exactly flush with the ends of the columns.	Poured.	427	5·97
2.	Do.	Do.	Do.	Do.	Rammed with care.	1,080	15·25

D. Third Experiments of the Commission (Columns 6·56 feet and 13·12 feet).

2.	Square, 7·165 × 7·165.	8 bars 0·591 inch, length 6·56 feet.	Interties 1·575 × 0·118 inches, spaced 13·11 inches apart.	About 0·1 inch from the ends and buried in the concrete.	Rammed, 7 cwts. cement. Do. 10 cwts. cement.	1,168 1,450	10·55 13·15
2.	Do.	8 bars 0·354 inch diameter, length 13·12 feet.	Interties 1·378 × 0·118 inches, spaced 13·11 inches apart.	Do.	Do. 7 cwts. cement. Do. 10 cwts. cement.	313 953	6·60 19·80

II. COLUMNS SPIRALLED CONFORMABLY TO ARTICLE 31[1] OF THE DRAFT CIRCULAR.

B'. Stuttgart Experiments (Columns 3·28 feet long).

3.	Oct., 10·83 inches diameter.	8 bars 0·433 inch diameter.	Spirals of 0·197 inch diameter, 1·53 inches pitch.	Spiral and reinforcement stopped 1·0 inch from the ends.	Rammed.	1,324	32·4 (1)[2]

[1] Article 31 of the Draft Circular provided as follows:—

If b is the least transverse dimension of a column,
d is the diameter of the longitudinal reinforcements,
s the distance apart of the interties or the pitch of the spirals,

and where m and m' are as defined on p. 5, then "m" may vary from 8 to 15.

$$m = 8 \text{ when } d = \frac{b}{10} \text{ and } s = b,$$

$$m = 15 \text{ when } d = \frac{b}{20} \text{ and } s = \frac{b}{3}.$$

When the transverse reinforcements are sensibly rectangular in plan,

m' may vary from 8 to 15,

$$m' = 8 \text{ when } s = b,$$

$$m' = 15 \text{ when } s = \frac{b}{3}.$$

When the transverse reinforcements are sensibly circular in plan

m' may vary from 15 to 32,

$$m' = 15 \text{ when } s = \frac{b}{2\cdot5},$$

and $m' = 32$ when $s = \frac{b}{5}$ and the pressure does not exceed 710 lbs. per square inch,

or when $s = \frac{b}{6\cdot5}$ do. do. do. 1,140 lbs. do. do.

or when $s = \frac{b}{8}$ do. do. do. 1,420 lbs. do. do.

Longitudinal bars, at least six in number, and having a total volume not less than 0·5 per cent. of the volume of the concrete nor less than one-third the volume of the spirals.

[2] For further reference to results of column 8, marked (1) to (12), see Table No. 23, p. 82.

TABLE No. 22—(*continued*).

1.	2.	3.	4.	5.	6.	7.	8.
Number of Experiments.	Section.	Reinforcements. Longitudinal.	Reinforcements. Transverse.	Position of Ends of Longitudinal Bars.	State of Concrete when Placed.	Increase of Resistance Due to Reinforcements. Per square inch of Total Section.	Increase of Resistance Due to Reinforcements. Per square inch of Equivalent Reinforcing Bar.
	Inches.					Lbs.	Tons.
			II. COLUMNS SPIRALLED CONFORMABLY TO ARTICLE 31 OF THE DRAFT CIRCULAR—(*continued*).				
			B′. Stuttgart Experiments (*Columns* 3·28 *feet long*)—(*continued*).				
3.	Oct., 10·83 inches diameter.	8 bars 0·433 inch diameter.	Spirals 0·276 inch diameter, 1·49 inches pitch.	Spiral and reinforcement stopped 1·0 inch from the ends.	Rammed.	1,380	26·0 (2)
3.	Do.	Do.	Spirals 0·394 inch diameter, 1·72 inches pitch.	Do.	Do.	2,105	28·2 (3)
			C′. Second Experiments of the Commission (*Columns* 3·28 *feet long*).				
2.	Square, 7·874 × 7·874.	6 bars 0·354 inch diameter.	Spirals of 0·236 inch diameter and 0·787 inch pitch.	Flush with the ends of the prisms.	Poured.	2,420	36·2 (4)
2.	Do.	Do.	Do.	Do.	Rammed.	2,365	35·2 (5)
			D′. Third Experiments of the Commission (*Columns from* 6·56 *to* 13·12 *feet long*).				
2.	Oct., 7·87 inches diameter.	6 bars 0·394 inch diameter, length 6·56 feet.	Spirals of 0·236 inch diameter, pitch 0·394 inch.	At 0·1 inch from the extremities and buried in the concrete.	Rammed, 7 cwts. cement. Do. 10 cwts. cement.	3,840—[1] 4,080	26·68 28·58 (6)
2.	Do.	Do.	Spirals of 0·236 inch diameter, 0·551 inch pitch.	Do.	Do. 7 cwts. cement. Do. 10 cwts. cement.	4,025 — 3,896 —	32·39 34·92

2.	Do.	Do.	Spirals 0·315 inch diameter, 0·984 inch pitch.	Do.	Do. 7 cwts. cement. Do. 10 cwts. cement.	2,670 — 3,655 —	24·12 33·02
2.	Do.	6 bars 0·354 inch diameter, length 7·55 feet.	Spirals 0·236 inch diameter, pitch 0·748 inch.	Do.	Do. 7 cwts. cement. Do. 10 cwts. cement.	2,945 — 3,300	34·92 39·38 (7)
2.	Do.	6 bars 0·315 inch diameter, length 8·53 feet.	Spirals 0·236 inch diameter, pitch 0·945 inch.	Do.	Do. 7 cwts. cement. Do. 10 cwts. cement.	2,233 — 2,361	33·66 35·57 (8)
2.	Do.	6 bars 0·276 inch diameter, length 9·84 feet.	Spirals 0·236 inch diameter, pitch 1·18 inches.	Do.	Do. 7 cwts. cement. Do. 10 cwts. cement.	1,153 — 1,864	21·59 35·57 (9)
2.	Do.	6 bars 0·315 inch diameter, length 13·12 feet.	Spirals 0·236 inch diameter, pitch 1·77 inches.	Do.	Do. 7 cwts. cement. Do. 10 cwts. cement.	1,536 1,693	33·02 (10) 36·83 (11)
2.	Do.	Do.	Spirals 0·197 inch diameter, 1·18 inches pitch.	Do.	Do. 7 cwts. cement. Do. 10 cwts. cement.	1,266 — 1,793	27·94 40·00 (12)

III. Spiralled Columns not Conform to Article 31[2] of the Draft Circular.

B″. Stuttgart Experiments (Columns having Insufficient Longitudinal Reinforcements).

3.	Oct. of 10·63 inches diameter.	4 bars of 0·276 inch diameter.	Spirals of 0·551 inch diameter, pitch 1·47 inches.	Spirals and reinforcements stopped 1·0 inch from the ends.	Rammed.	1,607	14·35

Columns with Spirals too Widely Spaced.

—	Do.	8 bars of 0·315 inch diameter.	Spirals of 0·591 inch diameter, pitch 3·18 inches.	Do.	Do.	938	11·30

[1] The figures followed by the sign — indicate maxima loads supported by the columns, which have buckled without crushing.
[2] See footnote [1] on p. 71.

bear exactly on the heads of the testing machine. The columns of this series, which were made with the minimum quantity of water necessary, have shown an increase of strength of 15·25 tons per square inch of the equivalent bar, whilst those made by pouring the fluid concrete into the mould only showed an increase of 5·97 tons per square inch at the end of twenty-eight days.

In the tests of Series D, the columns were 6·56 feet and 13·12 feet in length. Great differences of resistance were found in the non-reinforced columns of these lengths.[1] This fact diminishes the confidence one might have in the results of this series, and explains to some extent how identical reinforcements have given augmentations of resistance of 6·4 and 19·8 tons per square inch in columns which only differ by the proportions of the concrete. There is reason to set aside these extreme values.

All the columns of the first four series crushed without notable flexion.

The columns fulfilling the conditions which the Commission judged necessary to assure the efficiency of the spiralling have given the results B′, C′ and D′, which correspond to the Series B, C and D of the columns reinforced with intertied longitudinal bars.

The columns of Series B′ were made with little care in order to determine the minimum resistance one might obtain on works. For example, the spacing of the spirals from centre to centre has varied from half an inch to 3 inches in one single piece, and the longitudinal bars from 1⅜ inches to 5 inches.

In Series C′ it should be noticed that the spiralling has given to concrete

[1] The non-reinforced columns of 6·56 and 13·12 feet long were made at the same time as the columns of Series *D* and *D′* to serve as references. They were formed of the same concrete, and were rammed with the same care. Their resistance to crushing without buckling was for concrete containing

7 cwts. cement	3,440 lbs. per square inch	for columns	6·56	feet long.
	2,820 do.	do.	13·12	do.
Difference	620 lbs. per square inch.			
10 cwts. cement	2,630 lbs. per square inch	for columns	6·56	feet long.
	2,090 do.	do.	13·12	do.
	540 lbs. per square inch.			

To determine the cause of the differences of resistance of the columns 6·56 and 13·12 feet long pieces were cut from the top and the base of each of the columns, and it was found that the resistance of the base of each column exceeded that of the upper part by 1,365 and 2,620 lbs. per square inch for concrete containing 7 cwts. and 10 cwts. of cement respectively.

The explanation of these facts appears to be the following. The concrete gives up during the ramming and up till the commencement of setting part of the excess of water, which must in every case be added to facilitate the placing of the concrete. The water which thus comes from the lower beds adds itself in the concrete superposed to that which the latter contains. Thus the higher up one goes in the column the wetter is the concrete, with the above-mentioned result. The effect is the more marked the richer the concrete in cement. For the interpretation of the results of the experiments of Series *D′*, which included columns 7·55, 8·53 and 9·84 feet long, it was observed that the resistance of the concrete in these cases varied according to a linear law between the resistances observed for the concrete of 6·56 feet and 13·12 feet.

It is important to point out that in order to maintain a uniform quality throughout the whole height of columns it is necessary to employ drier concrete as the work ascends, and it is important to dispose the reinforcements in such a way as to permit of the employment of concrete gauged without excess of water.

These facts show how difficult is the comparison of reinforced pieces of different types and of the unreinforced pieces of the same type cast for comparison of the strength of the concrete. Pieces to be compared should have dimensions such that the final degree of humidity of the concrete is the same.

These facts explain the contradictions existing in experiments apparently comparable, and lead one to take account not of the isolated anomalies, but only of the main lines of the results of experiments.

poured without ramming and tested after twenty-eight days the same increase of resistance as concrete rammed with care.

The columns of Series D′ were prepared to study the resistance to buckling. Setting aside those indicated as having buckled without crushing, the others show an increase of crushing resistance, per square inch of the cross sectional area of the equivalent bar having the same volume as the longitudinals and spirals together, of 28·5 tons to 40 tons.

A number of columns tested at Stuttgart did not fulfil the conditions necessary to give to the spiralling the proper efficiency. In one case the longitudinal reinforcements were too feeble, in the other the pitch of the spirals was too great. The results show analogous increases to those of columns reinforced with longitudinal intertied bars. These experiments thus establish a transition between the two types of reinforcement. The following description of the test of a strongly spiralled column, which consequently presents features highly characteristic of this type of reinforcement, is given to enable a clear idea to be formed of the phenomena met with during the loading to destruction of such a column.

The column was 6·56 feet long, composed of concrete containing 7 cwts. of cement and reinforced to a total percentage of 4·91. The first fissures in the concrete covering the spirals were apparent when the pressure had attained 4,370 lbs. per square inch of the total section. The pressure was increased to a maximum of 7,050 lbs. per square inch, when buckling commenced. The loading was maintained until the moment when the deflection attained a value of 4 inches, the load at this time corresponding to a stress of 4,240 lbs. per square inch of the spiralled core, the external envelope of concrete over the spirals having by this time almost entirely disappeared. In the column thus bent there was no trace of crushing of the spiralled core, but in the parts of greatest curvature radial tension fissures were observed tending to cut the concrete in slices.

In columns which have crushed without bending, the failure of the column is produced sometimes by the thinning out and rupture of a spiral, sometimes by the shearing of the concrete on oblique planes.

Both in the columns which failed by flexion and those which ruptured, the concrete of the core away from the vicinity of the rupture presented no traces of alteration. This was demonstrated by cutting out test prisms from the cores of columns which had resisted high pressures. For example, crushing resistances of 3,930 and 2,470 lbs. per square inch were obtained from prisms cut from the base and the top respectively of a column containing 7 cwts. Portland cement and reinforced to 6·34 per cent., and which had resisted a pressure of 9,080 lbs. per square inch of the spiralled core. A column reinforced to 4·91 per cent., which had resisted a pressure of 8,740 lbs. per square inch, gave corresponding resistances of 2,260 and 2,030 lbs. per square inch respectively.

These facts indicate that cracking takes place prematurely in the outer cylinder of concrete covering the spirals, but that that does not compromise the solidity of the core on which the resistance to crushing depends. Such cracking must, nevertheless, be avoided in practice.

The experiments of Series D showed that, on the average, the pressure under which the first fissures appeared exceeded by about 940 lbs. per square inch the resistance of the non-reinforced columns, and that the excess resistance was almost the same for short columns reinforced to nearly 5 per cent., and for long columns only reinforced to 2 per cent. The pressures which spiralled pieces can resist without sustaining any damage whatever depends thus on the inherent resistance of the concrete, and is but little augmented with the percentage of metal.

In the experiments at Stuttgart, the results point to the same conclusions. In that case the resistance at which cracking of the envelope of the spirals took place exceeded by about 1,400 lbs. the resistance of the non-reinforced columns.

It may safely be allowed that in ordinary concrete, containing 6 cwt. of cement, for which a resistance of 2,280 lbs. per square inch may be reckoned on, the load which causes cracking in spiralled members is 2,850 lbs. per square inch. In members thus proportioned the pressure might thus be limited to 1,440 lbs. per square inch; that is, to 62 per cent. of the inherent resistance of the concrete, no matter how high the percentage reinforcement might be.

The pressure being thus limited to 1,440 lbs. per square inch for concretes of middling quality, and to 1,600 or 1,700 lbs. per square inch at the most for concretes of the best quality, and on the other hand, the coefficient of security 3, 4 or 5 being applied as regards crushing or buckling phenomena, it is evident that there is no reason for increasing the percentage of metal beyond that necessary to obtain resistances of 4,800 to 8,500 lbs. per square inch. For this reason the experiments made at the Laboratory of the Ecole des Ponts et Chaussées on cylinders very strongly spiralled, in which resistances up to 25,600 lbs. per square inch were obtained, have not been described. These experiments, however, show that the useful effect of spiralling extends to very wide limits.

When compression tests on concrete are made, creaking noises proceed from the concrete when the stress reaches a value which varies within somewhat wide limits. In the non-reinforced columns of Series D noises were heard, sometimes near the crushing load, sometimes when the stresses were less than 720 lbs. per square inch. In spiralled and longitudinally reinforced pieces the first noises were heard sometimes when the stress was in the neighbourhood of the crushing strength of ordinary concrete, sometimes when that figure had been exceeded by 700 lbs. per square inch. The cause of these noises is not known, but it is noticed that they precede but slightly the crushing of non-reinforced or longitudinally reinforced columns, and the cracking of the envelope in spirally reinforced columns. It is thus possible that these noises indicate to the ear the commencement of processes which are not visible to the eye.

The great ductility of spiralled concrete is another phenomenon which falls to be recorded. In short pieces, in which sensible uniformity of quality might be looked for, the shortening observed was from 1 to 3·6 per cent. of the original length. In longer pieces, where there is considerable variation from the top to the bottom, the shortening measured at the middle of the length has varied from 0·5 to 0·75 per cent. in columns 6 feet long, and from 0·3 to 0·4 per cent. in longer columns. The upper part, which had the least resistance, yielded before the concrete elsewhere had resisted the maximum load, and therefore the maximum deformation of which it was capable. The considerable bending observed in spiralled pieces is another proof of the ductility of concrete reinforced in this way.

Pitch of Spirals.

The network of spirals and longitudinal reinforcements which constitutes the cage is intended to prevent the transverse swelling of the concrete under the load. It opposes the escape of material between the meshes, and these ought evidently to be the closer, the greater the load designed for. Only experience can dictate the proper arrangement from this point of view.

In the Stuttgart experiments the spacing centre to centre of the spirals was from $\frac{d}{3\cdot 4}$ to $\frac{d}{2\cdot 3}$ in columns which gave medium results, and $\frac{d}{6}$ in those in which the resistances were conformable to the draft provisions of the Commission, d being the diameter of the circle inscribed in the octagon forming the transverse section of the column.

The feebly spiralled columns tried with success by Professor Guidi at Turin had the spirals spaced at $\frac{d}{5}$.

In the columns of Series D good results were obtained with a spacing of the spirals of $\frac{d}{8}$ for greater resistances, and $\frac{d}{4\cdot 4}$ for less.

The Commission recommended that the pitch of the spirals be not unduly diminished, owing to the difficulty occasioned thereby of ramming the concrete.

As all the experiments point to the ramming having a considerable influence on the mechanical qualities of the concrete, it is suggested that the efficiency of the network preventing lateral spreading should be increased for increasing loads by augmenting the section of the longitudinal rods rather than by decreasing the pitch of the spirals.

Elasticity.

The irregularity of concrete is still greater from the point of view of elasticity than from that of resistance. From a concrete composed of 6 cwts. Portland cement, 14·35 cubic feet of sand, and 28·70 cubic feet of gravel the Commission has obtained moduli of elasticity under a light load varying from $2\cdot 28 \times 10^6$ to $5\cdot 69 \times 10^6$ lbs. per inch^2, according to the quantity of water used in gauging and the method of ramming.

The modulus of non-reinforced concrete diminishes as the load increases, especially beyond a limit in the neighbourhood of one-half or two-thirds of the crushing load, and it is necessary, having regard to buckling, to take into consideration the modulus under pressures superior to the working load. Consequently, a modulus of elasticity of 2,130,000 lbs. per square inch, or 950 tons per square inch, may be properly allowed.

This irregularity in non-reinforced concrete necessarily shows itself in the tests of reinforced columns.

In Series A, the non-reinforced column had not the same dimensions as the reinforced one, and the results of this series cannot be compared. Only the columns of 15·75 inch side, in which the percentage varied from 0·28 per cent. to 3·97 per cent., are suitable for purposes of comparison.

There has been calculated the total resistance per square inch which the different columns gave when their shortenings had the same value of 0·02 per cent. In order to compare two columns the differences of their unital resistances has been divided by the shortening for which the comparison was made, and by the difference in the unital reinforcement.[1] The quotient gives a measure of the useful elastic effect of the excess of reinforcement in the more heavily reinforced column, and it ought, hypothetically, for average quality of steel to be about 29,100,000 lbs. per inch^2, or 13,000 tons per inch^2 if the reinforced columns were formed of identical materials simply associated in a common shortening.

[1] The unital reinforcement is the area of equivalent reinforcement per unit area of column or the percentage reinforcement ÷ 100.

On comparing the columns reinforced to 3·97 per cent. and 0·28 per cent. respectively, an apparent modulus of elasticity of 1,710,000 lbs. per inch^2, or 7,600 tons per inch^2, was found for the metal in excess of 0·28 per cent.

In Series C and D similar calculations give the apparent modulus of elasticity of the metal as varying from 17,100,000 lbs. per inch^2, or 7,600 tons per inch^2, to 10,000,000 lbs. per inch^2, or 4,400 tons per inch.^{2}

It was found experimentally in series A, that except in the neighbourhood of the ends, relative slipping of the concrete and steel was absolutely negligible.

These facts lead to the conclusion that since the metal does not slip and in consequence sustains the same shortening as the surrounding concrete, it certainly furnishes resistances corresponding to its modulus of elasticity. Its apparent effect having been much less, we are led to conclude that the modulus of elasticity of the concrete has been less in the reinforced columns in question than in those not reinforced. This weakness in the concrete might be due to three possible causes: either an increase in the quantity of water used in mixing the concrete, owing to the greater ease of working it in the moulds, or the less perfection of ramming owing to the constraint of the reinforcements, or owing to the prevention by the reinforcements of the full contraction of the concrete during its setting, and which, perhaps, will hinder it from acquiring all the mechanical qualities of which it is capable.

The first cause suggested did not operate in any of the experiments made by the Commission, because the quantity of water used was weighed with care, and the second did not affect the columns of Series A, as these were all similar.

Whilst the experiments of the Commission and also those of Professor Guidi at Turin gave results for the elasticity of the metal inferior to that indicated by the theory of elasticity, the experiments of Professor Bach at Stuttgart, under the auspices of the Society of German Reinforced Concrete Engineers, gave results for the moduli of elasticity of longitudinal reinforcements corresponding sensibly to the figures deduced from theory.

All that may be affirmed regarding the elasticity of pieces longitudinally reinforced is that in careful experiments the apparent elastic effect of the longitudinal bars has varied from less than half the value indicated by the theory of elasticity to values sensibly equal to that value. Under these conditions it seemed reasonable to adopt the simplest solution, which consists in allowing from the point of view of elasticity the coefficient of equivalence between the metal and the concrete which were adopted from the point of view of resistance to crushing. These coefficients vary from 8 to 15, combined with a modulus of elasticity 2,130,000 lbs. per square inch, or 952 tons per square inch; they attribute to the metal values of the modulus of elasticity varying apparently from 7,616 tons per inch^2 to 14,280 tons per square inch, which agree with the results obtained at Paris, Stuttgart, and Turin.

The causes indicated above that diminish the coefficient of elasticity act in the same sense on the resistance of concrete to crushing.

The Commission decided that whilst awaiting further experiments on the effect of transverse reinforcement or spiralling on the elasticity of concrete, it is prudent to entirely neglect any increase which such reinforcement might give.

Deformation of Plane Sections in Bending.

Although not rigorously exact, the hypothesis of the conservation of plane sections serves as a rational basis to the theory of resistance of homogeneous materials. Its application to a heterogeneous material such as reinforced concrete *à priori*, open to serious doubt. One might readily understand that the slipping of the reinforcements and the cracking of the concrete in tension would make the hypothesis entirely unworkable.

A very great number of experiments made on the beams 7·87 inches × 15·75 inches, and 13·12 feet span, have proved that sections originally plane remain sensibly so during bending, except in the immediate neighbourhood of the supports and of the points of application of heavy concentrated loads.

Away from these exceptional points, where in actual structures it might be useful to consider specially the strength from the point of view of local stresses, the warping of sections originally plane does not modify notably the longitudinal deformations, which serve as a basis for the calculation of deflected pieces. Any deviation from the hypothesis of the conservation of plane sections may thus, without sensible error in the calculation of tensions or compressions, be neglected.

Application of the Laws of Simple Deformations to the Calculations of Bent Pieces.

In the theory of bending of homogeneous materials, it is allowed that to a certain longitudinal elongation or contraction there corresponds a certain tension or compression in bending and also in traction or thrust.

To discover to what extent this assumption is applicable to reinforced concrete, the following investigations were carried out:—

A number of prisms were made simultaneously with the beams of 7·87 inches × 15·75 inches section, and 13·12 feet span, and the moduli of elasticity of the concrete and the reinforcements respectively were measured. Applying with these figures the above hypothesis, there was determined by calculation the position of the neutral axis, in the period of deformation after the limit of elasticity of the stretched concrete had been passed; that is, in the period in which the tension of the elongated concrete ought to be constant. Results were found almost rigorously to conform to those given by actual measurement during experiment.

It has been concluded that the hypothesis in question might, concurrently with the hypothesis of the conservation of plane sections during bending, be safely applied in calculation, assuming at the same time that the laws of deformation by tension or thrust are those already formulated.

Adhesion of the Concrete to the Metal.

For simple tension or compression on isolated bars imbedded in concrete the Commission found values of the resistance to unbedding of from 97 to 420 lbs. per square inch of the surface of contact. For bars surrounded by spirals or placed in the folds of perpendicular stirrups, these figures varied from 225 to 560 lbs. per square inch.

These figures give no indication, however, of the tendency to slip in members where bending imposes other stresses on the member. For this information, beams in which failure took place by slipping of the reinforcements were studied, and in doing so it was assumed, according to the usual hypothesis conform to the theory of perfectly elastic bodies, that the slipping stress varied with the shearing stress. Consequently the resistance to slipping furnished by the ends of the longitudinal reinforcement, which projected beyond the supports, was neglected, since there is no shearing force there. The values of the adhesion thus found varied from 185 to 213 lbs. per square inch. Other experimenters applying the same mode of calculation have found similar values.

Resistance to Shearing Forces.

The experiments indicate that whilst vertical stirrups are useful in a beam just before the formation of cracks and after cracks have formed, stirrups inclined in the direction of the tension, or longitudinal reinforcing bars bent upwards towards the supports, give much more help to the concrete under working loads.

Beams which were reinforced in both tension and compression areas, and which had no stirrups, continued to support a load in virtue of the resistance of the reinforcements after the concrete had fissured for $\frac{5}{8}$ of its full depth in the part exposed to shear. The resistance of the reinforcements to the shearing forces was probably aided by a kind of frictional resistance in the compression area of the beam.

Experiments on the Resistance and Deformation of Works Constructed for the Exhibition of 1900.

Experiments were made under the direction of M. Rabut on—

1. Two floor slabs.
2. A ribbed floor.
3. A footbridge.
4. A retaining wall.

These structures were designed by the Hennebique empirical formulæ, and constructed according to the usual practice of that firm. The experiments were carried out to determine—

1°. What margin of security might be relied on.

2°. What degree of solidarity there was between the different members.

3°. If any relation could be determined between the loads and the deformations.

On the first point the experiments showed that in the case of the slabs they carried a surcharge equal to from five to six times that for which they had been designed. The ribbed floor supported without a rupture a load three to four times greater than that for which it had been designed, whilst the retaining wall withstood a very much greater load than that which ought to result from the thrust of the earth.

On the second point it was observed that concentrated loads were distributed not only on the single rib on which they rested, but sometimes on five and even on as many as seven.

On the third point, although a precise statement of the relation between the deflections and the stress on the steel or concrete has not been possible, the fact that the deflections under growing loads follows a continuous law, in spite of the formation of cracks, was clearly demonstrated.

Detailed information regarding the design and construction of these structures, together with the records of the tests and the analyses of the results, form Part I. of the French edition of the Report.

CHAPTER VI

Some Conclusions from the Study of the Elementary Properties of the Materials Constituting Reinforced Concrete

Calculation of Stresses and Deformations.

Once the hypothesis of the conservation of plane sections is accepted with reference to reinforced concrete structures, the deformations and the stresses under any loading whatever may be calculated from the known properties of the components, but in every case the calculation is a complicated one and frequently almost insoluble.

So far as stresses are concerned, the tension in the concrete ought to be neglected, which leads to a very considerable simplification in the calculations, and the Commission suggested that that method should also be applied to the calculation of deformations. The deformations thus calculated are in excess of those to be expected, but the calculated values might be corrected by means of a coefficient of correction, obtained by a comparison of the calculated value with that actually found in a large number of experiments. Such a coefficient would be a function of the percentage reinforcement.

Coefficients of Security.

The Commission were of opinion that, taking into account the excellent protection afforded to the metal from abrasion, oxidation, and largely from shocks, and that in well-designed reinforcements holes and grooves are avoided, the metal may safely be stressed by working loads to half its apparent elastic limit.

The risk incurred in exceeding the limit of elasticity is twofold—viz., the rapid increase of the deflection and the destruction of the adhesion. The regulations require the help afforded by the concrete to be neglected in calculating the tension in reinforcements as the reinforcements have to resist the whole tension across any crack that may be formed. Diminution of adhesion as a result of high tension is likely to take place only at the centre of simple spans or over the supports of a continuous beam. In the former case the shear near the middle of the span is generally small, but at the supports of a continuous girder the shears are maxima, so that in this case care must be taken that the slipping stress is not excessive.

The maximum compression fixed for concrete implies a certain shortening of the latter, and consequently of the steel embedded in it. The compression corresponding to this strain of the steel is always well within half the elastic limit.

The Commission proposed as a factor of safety for the concrete of 3·5, owing to the large influence workmanship has on its qualities.

Simplification of the Calculations.

In Table No. 22, on p. 70, in the figures making comparison of the strength of columns reinforced with longitudinal bars intertied, the effects of the longitudinals and the interties are not separated, and the coefficient of equivalence is taken at the same value, 8 to 15 for both. This is to some extent justified by an examination of the Stuttgart results. There three types of column were tested, which in addition to the longitudinal bars of 0·591 inch diameter, had interties spaced at 9·84, 4·92 and 2·46 inches. Differences of resistance were found varying from 8·2 to 14·6 tons per square inch of the section of the equivalent longitudinal bar, having a volume equal to that of the ligatures. These figures correspond to the figures given by the longitudinal bars.

On the contrary, longitudinals and spirals produce widely different effects, which must not be confounded in calculations. In order that steel of middling quality may give a resistance of about 16 tons per square inch, which is generally about its apparent limit of elasticity, it must undergo a contraction of about 0·1 per cent. Spiralled concrete supports before crushing contractions of from 1 to 3 per cent. The resistances provided by the enclosed longitudinal bars are thus not less than 16 tons per inch^2, and are not much beyond this figure, as once the yield point is passed the resistance increases but slowly with the strain. It follows that one is very near the truth in deducting from the total resistance of a spiralled column, first the resistance of the non-reinforced prism of the same dimensions, then the resistance of the longitudinal bars, reckoned at 16 tons per square inch, and attributing the remainder of the effect to the spiralling.

In Table No. 23 the results are given of this calculation made for the columns figuring in Table No. 22 (division 8, marked (1) to (12)), which fulfilled the conditions indicated by the Commission, and which did not buckle. The second horizontal row gives the increase in strength, after deducting the resistance of the non-reinforced comparison columns, reckoned in tons per square inch of the equivalent longitudinal reinforcing bar, having the same volume as the whole of the metallic reinforcing. The lowest line gives the results when the equivalent longitudinal bar has the same volume as the spirals only, and when in addition to the resistance of the non-reinforced comparison columns, the resistance of the longitudinal reinforcement is also deducted.

TABLE No. 23.

1	2	3	4	5	6	7	8	9	10	11	12
32·4	26·0	28·3	36·2	35·2	28·6	39·4	35·6	35·6	33·0	36·8	40·0
65·4	36·0	35·2	45·4	44·5	32·4	49·5	43·8	43·8	49·5	55·8	57·8

These figures complete the elements necessary for the design of compression pieces.

The Commission pointed out that there is necessarily no hard-and-fast division between longitudinally reinforced columns and spiralled columns; the two types merge into each other in their extreme examples, but that the *rôle* of longitudinal reinforcement is essentially different from that of transverse reinforcement, the former actually resisting the longitudinal stress, the latter simply helping the enclosed concrete to resist it.

The Commission suggested values of m the coefficient of equivalence of the longitudinal reinforcement and the concrete varying from 8 to 15. Similar values for m' would apply, as previously explained, to the lateral interties between the longitudinals, whilst for spiralling the values of m would vary from 8 to 32. In every case stress was laid on the necessity for keeping the compression of the concrete within 62 per cent. of the resistance of non-reinforced concrete.

Although in the later stages of the test of a reinforced column the spiralled core alone resists the whole load, it is not advisable, except under certain circumstances, to complicate the calculations by relating the resistance to the area of the core instead of to the total area of the column. It might be advisable to do so when the ratio of the total section to the core differed from 140 or 150 to 100.

So long as the ratio of length of a column to its least transverse dimension is not more than 20, no calculations relating to buckling need be made.

When buckling must be taken into account, either Euler's or Rankine's formula may be used. The former is rigorously applicable only to perfectly straight and symmetrical pieces, and for its application the modulus of elasticity for all the stresses concerned must be known. As the modulus of elasticity of concrete does not fall below 950 tons per square inch, the resistance to buckling of non-reinforced columns may be calculated, using that figure (see p. 17).

For reinforced columns of the ordinary type the section Δs of metal may be replaced by a section $m\Delta s$ of concrete and Euler's formula applied, using the above-mentioned value for the modulus of elasticity, m having the values previously defined.

In the case of a strongly spiralled column, where the stress that might be applied without fear of crushing approaches that at which the limit of elasticity rapidly diminishes, the question is rather more complicated.

In Table No. 22 there is indicated the resistance attributed to the reinforcement, but in studying buckling the total resistance of each column has to be taken into account. This is given in division 4 of Table No. 24.

It was pointed out on p. 74 that the resistance of the comparison columns varied from 2,090 lbs. to 3,440 lbs. per square inch. In order to render the figures comparable amongst themselves, it was necessary to correct the observed resistances to those which would have been obtained if the concrete had had a uniform resistance of 2,280 lbs. per square inch.

In division 7 of Table No. 24 there is indicated the resistances to crushing, calculated by attributing to the concrete a resistance of 2,280 lbs. per square inch, together with a coefficient of equivalence of 15 for the longitudinal and 32 for the spiral reinforcements.

TABLE No. 24.

1.	2.	3.	4.	5.	6.	7.
Lengths of Columns.	Percentage of Longitudinal Bars.	Percentage of Spirals.	Maximum Load Supported in lbs. per square inch of Total Section.	Corrected Loads for Columns which Crushed.	Corrected Loads for Columns which Buckled.	Calculated Crushing Resistances.
Feet.				Lbs. per square inch.	Lbs. per square inch.	Lbs. per square inch.
6·56	1·42	4·92	7,280—6,690	6,350	6,110	6,325
6·56	1·42	3·49	7,035—6,510	—	5,870—6,170	5,290
6·56	1·42	3·46	6,110—5,840	—	4,950—5,930	5,260
7·54	1·15	2·58	6,270—5,830	5,570	5,220	4,550
8·53	0·91	2·05	5,460—4,805	4,635	4,510	4,080
9·84	0·69	1·64	4,270—4,223	4,135	3,430	3,690
13·12	0·91	1·09	4,350—3,780	3,968	—	3,400
13·12	0·91	1·13	4,080—3,880	3,550—4,070	3,810	3,370

In each case, with one exception, the crushing loads are in excess of those calculated. In the case where the crushing load was actually less than the calculated crushing load, the percentage of reinforcement was the greatest, the longitudinals amounting to 1·42 per cent. and the spirals to 4·92 per cent. The draft regulations suggested by the Commission indicated that the ratio between the percentage of longitudinal reinforcement and spiral reinforcement should not fall below $\frac{1}{3}$. In the column in question this condition was not observed, and, further, the pitch of the spirals (0·39 inch) did not allow of good ramming.

It may be observed also, taking into account the general behaviour of the phenomena, rather than the anomalies, that the loads which have produced buckling, corrected as above described, are greater than the calculated resistances to crushing.

If, thus, for a column having the same proportions and the same percentage reinforcement as one or other of the columns tested, the crushing resistance is calculated by the above method, using any given factor of safety from that point of view, then as regards buckling the factor of safety will be at least equal to that chosen against crushing.

It will be the same in a greater degree for columns less reinforced than those the results of which are tabulated, since the modulus and the limit of elasticity on which depend the resistance to buckling diminish much less quickly with the percentage of metal than the resistance to crushing actual or calculated.

For analogous reasons buckling precedes crushing for columns more strongly spiralled than those experimented on, but all risk is avoided if the load applied to a column is not allowed to exceed, however high the percentage reinforcement, the safe load for the column of the same proportions in the series of experiments in question.

This limitation, although not quite logical, will not impose any undue restraint on constructors, because the percentage reinforcement chosen for the columns of various proportions was not arrived at by chance, but as the result of earlier experiment to determine the most efficient percentage for the columns of varying proportions.

The most efficient percentage for the shorter columns was found to be about 3·5 per cent., and it is possible that for the slenderest column slightly greater resistances might be obtained by higher percentage reinforcement than 1·09 or 1·13.

The limits of load deduced from the experiments are given in Table No. 25 and were determined as follows :—

The ratios of length to least dimension were plotted as abscissæ, and the load supported by the different columns as ordinates. The mean curve was traced by suggestions of the known forms of the curves of resistance to buckling. The ordinates of this curve divided by 4 gave the values appearing in Table No. 25.

TABLE No. 25.

Ratio $\frac{\text{length}}{\text{least dimension}}$ . .	10	11·5	13	15	17	20
Maximum working compression, lbs. per square inch.	1,422	1,266	1,138	995	924	853

To indicate the extent of the anomalies observed, it suffices to say that if instead of the mean values obtained by the curve, the actual extreme values had been taken account of, the factor of safety given by the above figures instead of being uniformly 4 would have varied from 3·5 to 4·8.

The limits of pressure just indicated are suitable for concrete having a resistance of 2,275 lbs. per square inch when tested in short columns between the heads of an hydraulic press.

Columns in buildings are under conditions at least as favourable when the members on which they rest as well as those they support, and to which they are strongly bound by the prolonged reinforcements, might be considered as exempt from angular displacements. When the loading in such cases is unsymmetrical, it will be observed that it is less than the maximum in view of which the column has been designed.

To obtain data for designing a column which has to be subject to a bending moment, the amount of which it is impossible to calculate, such as that due to an eccentricity of the load, the Commission experimented on eccentrically loaded columns. They found that with columns reinforced with intertied longitudinal bars, when the load was applied at a point a quarter of the width of their bases from the centre, the maximum load which they carried was equal on the average to 60 per cent. of the maximum load of columns squarely supported between the heads of the press.

A spiralled column loaded at the outer edge of the middle third, but prevented from acquiring an inclination of more than 3·4 in 100 at the ends, conditions worse than any to be met with in practice, the resistance was not sensibly inferior to that of a similar column centrally loaded.

The Commission concluded that it would be prudent to reduce the average load on columns which have not been calculated from the point of view of flexion to 60, 70 or 80 per cent. of the load allowed for centrally loaded columns, according as the column is placed in the angles, the façades or symmetrically under beams in structures.

Members Subject to Compression and Bending.

The Commission recommended for the design of such members the method which attributes to each element of the material a stress proportional to the product of its longitudinal deformation and its corresponding modulus of elasticity.

We have previously seen that all longitudinal bars of metal might be replaced by cylinders of concrete of section m times greater than that of the metal, and that this coefficient of equivalence might receive the same value, whether it was applied to resistance or elasticity. These experimental results are equally applicable to bending or compression.

Experiment proves that concrete is capable of greater shortenings without crushing in bent beams than in columns, so that the reinforcements in the former develop more fully the resistances of which they are capable, and there is no reason to suppose that even in slabs which support compression in conjunction with the bending of a beam are the values of the resistances less than in columns strongly reinforced. The maximum value of m should thus be applied to members under simple bending.

On the other hand, it may be necessary to reduce the value of m if precautions are not taken to assure maximum efficiency to the longitudinal bars, when the ratio of the compression produced by the longitudinal resultant of the exterior forces to that produced by the bending moment rises.

It has been previously explained that the elastic effect of transverse reinforcements is small, and even although the resistance of a properly spiralled member is great, yet the modulus of elasticity at high stresses is low. There is consequently no reason to take account of the elastic effect of the transverse reinforcements in members subjected to bending, whilst the elastic effect of the longitudinals is fully accounted for by the equivalent increase of section of the concrete from the point of view of resistance.

To simplify the calculations, the compressive stress in members subject to bending is taken as proportional to the deformation, and the tensions in the concrete are not taken account of. It is evident that the tensions should not be taken account of, because at the crossing of the minute cracks which cannot always be avoided, the whole load is taken by the reinforcements. In other respects the tensions of the concrete are not negligible, as they influence not only the deformation of the neutral line, but its position in the cross section. It is established by experiment that within the limits of working stress, cracks have no sensible influence on the deformations of bent members or on the crushing of compressed members. From this point of view the concrete ought to be treated as if it produced everywhere the tension of which it was capable.

When the tensions in a member subject to bending are not taken account of, the total calculated compression must be augmented to furnish the necessary moment of resistance, so that the calculated value of the maximum compression is increased above what is really its actual value. The error is the greater the greater the proportion the tension omitted from the calculations bears to the tension value of the reinforcement—*i.e.*, the error is the greater the lower the percentage of reinforcement.

This error may to some extent be compensated for in the choice of the value of the ratio m. By increasing the value of m above its true value—*i.e.*, by exaggerating the superiority of the elasticity of metal over that of concrete, the extensions calculated for the former are reduced and the shortenings calculated for the latter are increased. A greater cross-sectional area of concrete is thus required over which the compression component of the resisting moment is distributed. The result of using such an inflated value of m is virtually to lower in the beam the calculated position of the neutral axis, and to reduce the calculated value of the maximum compressive stress. The absolute values of the two kinds of contrary errors thus committed vary, if not proportionally, at least in the same sense: both are maxima in slabs and beams of rectangular sections and minima in beams of T section formed of a slab and rib.

The error committed in neglecting the tensions of the concrete is less as the percentage of the reinforcement is increased, whilst the error resulting from the exaggeration of m is less as the ratio of thickness of slab to height of rib is reduced. These errors compensate in a sufficiently satisfactory manner in slabs and beams of whatever proportions.

In fixing on 15 as the value of m, the ratio between the modulus of elasticity of steel to that of concrete in compression, the following considerations influenced the Commission:—

The modulus of elasticity of concrete in compression varies from 1,020 tons per square inch to 2,540 tons per square inch under light loads, the former applying to concrete gauged with excess of water and imperfectly rammed, the latter containing only the water necessary for its use and thoroughly rammed.

A high modulus of elasticity is generally accompanied by a high resistance, and *vice versâ.*

It must be remembered that as the stress increases, the value of the modulus of elasticity is reduced, and, consequently, the lowest value which can be met with under working circumstances must be used. This has been taken as 933 tons per square inch, and as the modulus of elasticity of the steel generally used is about 14,000 tons per square inch, the value of m is 15. The value of the compressive stress to be allowed in the concrete is closely related to the value of m. The higher the value of m chosen, the lower the compressive stress which ought to be reckoned on.

Adhesion of the Concrete to the Metal.

Assuming, according to the elementary theory of perfectly elastic bodies, that the tendency to slip at any section is proportional to the shearing stress on that section, the resistance offered to the slipping of the longitudinal reinforcements has been calculated, in the beams of the first series which failed in that way, at from 170 to 213 lbs. per square inch of the area of contact. In many existing works the same mode of calculation indicates values as high as 142 lbs. per square inch as the measure of the tendency to slip.

The Commission carefully considered the question whether common practice in this respect ought to be modified, keeping in view the difficulty of doing so without largely increasing the cost or hindering the ramming.

Experiment showed that in the relative slipping of the reinforcement and the concrete, as in the other deformations of reinforced concrete, a gradual diminution in the modulus of elasticity took place as the stress was augmented, and after the real elastic adhesion was destroyed, the frictional resistance to slipping was still considerable.

In the second series of bending tests it was found that with beams similar in all respects to those of the first series, but supported about 1 foot 4 inches from one end instead of about 4 inches as in the first series, failure did not take place by slipping of the reinforcements. This fact is explained by the additional resistance to slipping which the reinforcements obtained from the part beyond the supports. According to theory, however, the shearing stress was zero there, and in consequence there should have been no additional resistance to slipping obtainable from this part.

To investigate this point, the longitudinal slipping was measured in two places, one between the support and the load when the shear and presumably the tendency to slip was maximum and constant, the other in the projection beyond the support when the shear was everywhere zero. It was found that, up to a certain limit, the observed facts agreed with the theory. The slipping was zero outside the supports, but between the load and the support it had quite an appreciable value, which varied with the load. Beyond a certain value, slipping occurred throughout.

These facts led to the following conclusions:—

So long as the elasticity as regards slipping is intact, the hypothesis that the slipping forces are proportional to the shearing forces is nearly correct. When, however, the elasticity has been reduced or broken down at a part, the slipping spreads itself over the whole length of the reinforcement, and equilibrium is established with the maxima stresses sensibly inferior to those indicated by the formula. This would explain the phenomena observed in the experiments, and this interpretation may be exhibited graphically by Fig. 27 for the case of a beam with a concentrated load or with a distributed load. The ordinates of the full straight line represent the shear at the various sections in the half span, those of the dotted curved line the slipping stress at these sections, and in the part of the beam overhanging the support.

These considerations have led the Commission to the view that a low unital stress should be adopted for slipping where no additional resistance can be given; but where additional resistance to slipping is obtainable, while its exact calculation

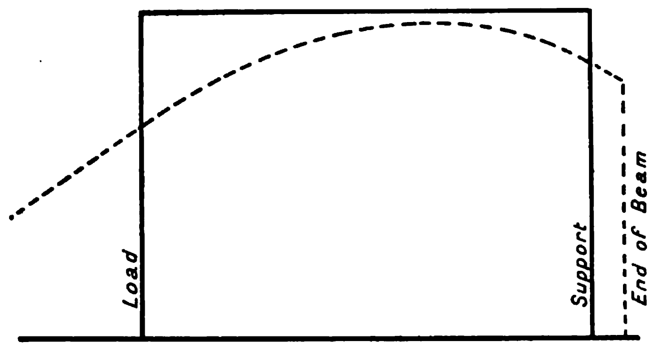


Diagram for Concentrated Load.

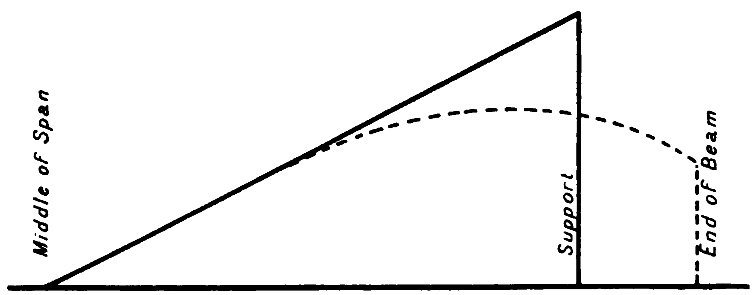


Diagram for Distributed Load.

FIG. 27.

may not be possible, some allowance should be made by increasing the stress allowed.

Transverse Reinforcement of Members subject to Bending.

In ribbed beams transverse reinforcements, either alone or with the help of longitudinal tension reinforcement raised up towards the ends calculated to resist the shear, are generally used, whilst in slabs where the ratio of the depth to span is not excessive no transverse reinforcements are generally employed.

The Commission thinks the practice justified for the following reasons:—

The ribs are generally executed several days before the slabs, and unless great care is exercised, and even then, the junction between the two portions forms a decided plane of weakness, and slipping can only be prevented by reinforcements passing across this plane. These reinforcements should be capable of resisting the whole of the tendency to slip on that plane, and consequently they must be able to resist the shearing force.

On the other hand, slabs are usually put in in one operation, and it is thus only from the point of view of rupture across the full concrete that their resistance to shear ought to be assured.

So long as it is not fissured concrete resists shearing, and when the reinforcement is less than 1 per cent., a stress of 65 lbs. per square inch may be resisted. The resistance of intact slabs is thus generally assured by the concrete alone. The experimental slabs loaded to destruction perished not by oblique shearing, but by elongation of the reinforcements and crushing of the concrete. The beam P, on p. 36, without transverse reinforcements continued to resist shear after the formation of oblique fissures cutting across more than five-sixths of the section. The shearing force could only have been equilibrated in the planes of the cracks by the transverse resistance of the longitudinal reinforcements, and by a kind of friction developed in the particles of the compressed concrete remaining in contact in the prolongation of the fissures.

In this connection it may be pointed out that the resistance longitudinal reinforcements present to shearing, vertical or oblique, is always superior to the shearing effort, and that except in beams carrying all the load concentrated in a single point, the tension of the reinforcements diminishes as the shearing effort increases to such an extent that there remains sufficient free resistance in the metal to resist the shear.

The longitudinal reinforcements are thus almost always sufficient to prevent rupture on oblique planes in beams of whatever dimensions if the solidity of the enveloping concrete is assured. The danger of the concrete itself breaking up is less when the percentage reinforcement is low, and when it is split into as many bars as possible.

Slabs are thus well conditioned from both these points of view, and it is not generally necessary to employ transverse reinforcements, the number and weakness of temporary fixing of which, render ramming difficult. If, however, the ratio of depth to span is great, and consequently the longitudinal reinforcements small in respect to the shearing effort, there should be added transverse reinforcements calculated to resist the shearing effort with the help of the concrete and the longitudinal reinforcements.

Empirical Formulæ.

The Commission recognised the necessity, in structures where the calculation of the dimensions from a purely theoretical basis would be too complicated for practical purposes, of the use of empirical expressions. The following observations were made regarding the basis on which such expressions should rest so as not to be irrational or to conflict with known facts.

In a structure of ribs and slabs, the ribs impose almost completely their longitudinal deformations on the contiguous portions of the slabs they support, the effect dying out with distance from the rib.

It is covnenient, as previously described, to allow that for a fixed width on both sides of each rib the slab is completely solid with it from the point of view of bending, the solidarity ceasing abruptly outside this width.

In the case of highly concentrated loads, such as a wheel load, the pressure is transmitted not only vertically but obliquely, forming a kind of cone, the base of which has greater dimensions the greater the total thickness of causeway, filling and slab. The unloaded parts outside this imaginary conical zone participate in the support of the latter, and the deformation extends transversely the greater the span of the slab. Any expression for the distribution of such a load ought thus to contain as variables the combined thicknesses of causeway, filling and slab and the span of the latter. The coefficients should be determined by the analysis of experimental results, and so as to give results conform to successful practice.

In the case of slabs carried by two sets of beams at right angles, the problem, analogous to that of a plate supported or fixed along all four sides, is complicated by the heterogeneity of the material. Any expression used should give the results as for ordinary beams when the spans are infinitely different, and when they are equal should give results agreeing with successful practice.

In considering the division of the loads between parallel beams, it is not possible to do more than keep in view the principle of the solidarity of the main beams, the secondary beams and the slabs, which can only be applied by reference to experiments on analogous works.

Tests.

The importance of rapid tests by rolling load has been drawn attention to in the regulations, as the deformations resulting from variations of temperature during tests of longer duration are frequently of the same order as those produced by dead loads.

CHAPTER VII

Résumé of Notes Presented by M. Considère

A. Tension Tests on Prisms of Reinforced Concrete.

The description and particulars of the tests are given in Chapter IV., 5, p. 23, and the results are exhibited graphically in Fig. 28, p. 92.

Precautions were taken to prevent any twisting of the test pieces, and the measurements taken showed that they were successful, consequently the variations produced by the loading and unloading give reliable information.

In the test pieces the area of the concrete was 15·5 square inches and the area of the metal 0·175 square inch, and its modulus of elasticity $29{\cdot}1 \times 10^6$ lbs. per square inch.

An elongation dl in the prism, which produces the total augmentation of tension dT, produces in the reinforcement an algebraical augmentation of tension dt_m, equal to $dl \times 0{\cdot}175 \times 29{\cdot}1 \times 10^6$ lbs., and the difference $dT - dt_m$ is the increase in the tension in the concrete which the elongation has produced. To deduce from the variations of tension thus determined the absolute tensions of the concrete or the steel, it is necessary to know the reciprocal and equal reactions between the associated materials at the beginning or the end of the period during which the variations have been studied.

The twenty-fifth and last unloading of an experimental prism (No. 3 prism), during which the total tension was reduced from 3,945 lbs. to 441 lbs., the minimum load necessary to keep the whole apparatus taut, may be taken as an example.

The point where the pencil stopped is indicated on Fig. 28 by the letter A, and if the irregularities due to the apparatus are overlooked, it is seen that the curve of unloading differs little from a straight line, so that without appreciable error it may be taken that if the unloading had been finished, the curve of unloading would have been a straight line terminating at B.

To determine the reciprocal reactions between the concrete and the reinforcement, the length of one of the reinforcements, the ends of which projected beyond the ends of the prism, was measured. The concrete overlying this reinforcement was then very carefully removed, and the reinforcement slackened in its bed, but without destroying the latter, so as to conserve the curves which would influence its length. The increase in length was found to be ·00433 inch on a length of 55·12 inches. The contraction which the adhesion between the concrete and the reinforcement had imposed on the latter even after twenty-five applications of a load of 3,945 lbs. was thus 0·008 per cent.

To completely annul the compression which the contraction of the concrete had imposed on the reinforcement, it would have been necessary to have increased the length of the prism by 0·008 per cent.; that is, to take it to the point marked C on the diagram. If through this point of equilibrium of the reinforcements a straight line FF' is drawn having an inclination to the horizontal of $0{\cdot}175 \times 29{\cdot}1 \times 10^6$ to 1, then each vertical intercept between FF' and OO' represents algebraically the tension in the reinforcement at that phase.

The tension produced by the concrete at any deformation is measured by the intercept between the deformation curve and the line FF'.

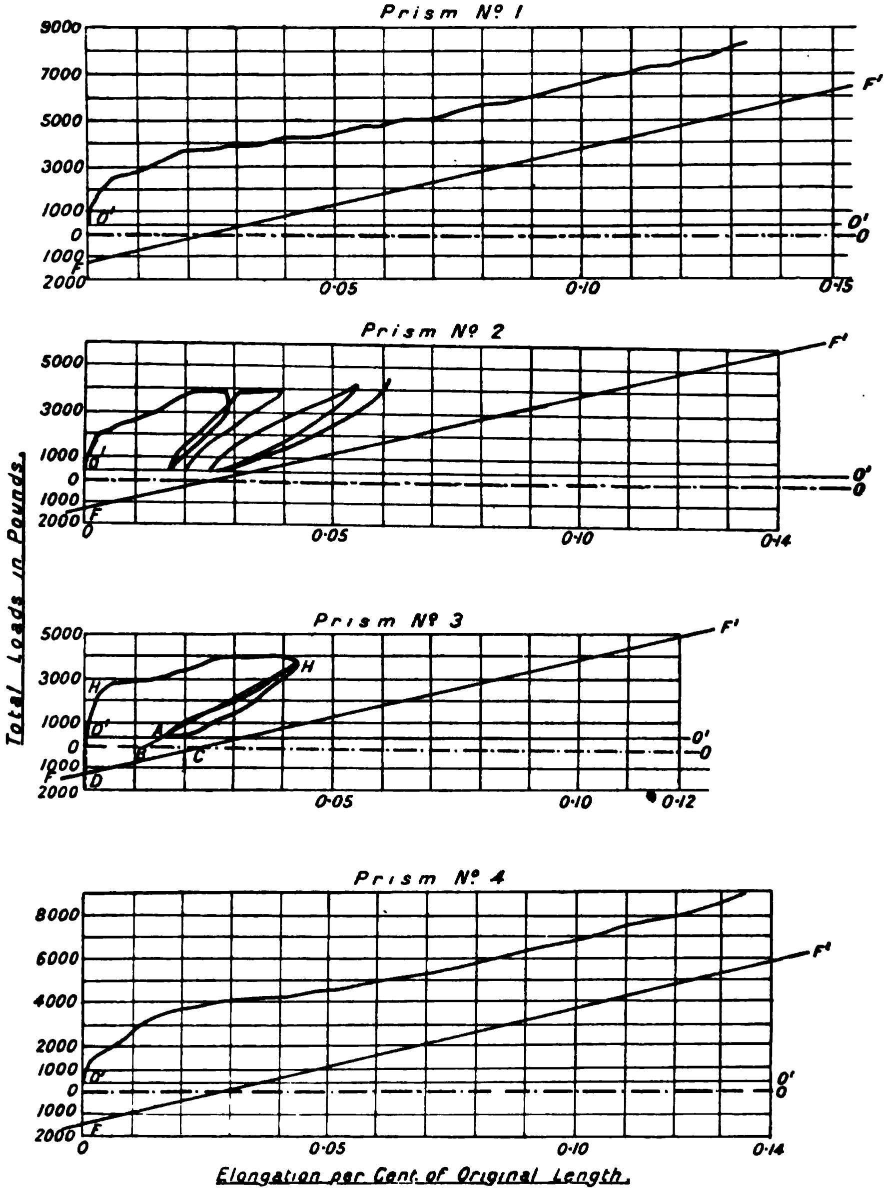


FIG. 28.

Since the prisms were made of the same materials with equal care and by the same workmen, it might be allowed without sensible error that the line FF' has the same relation to the origin of the four deformation curves.

The following inferences can be drawn from the four graphs :—

1°. *Initial State.*

To obtain the true origin the deformation curve must be continued back to the zero load-line at the inclination it has immediately above the 440 lbs. load-line. The intercept OD measures the tension in the concrete due to its being prevented by the adhesion of the reinforcements from taking its full contraction. This tension amounts to 1,146 lbs., or about 74 lbs. per square inch. The compression in the reinforcements due to the same cause is 1,146 lbs., or 6,540 lbs. per square inch. This compression in the metal corresponds to a strain of $\frac{6{,}540}{29 \cdot 1 \times 10^6}$; *i.e.*, of $2 \cdot 25 \times 10^{-4}$, or 0·000225, or a shortening of 0·0225 per cent.

2°. *Elongation without Cracking oj the Concrete.*

Prism No. 1 underwent an elongation of 0·135 per cent. before the first crack was apparent to the eye. Prism No. 2 underwent a maximum elongation of 0·061 per cent., with repetitions of smaller elongations, without apparent injury. The reinforcements were removed, and it was not till almost the end of the operation of cutting out that a crack was observed which cut the prism into two parts 29 and 10 inches long respectively. The operators thought that this crack was the result of the cutting out of the reinforcement.

Non-reinforced concrete of the same quality as the above prism does not appear to undergo without breaking an elongation of more than 0·008 per cent., or about one-eighth of the elongation undergone by prism No. 2 without breaking. The two portions of this prism were subjected to bending, and gave tension resistances calculated by the usual formulæ of 125 and 133 lbs. per square inch, figures which are not far removed from the initial tension resistance of the concrete employed, and which had not been previously stressed.

Reinforced concrete should thus take without breaking, and at the same time conserving a large part of its resistance, elongations much superior to those demanded of it in structures; but it is also evident that cracks might in certain places be present beforehand or develop during loading of which it will be necessary to take account in the calculations of resistance.

3°. *Law of Deformation oj Reinforced Concrete.*

Measuring from the different graphs the tension produced by the concrete, which is indicated by the intercept between the graph and the line FF', we find that for elongations of 0·004 per cent., 0·02 per cent., 0·06 per cent. to 0·135 per cent., the tensions in the concrete were 170, 213 to 242 and 200 to 185 lbs. per square inch.

During the elastic period, corresponding to an elongation of less than 0·004 per cent., the tension had values growing regularly till a limit of 170 lbs. per square inch had been reached, which would be the limit of the resistance to tension if there were no reinforcement.

As the elongation increases from 0·004 per cent. to 0·02 per cent. the tension increases till it exceeds from 30 to 40 per cent. the resistance to tension of non-reinforced concrete. This increase disappears on further increasing the elongation to 0·06 per cent. approximately, and afterwards the tension remains constant and at the value of the resistance of non-reinforced concrete until a much greater elongation, about 0·135 per cent., was realised.

If for simplicity the temporary increase of resistance above referred to is neglected, it might be said that the modulus of elasticity of reinforced concrete is the same as that of non-reinforced concrete, so long as the applied stress does not exceed the resistance to tension of the latter. When that tension is exceeded the modulus of elasticity is reduced to zero, and in the places where there are no cracks the help it gives to the reinforcements is not zero but constant. The variations of tension in the reinforcements during this period are equal to the total variations of tension in the prism, since the resistance of the concrete remains sensibly constant. They are equal to the product of the variations of their lengths by the coefficient of elasticity of the metal, and might thus be calculated whether there are cracks in the concrete or not.

4°. *Loading and Unloading.*

As will be observed from Fig. 28, the unloading curve is very different from the curve of first loading. It is almost rectilinear, and presents inclinations slightly greater at the ends than elsewhere.

The curve of reloading differs little from that of unloading; the slight curvatures at the ends are in opposite directions. The mean modulus of elasticity is the inclination of a straight line through the extreme ends of these lines, and the greater the previous elongation the lower the value of the mean modulus.

5°. *Effects of Tension on the Resistance and Elasticity of Concrete to Compression.*

In some structures, *e.g.*, arches, the concrete might be submitted alternatively to tensions and to compressions, and it is important to know the influence that elongations exceeding the elastic limit exercise on the properties of the concrete from the point of view of compression.

A portion of prism No. 1, which had undergone the considerable elongation of 0·0135 per cent., was submitted to compression. It was found that its resistance to compression, which before the tension stress was applied was between 1,800 and 2,000 lbs. per square inch, had fallen to between 1,570 and 1,650 lbs. per square inch. The modulus of elasticity was reduced from 1,900 tons per square inch to 950 tons per square inch.

These results are reassuring, because the diminutions of resistance and elasticity in question, of which the former is small and the second presents no danger, except from the point of view of buckling, were produced by an elongation five to eight times greater than that experienced in structures.

6°. *Effect of Repetition of Loading.*

Prism No. 3 was submitted to the application, repeated twenty-five times, of a tension of 3,950 lbs., and it was observed that towards the end of the experiments the deformations no longer sensibly increased. By the method already described of measuring the ordinates from the graph to the line *FF'*, it is observed that the tension in the concrete is reduced from 235 to 185 lbs. per square inch—*i.e.*, to 80 per cent. of its initial value.

The tests made at Quimper showed that the final value was reduced to 0·70 of the initial value after many thousand repetitions.

7°. Final State after Unloading.

This state ought to be considered from two points of view, of which the first throws light on the second :—

1°. The reciprocal reactions of the concrete and the metal.

2°. The final deformations of the metal.

In order to determine by means of the graph of prism No. 3 the state of interior equilibrium which complete unloading would produce, the curve of unloading HA must be continued to B. The intercept between B and FF' gives the tension in the concrete, which divided by the area of the reinforcements gives the unital compression in the latter.

It is evident that even after elongations of 0·03 to 0·04 per cent. the reinforcements still retained a small amount of the initial compression imposed on them by the concrete.

In prism No. 2, after an elongation of 0·061 per cent., the original interior stresses were annulled, and stresses of a contrary sense and of small amount were produced.

In prism No. 1 the interior state of equilibrium was determined by measuring one of the reinforcements which projected beyond the ends of the prism, then releasing it from the concrete and measuring it again. The shortening observed was 0·01 per cent. It follows that after this prism had undergone the considerable elongation of 0·135 per cent., the original compression in the reinforcements was replaced by a tension of $0{\cdot}01 \times 100 \times 29{\cdot}1 \times 10^6$—*i.e.*, 2,910 lbs. per square inch—which equilibrated a compression of 33 lbs. per square inch in the concrete.

The inference from these results is that the interior reactions produced by the contraction of the concrete are reduced by the effect of the deformations, and may even be replaced by reactions of a contrary sense when the elongations exceeded a limit of from 0·05 per cent. and 0·06 per cent., which is greater than the elongations met with in actual structures.

This fact agrees with the alteration of the elasticity of concrete by large elongations. It is, of course, in virtue of its elasticity that concrete reacts against the reinforcements either in tension or compression, and it follows when this elasticity is reduced the property previously possessed by the concrete of imposing stresses on the reinforcements is diminished or lost.

It is evident that, since the effects of large but not excessive tensions is to allow the reinforcements to return to their normal length, the contraction of concrete kept in air imposes strains of the same order on the reinforcements as the external tensions.

Where the concrete has been immersed in water, the deformations due to the contraction of the concrete are much less than when the concrete is kept in air.

The Quimper experiments confirm the above facts. It may be mentioned that the average reinforced concrete used in construction is not submitted to elongations greater than from 0·015 to 0·030 per cent.

B. Bending Experiments in large Reinforced Concrete Beams.

All the beams considered here are 13·12 feet long, 15·75 inches deep and 7·87 inches wide. The *résumé* of the methods and results of the experiments are given in Chapter VI., section 9, on pp. 30—41, and only the main features of the results and the deductions from them are considered here.

1°. *Initial State.*

The contraction of the reinforcing bars, 0·874 inch diameter, was observed during the setting of the concrete by measuring the over-all length of one of the reinforcements from time to time. The reinforcement showed a progressive shortening averaging 0·0264 per cent. on the length of the bar, and since the contraction at the ends was necessarily zero, the maximum shortening which would occur at the middle would not be less than 0·031 per cent. Further contraction took place after these results were recorded, so that it is probable that the shortening of the reinforcement exceeded considerably 0·031 per cent. at the middle of the span when the tests were made.

As a check this figure was also arrived at by an entirely different method. The permanent elongation sustained by the reinforcement during the test was measured, and was found to be 0·015 per cent. at the centre of the span. The reinforcement was freed from the concrete and a further elongation of 0·033 per cent. was recorded, making a total of 0·048 per cent. in the middle part of the beam. Thus, making allowance for unavoidable inaccuracies, it is concluded that for concrete of these proportions the contraction due to setting in the middle part of a beam reinforced as described would be from 0·035 per cent. to 0·040 per cent.; and if the metal had a modulus of elasticity of $31{\cdot}3 \times 10^6$ lbs. per square inch, a compressive stress of about 11,000 lbs., or about 5 tons per square inch, would be produced.

This figure is much greater than that found for the stress in the tension test pieces. The difference is partly due to the greater dimensions of the beam and partly to the better quality of the concrete. It is evident, of course, that the tension a concrete might exercise in setting would be proportional to its resistance.

2°. *Deformation of Plane Sections.*

Four beams were studied from this point of view, two without visible fissures and two having the tension part of the beam cut through by the interposition of a double sheet of tin foil.

In all the beams, slit or otherwise, the indications of the apparatuses measuring the deformations of the concrete gave, when plotted on a diagram, almost rigorously straight lines, conformably to the hypothesis of the conservation of plane sections during bending.

In the beams not slit, the elongations of the reinforcements agreed also almost rigorously with the same hypothesis; that is to say, were sensibly equal to those of the layers of concrete at the same distance from the neutral axis of the beam as the reinforcement.

It was otherwise, however, for the slit beam, where it was found that slipping of the reinforcements was taking place towards the middle of the span, and gradually increasing with increasing bending moment. In the part of the beam under observation, owing to the method of applying the loads, there were no shearing stresses.

The observations made showed very considerable warping in different parts of the beam.

3°. *Position of the Neutral Axis.*

It is important to compare the observed positions of the neutral axis with those given by the formulæ based on the hypothesis of the conservation of plane sections, and on the application to flexion of the laws of deformation of reinforced concrete.

From the tension tests previously dealt with it is observed that the deformation of concrete in tension presents two phases separated by a short transition period.

In the first, which corresponds to very small elongations, the concrete behaves as a perfectly elastic body. In beams reinforced only on the tension side, such as those under consideration, the neutral axis ought to be situated a little above the middle of the height, in virtue of the superiority of the elasticity of the metal over that of the concrete which it replaces.

In the second phase, the tension of the stretched concrete remains constant. The position of the neutral axis corresponding to the additional deformations produced during this phase ought thus to be the same as if the tension of the concrete was zero.

It is given by the formula :[1]

$$x = 1 + n - \sqrt[2]{(1 + n)^2 - 1}.$$

when x is $\dfrac{\text{distance from centre of tension reinforcement to neutral axis}}{h}$;

h, the distance from centre of reinforcements to the upper surface of the beam;

n, $\dfrac{mw'}{bh}$;

b, the width of the beam;

w', the section of the tension reinforcements;

m, the ratio $\dfrac{\text{modulus of elasticity of tension reinforcement}}{\text{modulus of elasticity of concrete in compression}}$.

To calculate the position of the neutral axis in the beams experimented on, it is necessary to introduce the value of m appropriate to the concrete employed.

The mean value of the modulus of elasticity of a test prism made of the same concrete was equal to $3{\cdot}41 \times 10^6$ lbs. per inch^2 when the compression was between 570 and 1,420 lbs. per square inch, as these were stresses approximately marking the second phase of the bending in question. The modulus of elasticity of the steel was known to be $32{\cdot}5 \times 10^6$ lbs. per square inch. m therefore had the value 9·5.

Table No. 26 affords a comparison of the positions of the neutral axis as observed and as calculated for the various percentages and for the variations of bending moment set out in the Table.

TABLE No. 26.

[2] Reference letter.	Percentage of Metal.	Variations of the Bending Moment.	Distance from the Neutral Axis to the Centres of the Reinforcements.	
			Actual	Calculated.
		Foot-tons.	Inches.	Inches.
G.	0·50	7·56 to 13·05	10·709	10·630
F.	0·98	7·56 to 14·90	9·134	9·410
A.B.	1·94	7·56 to 16·73	7·795	7·953
C.D. (slit)	3·14	8·83 to 19·40	6·614	6·772

[1] This formula applies where there are tension reinforcements only.

[2] See Table No. 12, pp. 32, 38, for particulars of beams.

It is seen that the results of experiment accord closely with the values calculated according to the hypothesis of the conservation of plane sections applied to the phase in which the tension is zero, and that even for a beam cut by a transverse fissure and notwithstanding the slipping of the reinforcements.

4°. *Law of Deformation of Reinforced Concrete.*

Table No. 27 shows the results of some calculations made on one of the beams to verify the law of deformation obtained by the direct tension tests.

The bending moments applied to the beam are given in column 1. The elongations of one of the reinforcements, which were measured directly and without possibility of error, are given in column 2; in column 3 the unital stress obtained by multiplying the observed strain by the modulus of elasticity of the metal, $32{\cdot}4 \times 10^6$ lbs. per square inch; and in column 4 the total tension supplied by the reinforcements, the area of which was 2·4 sq. ins. In column 5 the distance from the centre of the reinforcements to the centre of compression, determined from the calculated position of the neutral axis, is given. In column 6 the moments of resistance obtained by multiplying the figures of column 4 by those of column 5 are given.

By subtracting the moment of resistance due to the reinforcement from the total applied bending moment, the moment of resistance due to the tensions in the concrete is obtained. These are given in column 7.

TABLE No. 27.

1	2.	3.	4.	5.	6.	7.
Total Applied Bending Moments. (Foot-tons.)	Percentage Elongation of Reinforcement.	Variations of Tension of the Reinforcement.		Lever Arm.	Variations of Moment of Resistance.	
		Tons per Square Inch.	Total Tons.	Ins.	Due to Metal.	Due to Concrete in Tension.
					Foot-tons.	Foot-tons.
3·43	0·00365	0·53	1·26	11·496	1·21	2·22
7·09	0·01030	1·49	3·59	11·811	3·54	3·56
12·60	0·02725	3·95	9·47	12·126	9·58	3·03
18·08	0·04350	6·10	14·63	12·165	14·85	3·24

Similar results were found in the cases of the other beams tested.

It will be observed that the moment of resistance produced by the stretched concrete increases at first very rapidly so long as the deformation is small. After attaining a maximum value it diminishes slightly, and thereafter remains sensibly constant. These facts agree well with the inferences drawn from the direct tension tests. In bending, there is not observed immediately beyond the limit of elasticity the transient augmentation of resistance noticed in the direct tension tests. This is explained by the fact that the various layers of concrete in tension are not equally elongated at the same instant; in fact, most of the layers are not stressed at all beyond the tension at which the modulus of elasticity becomes constant.

The final moment of resistance of 3·24 foot-tons supplied by the concrete corresponds to a tension of about 135 lbs. per square inch. This is the increase in tension caused by the mechanical test, and is additional to the initial tension resulting from the presence of the reinforcements. This latter tension might be put at 213 lbs. per square inch. Thus the total tension of the concrete during bending after the limit of elasticity had been passed would be about 350 lbs. per square inch, and would be constant at this figure.

5°. *Adhesion and Slipping of the Reinforcements.*

From the experiments of the Commission this very conspicuous fact emerges, that the extremities of the reinforcements are not sensibly displaced relatively to the concrete before the adhesion had been completely destroyed elsewhere, and also that in the neighbourhood of the application of concentrated loads there were important relative displacements.

Three different phases of the phenomenon united by short periods of transition were clearly marked in the bending experiments recorded on pp. 32, 38.

In the first, which extended to a bending moment of about 11·30 foot-tons and to a slipping of the reinforcement of 0·00113 per cent., the displacement of the reinforcement was small and increased proportionately to the load.

In the second phase, comprised between bending moments of 18 and 29 foot-tons and between slippings of 0·004 per cent. and 0·012 per cent., the slippings still increased proportionately to the load, but with a progression six or seven times more rapid than in the first phase.

Between these two phases of rectilinear displacement there is a curve of transition. In the third phase, which immediately preceded the failure of the beam by the destruction of the adhesion between the reinforcements and the concrete, the adhesion seems to have gradually disappeared outwards from the point of maximum stress over the whole length of the reinforcements.

Omitting this last period of doubtful and complex nature, it was found that at the end of the second phase the reinforcement had slipped 0·012 per cent. relatively to the concrete which surrounded it.

The deformation of the concrete was necessarily the greater as the reinforcement was approached, and from closely approximate calculations the maximum slipping of the concrete in contact with the reinforcement was about 0·6 per cent.

This high figure, which would have been rejected *à priori* as inadmissible before knowing the laws of the deformation of the concrete, appears within the bounds of probability, since it is known that concrete might without breaking undergo elongations greater than 0·2 per cent. when submitted to tensions or shortenings of 2 or 3 per cent. when spiralled and compressed.

This property which concrete possesses of slipping without breaking, appears to play an important *rôle* in reinforced pieces of permitting the distribution over great lengths of intense shearing stresses, which tend to be produced on certain points in the neighbourhood of concentrated loads.

In a beam on which the bending moment was increased till cracking commenced, then removed and re-applied thirty-nine times, it was found that, although the elongations and contractions of the concrete ceased to grow, the slipping was progressive. The load imposed on the adhesion of the concrete to the reinforcements in these repeated loadings thus exceeded the limit of elasticity, and even, it seemed, that of stability, although the bending moment was only half that of the moment which on its first application determined the slipping of the reinforcement in another identical beam.

Considered by itself, this fact is rather disquieting; but it is known from other sources that beams in which the adhesion has been highly stressed have supported a great number of repetitions of bending moments without the deterioration or deformation growing indefinitely. A prism tested at Quimper underwent more than 132,000 repetitions of a load which stressed the adhesion highly.

It would seem that the maximum stress imposed on adhesion by the application of a concentrated load is at first localised, and that it gradually propagates itself along the reinforcement till at no place its value exceeds what the concrete can carry without the slipping progressing indefinitely. Further experimental information however is required on this point.

In those beams tested up to the point at which slipping took place, an attempt was made to obtain information regarding the resistance opposed to the slipping of the reinforcement in bent pieces, but only indications without scientific precision are obtainable. In fact, the law of the distribution of the slipping stresses was ignored on the length of the reinforcements, and consequently no account was taken of the maximum values the stresses attained in the most highly stressed places. It was thus a purely conventional mode of calculation, to the results of which no scientific value is attributable, but which may be useful for the checking of design and the interpretation of the results of experiments.

If the concrete produced no tension, one of the components of the resisting couple would be furnished solely by the reinforcements, and the slipping tendency of the tension reinforcements would be at each point proportional to the shearing effort. In the beams experimented on as the loads were concentrated at about 3 feet from the abutments, the shear was constant between the point of application of the load and the abutment, and the total slipping tendency was thus equal to the maximum tension in the reinforcement, so that the slipping stress per unit area of the surface of the reinforcements was easily arrived at.

In this way a resistance to slipping of 185 lbs. per square inch was found for the beam with four bars each 0·874 inch diameter, and 213 lbs. per square inch for the beam with two bars each 1·57 inches diameter. In both cases the reinforcements were placed in the folds of vertical sheet-iron stirrups. An examination of the cracks led in both cases to the view that the displacement of the reinforcements may not have taken place by the failure of the resistance to slipping properly so called, but by the dislocation of the bed of concrete covering the reinforcements.

From experiments made by M. Mesnager on the tearing and driving of rods from cubes of concrete, values of from 190 to 235 lbs. per square inch were obtained, cracking of the concrete preceding or accompanying failure by slipping.

These facts show clearly that the values obtained by experimenting with rods buried in very large blocks of concrete which have shown resistances as high as 640 lbs. per square inch cannot be applied to the case of slipping in ordinary reinforced concrete where the section of concrete used is never large enough to prevent cracking before slipping.

6°. *Influence of Fissures on the Resistance of the Compression Areas.*

In order to determine whether the hamstringing that takes place in beams does not provoke the premature crushing of the concrete, two beams were manufactured with an artificial fissure extending across 65 per cent. of the section.

With a view to causing rupture by crushing of the concrete, the reinforcement consisted of two 1·57 inches diameter rods, giving the abnormal percentage of 3·14.

The beams, however, collapsed by the slipping of the reinforcements, and the only inference that could be drawn was that the compression in the concrete at the moment of collapse was not the maximum compression that might have been resisted. Assuming all the tension required was furnished by the reinforcements, the mean compression on the compression area was about 1,250 lbs. per square inch. If the elasticity had remained constant, and consequently the distribution of stress linear, the maximum intensity of stress would have been 2,500 lbs. per square inch. The elasticity, however, is reduced, but nevertheless the maximum stress should not be evaluated at less than 1,850 lbs. per square inch.

The resistance to direct crushing of blocks of the same concrete not reinforced did not exceed 2,000 lbs. per square inch. It would seem, therefore, that the fissuring which occurs in the sagging of bent prisms does not hasten in an appreciable manner the crushing of the part of the concrete subject to compression.

This fact is perhaps explained by the ductility that concrete presents from the point of view of slipping, and by the help derived by the compressed areas from the increased resistance to transverse swelling which precedes crushing.

C. Effects of Transverse Reinforcements.

1°. *Bending Experiments.*

The opinion has been expressed that in consequence of the inequality of the limits of elasticity which concrete presents after it has been submitted to tension or to compression, shear ought to produce a transverse swelling which would put a tensile stress on the vertical reinforcements, and thereby allow them to produce a useful effect in resisting the shear of the concrete.

In order to obtain information on this question the experimental method was resorted to. Measuring apparatuses were placed vertically on the lateral faces of two of the beams, with the usual vertical reinforcements. In one case the swelling was only 0·005 per cent., which represented a mean stress in the stirrups of about 0·6 ton per square inch. In another, in which failure took place by shearing, the swelling did not exceed 0·01 per cent. before the appearance of cracks, which indicates at the most a tension of 1·4 tons per square inch of the vertical reinforcements.

The measuring apparatuses were placed in vertical planes equally distant from the support and the point of application of the load, where the beam resisted both the maximum shear and a considerable bending moment. In the height of the beam on which measurements were made, there was comprised both tension and compression areas, so that the variation ascertained in the total height would result from the superposition of the effects of shearing in the whole section on the vertical contraction in the tension area, and on the vertical swelling on the compression area. It is thus certain that if the resultant vertical deformation in the whole depth of the beam was a swelling in the tension area, it was much less than the mean, consequently the help derived from the tension in the stirrups in this portion must have been very small. On the other hand, the vertical reinforcements have been able to furnish really important tensions in the compression areas, and consequently assist the lateral strength of the concrete.

When one recalls that it is in the tension area that oblique cracking due to shear commences, and that it is the longitudinal reinforcements which resist slipping, one is led to the conclusion that so long as the concrete is not cracked the stirrups hardly act by tension.

Theoretical considerations indicate that so long as the limit of elasticity is not passed, these stirrups hardly assist the shear, but the action thereafter assumes a great importance; in fact, well-placed stirrups may prevent the ruin of a structure after the concrete is completely cracked.

2°. *Torsion Experiments.*

The experiments above referred to attempted to elucidate directly the *rôle* of the transverse reinforcements in bent pieces, but no information was obtainable from them on the characteristic effects of pure shear. To this end M. Mesnager devised the torsion tests described on p. 27.

It was established that under a moment of torsion equal to one-half the breaking moment the elongation of the reinforcements was only such as produced a stress of about 0·60 ton per square inch, and although it is the case that shearing produces a lateral swelling of the concrete, it is so slight as to merit attention only under very heavy loads.

The vertical deformations produced by shearing add themselves to those resulting from the swelling of the concrete in the compression areas, and in consequence the reinforcements perpendicular to the axis there give tensions which increase the resistance to shear and to compression.

In the tension areas the transverse variations in volume produced by shear and by bending oppose each other, the effect of the former predominating at the supports, and of the latter at the places where the bending moment is a maximum.

D. Resistance of Reinforcements to Shear.

It is known that in imperfectly supervised work, and there is always some such, the adhesion fails occasionally between the ribs executed at first and the slabs executed later. It should be provided that the reinforcements traversing any such plane of separation should in themselves be sufficient to resist the whole of the horizontal slipping tendency.

The resistance of these reinforcements is complex, and comprises in the simplest case, which is that of vertical reinforcements, two elements—viz., the resistance of the metal to shear and the friction developed between the surfaces in contact. The last element is extremely variable, and in order to keep it as uniform a quantity as possible during the experiments, the sub-Commission has prevented adhesion by interposing sheets of oiled paper. These experiments are described on pp. 25, 26.

It is observed that whilst the percentages employed are nearly equal in order to equalise the tensions produced by the setting of the concrete, and consequently the friction developed between the surfaces of the concrete, the forms and dimensions of the reinforcements are very various.

The experiments have established the important fact that the resistances obtained relatively to the section of the metal have been sensibly proportional to the characteristic resistance of the metal; or, in other words, the shearing resistance was sensibly proportional to the area of the reinforcement and independent of its form.

The qualities of both elements in the test cube influence the strength of the whole, and one easily grasps the mechanism of this influence in examining the results of experiments.

The concrete crushed near the surfaces of slipping and the reinforcements took the form indicated in Fig. 29. They resisted thus not by shear alone, but by shear combined with bending to a short radius.

It is thus possible that the resistance of the reinforcements depends on the extent of the zone of crushing and on the quality of the concrete. The results obtained apply only to concrete containing 6 cwts. of cement to 14·35 cubic feet of sand and 28·7 cubic feet of gravel.

The graphic representation of the results obtained by measuring carefully the relative displacements of the two surfaces of contact indicates a point analogous to the elastic limit, beyond which the displacements grow considerably. This point is at a stress of about 8 tons per square inch of the section of the reinforcements, and the maximum load supported varied between 11·5 and 16·5 tons per square inch of the cross section of the reinforcements.

These figures indicate the effect of the quality of the concrete. Had the concrete been of sufficient quality to entirely prevent crushing, the reinforcements would have developed the ordinary resistance to shear. It is possible that the percentage of reinforcement used was a little too high for the quality of the concrete.

It is important to remark that the resistances obtained would not have been so large if the rods had been nearer the surface of the concrete, as the thin covering would have been more easily burst.

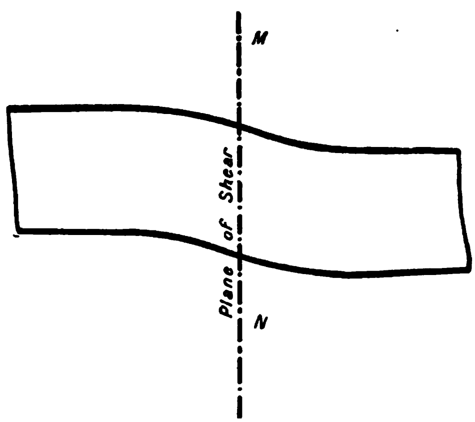


FIG. 29.

No experiment has been made on the resistance which proceeds from friction on the surface of slipping. The coefficient of friction of concrete on concrete lies between 0·60 and 0·75, so that the resistance given by friction might attain those proportions of the tension resistance of the reinforcement. It would not be advisable to add the whole or the greater part of this resistance to the bending-shear resistance due to the metal, because it is known that the superposition of different stresses on metal hastens its rupture.

E. Experiments on Ribbed Slabs.

The propriety of the application of the hypothesis of the conservation of plane sections during bending and of the exactitude of the formulæ which give the position of the neutral axis in rectangular pieces subjected to bending has been discussed on pp. 78, 79. A research was carried out on two ribbed T slabs with a view to finding out whether the same hypothesis and formulæ applied in that case.

Precautions were taken to eliminate, as far as possible, merely local warpings or irregularities, and to obtain a record of the deformations of the structure as a whole.

The salient feature of the results is their striking regularity, as plotted graphically on p. 47.

The uniformly distributed load which the first slab could carry without stressing the reinforcements beyond 6·3 tons per square inch was about 6·4 tons. Up to this load, the deformations were sensibly plane. Beyond it, however, slight deviations were observed. In the second slab the load was applied by means of a balanced platform to the upper surface of the slab immediately over the rib. The deformations were perfectly uniform for loads up to 9 tons; but beyond that load, and largely exceeding the load which would be applied in practice to a similar slab, plane sections showed sensible warpings in the opposite sense to those observed in the first slab.

Comparing these results with those of the tests made on the floors constructed at the Paris Exhibition, the conclusion may safely be drawn that in ribbed slabs plane sections remain plane during bending, any deviation from the plane being of so slight magnitude as not to merit attention.

The determination of the position of the neutral axis in any structure is the key to the correct calculation of the stresses in the concrete and moment of resistance of the member.

The measurements taken on the slab 4 feet wide show that the neutral axis occupied a position 2·24 inches below the upper surface of the slab. In the slab 6 feet 6 inches wide the neutral axis was lower under light loads, but afterwards rose to and retained till the end of the experiment a position sensibly identical to that in the first slab. The ratio x of the distance of the centre of gravity of the tension reinforcements from the neutral axis, to its distance from the upper surface of the slab, is thus 0·70, and assuming the efficient width of the slab to be 90 per cent. of its total width, and putting in the known data in the expression given on p. 97 for the value of x the value of m, the ratio of the coefficients of elasticity of steel and concrete is found to be 10·9. From this figure and from the known coefficient of elasticity of the steel used, the modulus of elasticity of the concrete is calculated at 1,270 tons per square inch.

The modulus of elasticity of this concrete was also obtained experimentally by the compressive tests of two prisms, the mean values up to a stress of 1,420 lbs. per square inch being 1,274 and 1,175 tons per square inch respectively.

The experiment in question has thus verified as exactly as possible the formula which determines the position of the neutral axis, and which translates algebraically the hypothesis of the conservation of plane sections and of the constancy of the tension in the area submitted to elongation.

Participation of Slabs in the Resistance of the Ribs.

The deformations were measured not only in the ribs, but in the slabs at points removed from the ribs, and it was found that in the slab 4 feet wide the neutral axis rises higher in the slab the further away from the rib the measurement is taken. That is, of course, as is to be expected, since the slab acts in two ways—viz., as a part of the compression area of the rib and as an independent beam, and in proportion as the latter action preponderates the neutral axis rises, in order that the necessary tension resistance may be obtained.

The neutral axis remained sufficiently close to the lower surface of the slab throughout the test that the tension was neglected, and account only taken of the compressions in the calculation of the resistance of slab and rib. Now, whatever the value of the coefficient of elasticity, these compressions are proportional to the products obtained by multiplying the thickness of the slab situated above the neutral axis by the shortening of the upper fibre of the slab.

The sections in which the three apparatuses were placed—viz., on the axis of the rib, at 13·40 inches and at 22·44 inches from it—gave products in the ratio 21, 20 and 17·6 respectively. The geometric mean of these figures is 19.

Consequently in the slab in question, in which the ratio of width of slab to span was 0·42, the slab gave $\frac{19}{21} = 0{\cdot}90$ of the resistance it would have furnished if in all its width it had been absolutely rigid with the rib. So that to calculate the resistance of this floor one might consider the slab as entirely solid in the deformation of the rib on condition of taking account of only 0·90 of its width.

In the slab 6 feet wide the ratio of breadth to span attains the exceptional figure of 0·70, and by making the above assumptions—which do not apply with so great accuracy in this case as in the former—the participation of the slab in the resistance of the rib was 0·55 of what it would have been in the case of absolute solidarity.

To determine as a function of the span taken as unity the effective widths of the slabs corresponding to spacing of beams of 0·40 and 0·70, it is necessary to multiply these figures respectively by 0·90 and 0·55. The products 0·36 and 0·385 are thus obtained. These results appear somewhat anomalous, as it is improbable that the useful effect of a slab increases only in the slight proportion of 0·36 to 0·385 when its width grows in the proportion of 0·40 to 0·70, although it should be remarked that the influence of secondary bendings rapidly becomes more important with the increase in width.

One is led to the conclusion by the consideration of these and of other experiments that, when the width of the slab is 0·40 of the span, its effective width may be taken about 90 per cent. of its total width, and that the help it gives the rib grows more and more slowly as the width of the slab becomes greater.

F. Variations in Length of Bars of Metal Buried in Concrete during Setting.

On p. 23 the experiments carried out to throw light on the above question are described.

Table No. 28 gives the results found.

TABLE No. 28.

Percentage.	Pieces kept in Water.		Pieces kept in Air.	
	Tension in the Metal.	Compression in the Concrete.	Compression in the Metal.	Tension in the Concrete.
	Tons per square inch.	Lbs. per square inch.	Tons per square inch.	Lbs. per square inch.
0·23	1·40	7·1	2·51	12·8
0·49	1·40	15·2	1·95	21·3
1·00	1·27	28·5	1·68	37·5
1·44	1·12	36·0	1·68	54·0
1·96	1·12	49·0	1·68	73·0
2·96	1·12	57·0	1·68	111·0
4·00	1·27	114·0	—	—
4·93	1·12	123·0	1·68	185·0
9·00	1·27	256·0	1·40	282·0

It is seen that for cylinders kept in air there were found compressions in the metal which attained in the length of 19·68 inches a value of 2·51 tons per square inch for a percentage of 0·23, 1·95 tons per square inch for a percentage of 0·49, and which starting from 1·0 per cent. up to 9 per cent. remained sensibly constant and in the neighbourhood of 1·68 tons per square inch. The tension in the concrete balancing the above has varied almost proportionately to the percentage from 12 to 280 lbs. per square inch.

In the prisms kept in water the reinforcements took tensions almost independent of the percentage of reinforcement and of about 1·2 tons per square inch, whilst the opposing compression in the concrete varied from 7 to 256 lbs. per square inch.

The experiments appear to establish the surprising fact that when the percentage of metal exceeds 1 per cent. an increase in percentage has no sensible effect on the stresses set up in the reinforcement as a result of the setting of the concrete, and that the corresponding stresses in the concrete are *quasi* proportional to the percentage, whatever its value.

The experiments just described cannot be considered applicable to large pieces. Since the production of the stresses depends on the adhesion, their values are necessarily zero at the extremities, and so much the greater on the average, as the pieces are longer in relation to the transverse dimensions of the reinforcements up to a certain limit far removed.

Experimental proof of this can be had by referring to the internal stresses measured in other pieces—*e.g.*, in the tension prisms 6·56 feet × 3·94 inches × 3·94 inches, the compression of the reinforcements was about 2·92 tons per square inch. In the beams of 13·12 feet × 15·75 inches × 7·87 inches, when the concrete had set freely and unloaded in air the compression was about 5·1 tons per square inch, and was very much more in the compression reinforcements when the concrete of the beam set under a heavy load.

The incontestable result of this series of experiments is to demonstrate that to obtain figures applicable to structures concerning the effects of the variations in volume of concretes or mortars, the surest method is from the results of similar pieces. If smaller pieces are employed, it would be necessary to establish between the transverse and longitudinal dimensions of the reinforcements proportions analagous to those met with in structures.

G. The Influence of the Proportion of Gauging Water on the Strength and Elasticity of Concrete.

It is known from experiments made elsewhere that the marked inferiority of very wet mortars and concretes diminishes with time, but the experiments of the Commission show even after ninety days' setting that the resistance and elasticity vary with the proportion of gauging water.

Compression tests after ninety days on similar prisms 14·2 inches × 2·85 inches × 2·75 inches, composed of 6 cwts. of Portland cement, 14·35 cubic feet of sand and 28·7 cubic feet of gravel were made.

The first series were gauged with 8·8 per cent. by weight of water. There was obtained a resistance to crushing of 1,850 lbs. per square inch, and a coefficient of elasticity of 2,000 tons per sq. in. between stresses of 280 to 850 lbs. per square inch, and 1,160 tons per sq. in. between 1,280 and 1,700 lbs. per square inch.

The second series were gauged with 11·0 per cent. of water. A resistance of 710 lbs. per square inch was obtained, whilst the coefficient of elasticity between stresses of from 140 to 280 lbs. per square inch was 1,040 tons per sq. in., and 420 tons per sq. in. between stresses of from 560 to 710 lbs. per square inch.

H. Effects of Direct and Repeated Shocks on the Adhesion and Resistance of Concrete.

Fears have been expressed regarding the disintegration of reinforced concrete in situations where it is exposed to repeated and violent shocks.

Several sleepers in reinforced concrete were put into operation in April, 1893, on the main down line from Paris to Granville at the entrance to Dreux Station, and remained there till June, 1898. The road was of 78 lbs. per yard, double headed rails, and the maximum speed at this place about 40 miles per hour.

One of the sleepers which had been placed near a rail joint was examined by the second sub-Commission.

The composition of the concrete was not known, but from its appearance it consisted of a rich mixture very carefully made.

The two features under special examination were the adhesion of the concrete to the reinforcements and the resistance of the concrete. From both points of view the results were remarkable.

To measure the resistance of the adhesion parts of the reinforcements were torn from the concrete, and gave very high figures, varying from 820 to 1,310 lbs. per square inch of the surface of contact.

This whole resistance was not, of course, due to adhesion properly so called, but even making allowances for the other resistances, such as that due to the straightening of the wires and the purely frictional grip of the concrete, the values are still high. A careful examination of other parts of the reinforcements showed that the adhesion was perfect throughout.

The resistance of the concrete to compression in 3-inch cubes without reinforcement was 8,410 lbs. per square inch.

It is thus shown that a concrete sleeper, without doubt exceptionally good to begin with, suffered no deterioration in five years' use. To appreciate the importance of this demonstration account must be taken of the great rapidity with which the effects of shock diminish as the distance from the point of impact increases if the transmission of the stresses is not made by absolutely rigid and continuous pieces.

There is thus a considerable difference between the sleepers which receive directly the shocks from the chairs and the floor girders, and *à fortiori* the main girders to which the shocks are transmitted indirectly, at least in large works. Where ballast is used this difference is considerably augmented.

The experiment of Dreux should remove all apprehension for the duration and conservation of bridges or viaducts carrying roadways or railways.

APPENDIX

PART I

VERTICALLY IMPOSED LOADS AND WIND LOADS ON METALLIC BRIDGES AS DEFINED IN THE REGULATIONS OF AUGUST 29, 1891[1]

Vertically Imposed Loads.

Bridges on Railways of Normal Gauge.

The type train will consist of two engines with their tenders at the head of a train of loaded waggons, and will have the following weights and dimensions:—

—	Engine.	Tender.	Waggons (Loaded).
Number of axles	4	2	2
Load per axle (tons) . . .	13·88	11·82	7·88
Distance from leading buffer to first axle (feet)	8·52	6·56	4·92
Spacing of axles (feet) . . .	3·94	8·20	9·84
Distance of rear buffer from last axle (feet)	8·52	6·56	4·92
Total weight (tons)	55·52	23·63	15·75
Total length (feet)	28·86	21·32	19·68

The engines, with their tenders, will be placed at the head of the train, and the maximum stress in each member will be taken as the maximum stress in it produced by any possible position of the type train on the bridge. For floor members, the stresses due to an isolated axle load of 19·69 tons will be taken as the maxima if they are greater than those produced by the type trains.

Stresses allowed; nett area in tension or gross area in compression:

For mild steel in main girders over 100 feet span	=	7·3 tons per square inch.	
For other girders	=	5·4	do.
For rail bearers and cross girders . .	=	4·76	do.
For web bracing exposed to alternating stresses	=	3·81	do.

Bridges on Railways of Metre (3·28 *Feet*) *Gauge and Over.*

The weight per engine axle is reduced to 3·00 tons $\times$ L, where L is the gauge in feet. The dimensions of the engines and the weights and dimensions of the waggons will be the same as for normal gauge, and the tenders will be supposed to have the same weights and dimensions as the loaded waggons.

For the calculation of the stresses under a single axle load, a weight of 4·2 tons $\times$ L, where L is the gauge in feet, shall be taken.

[1] "Résistance des Matériaux" (Vol. I., p. 595), Bibliothèque du Conducteur de Travaux Publics; publishers, Dunod et Pinat, Paris.

Road Bridges.

The stresses in each member must not exceed the limits above specified when either—

(*a*) a uniformly distributed dead load of 82 lbs. per square foot occupies the whole platform of the bridge, including the footpaths; or

(*b*) as many continuous files of tumbrils as the width of the roadway will hold, the footpaths remaining loaded with a dead load of 82 lbs. per square foot. Each tumbril to have one axle and to be drawn by two horses, all of the following weights and dimensions:—

Tumbrils.	Weight	5·91 tons.
	Length, not including the shafts	9·68 feet.
	Width of roadway occupied	7·26 feet.
	Width of wheel track	4·98 feet.
Horses.	Weight	0·69 tons.
	Length to each horse, including harness and shafts .	8·20 feet.

It must be shown that the stress in each member will not exceed by more than 0·64 ton per square inch the limits fixed for railway bridges, when a vehicle with one axle weighing 10·83 tons, and having the same dimensions as the above tumbrils, but drawn by five horses, is substituted for one of the tumbrils, and also in the case where the tumbrils are replaced over the whole platform of the bridge by waggons with two axles drawn by four pairs of horses, having the following weights and dimensions:—

Waggons.	Weight on each axle	7·88 tons.
	Length of vehicle	19·68 feet.
	Width of track	5·58 feet.
	Spacing of axles	9·84 feet.
	Distance of front axle to front of waggon . .	4·92 feet.
	Distance of rear axle to back of waggon . .	4·92 feet.
	Width of roadway occupied	7·38 feet.
Horses.	Weight of a pair	1·38 tons.
	Length of a pair, including harness and shafts .	8·20 feet.

Where the roads are steep and the loading defined above is not considered possible, either at present or in the future, the loads to be provided for may, with the consent of the Administration, be modified, but in no case will they be reduced below 62 lbs. per square foot for the dead load, and half the above live loads.

Bridges Carrying Canals.

These must be able to carry the load of water corresponding to the normal water level, increased by 12 inches, without the stress exceeding in any part the above limits.

Wind Pressure.

Railway Bridges.

The stress in the metal under the influence of the highest winds shall not exceed by more than 0·64 ton per square inch the maximum stresses already stated.

The maximum wind pressure will be taken at 55 lbs. per square foot; but it may be assumed that the passage of trains will be suspended when the pressure exceeds 35 lbs. per square foot.

It will be assumed that the maximum pressure will be exercised on the nett surface, after deduction of openings, of each of the main girders; that it will be applied to the total nett surface of the windward girder of each span; and that the wind pressure applied to the total nett surface of the adjacent main girder screened by the first will be reduced in the ratio of the nett surface of the first to the total surface bounded by its contour; and, further, if there be more than two main girders on each span, the effect on the others will be considered negligible. For metallic piers it will be assumed that the pressure will be exercised to the full extent on the nett surface of all members.

In considering the hypothesis of a train on the bridge, its vertical nett surface will be reckoned as a rectangle 9·84 feet high, its lower edge 1·64 feet above the rail and extending the full length of the bridge. From the area of this rectangle will be deducted the nett area of the windward girder, and it will be supposed that the wind action on the girder to leeward of the train is zero.

It must be shown that the tendency to transverse slipping and the upsetting action of the wind on bridge platforms and piers, assuming the train defined above to consist of empty waggons, and taking into account any special circumstances connected with the structures, cannot attain dangerous values.

Road Bridges.

The above methods of calculation must also be applied to road bridges, with the exception that no account need be taken of the presence of vehicles on the bridge.

Bridges Carrying Canals.

A wind pressure of 55 lbs. per square foot of vertical surface to be applied to the structure. The surface of boats exposed to the wind may be reckoned as a rectangle 4·92 feet above the side of the tank, and having the same length as the bridge.

PART II

REGULATIONS OF AUGUST 29, 1891, AS TO TESTS OF METALLIC BRIDGES (ABRIDGED)[1]

Railway Bridges.

EACH metallic span will be subjected to (*a*) dead load test, (*b*) rolling load test. These tests will be made by test trains at least equal in weight to the type trains previously defined in these regulations. For tests by dead load the test train will be placed successively in the positions producing the greatest stresses in the principal members of the bridge.

For separate span bridges the test train will be placed successively on each span, so as to cover it completely, then to cover only one-half of each span.

For continuous span bridges each span will be loaded by itself as just described. Then there will be loaded simultaneously the two spans contiguous to each pier, exclusively of all the others.

For arched bridges the whole span will first be loaded, then each half alone, and finally the middle portion, by placing there the two locomotives head to head.

For the dead load tests the test train will remain in each of those positions for at least half an hour.

The live load tests will be two in number—viz., at speeds of 12·5 and 25 miles per hour. The latter test may be postponed till the track has somewhat consolidated.

For double line bridges with both tracks on one platform, the tests will be made on each track separately as explained above, the other track remaining unoccupied; then on both tracks simultaneously. For the live load tests the trains will travel in the same direction.

For bridges of exceptional type, special arrangements will be made for the test.

Road Bridges.

Metallic spans will be subjected to tests of two kinds—viz., (*a*) dead load, (*b*) live load.

For the dead load test the surcharge will be 82 lbs. per square foot of the platform, including the footpaths.

For live loads the loads will be arranged as far as possible as to weight and spacing like the type load used in the calculation. They ought to give at least with their yokes a minimum load of 82 lbs. per square foot, taking 7·38 feet as the width occupied. The number of files of vehicles ought to be equal to the width of the roadway divided by 7·38; but in the event of it being difficult to assemble the full number of vehicles, the surplus width might be occupied by a dead weight of 82 lbs. per square foot distributed on each side of the files.

For dead load tests of separate spans the load will be extended from one extremity to

[1] "Résistance des Matériaux" (Vol. I., p. 595), Bibliothèque du Conducteur de Travaux Publics; publishers, Dunod et Pinat, Paris.

the other, with a stop of half an hour when the load has reached half span, and the full load shall remain on the bridge for half an hour.

For continuous bridges each span will be first loaded separately, as described above; then the two spans contiguous to each pier will be loaded simultaneously, exclusively of the others.

For arched bridges, each span will be loaded over the whole span; then on each half, and finally on the middle portion only.

The live load test will be carried out by causing the files of vehicles to pass at a walking pace from one extremity of the bridge to the other.

There will also be made to pass over the bridge a vehicle with an axle load of 10·8 tons.

When a reduction has been allowed in the loads used in calculation, the test loads will be correspondingly reduced.

Loads notably greater than the test loads will only be allowed to pass over the bridge by special authorisation.

Bridges Carrying Canals.

The tests of canal bridges will consist in the measurement of deflections after filling to a depth of 12 inches over normal water-level.

Measurement of Deflections.

There will be measured at the time of the test the maximum deflection at the centre of each span, under the influence first of the dead load and then of the live load in movement.

When there are several bridges of identical construction and of span not exceeding 33 feet, the measurement of deflection need only be made for one of them.

Immediately after the test of each bridge the metallic part shall be examined in all its details.

For bridges over 33 feet span two permanent bench marks will be fixed clear of every part of the structure. The levels of the lowest parts of the girders or arches at the centre and also at the extremities of each span and of the supports shall be recorded relative to these bench marks.

The detailed reports of the test made by the engineer in charge, in the form prescribed by the Administration, shall contain a description of these bench marks, which will enable them to be found at any subsequent time.

PART III

IMPOSED LOADS AND TESTS STIPULATED IN THE REGULATIONS[1] OF FEBRUARY 17, 1903, FOR METALLIC STATION BUILDINGS (ABRIDGED)

THE snow load used in calculating stresses is to be taken at 12·3 lbs. per square foot of horizontal surface, and the wind pressure at 31 lbs. per square foot normal to its direction, which is assumed to be at an angle of 10 degrees to the horizontal downwards towards the earth.

If a^0 is the angle of inclination of the roof, the action of the wind might be replaced by a vertical surcharge equal to 31 $\sin^2(\alpha + 10)$ lbs. per square foot of surface covered, and a horizontal thrust having the same value 31 $\sin^2(\alpha + 10)$ lbs. per square foot of surface in elevation.

The maximum wind might occur even after a fall of snow, but in that case the weight of snow may be considered as 6·2 lbs. per square foot of horizontal surface.

After erection one or several of the main trusses are to be submitted, as far as possible, to tests intended to demonstrate their resistance to forces analogous to those which they will be called upon to support. The results obtained by measurements during the tests will be compared with those furnished by the calculations.

For works constructed or contracted for by the concessionaires, the tests are to be made in the presence of the engineer charged with the control of the tests, or of an agent delegated by him. Detailed special reports of the tests are to be sent to the Administration, which reserves the right to make any modification in the regulations to meet exceptional cases.

[1] "Résistance des Matériaux" (Vol. III., p. 506), Bibliothèque du Conducteur de Travaux Publics; publishers, Dunod et Pinat, Paris.

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