# Pathology and therapeutics of reinforced concrete

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# PREFACE

The constructive study of engineering failures can often provide information of great value to the designer; indeed, in some cases, it can provide information that cannot readily be obtained in any other way. The investigation of failures is, therefore, a very proper part of building research; it is something to which DBR/NRC has accordingly devoted attention whenever suitable opportunities have presented themselves.

This analytical treatment of failures is still not a widely accepted part of the practice of engineering in North America, certainly insofar as the publication of such studies is concerned. In Europe, on the other hand, several volumes have been published, even in recent years, summarizing investigations of failures of engineering structures and pointing out the lessons in design and construction that can thus be learned. A recent example is an English translation of a Hungarian text.

The present translation presents an English version of another notable European volume, the second edition of a French study of reinforced concrete. It was brought to the attention of DBR/NRC by a friend of the writer, Dr. Jacob Feld, Consulting Engineer of New York, who has made a special study of engineering failures. Dr. Feld suggested that this translation should be prepared and has this comment to make on Lossier's work:

> "The Lossier book on concrete difficulties is a landmark in technology in that it teaches what not to do, both in design and in construction. A description of a successful operation is of limited value since the same design may not work under somewhat different conditions. A description of a failure is of greater value since it points up the conditions which can cause trouble, and how that can be avoided. In the large volume of present-day technical printed matter, there is little of such valuable description; one is almost led to the erroneous conclusion that there are no unsuccessful designs. This error brings confidence in procedures and acceptance of 'more modern' theories with reduced factors of safety, which also cover the factors of ignorance, and much too often with drastic results. The word 'failure' has lately lost its taboo in engineering discussions so that all can learn from the errors or mis-steps of the brave and honest men

who are willing to expose unexpected and unwanted incidents in their professional careers. Lossier has been a trail blazer in this effort to give instruction not (to follow the old saying) to those who cannot procure it for themselves, but so that others need not require experience to learn the lesson."

The Division is grateful to Dr. Feld for his interest in this matter and for his assistance in checking the translation; to the publishers of the original volume, Dunod, Paris, for permission to publish the translation in this way; and to Mr. D.A. Sinclair of the NRC translations staff for preparing the translation.

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# THE PATHOLOGY AND THERAPEUTICS OF REINFORCED CONCRETE

At various times, especially at the beginning of this century, public opinion has been aroused by the failure of structures in reinforced concrete, despite the very small number of such failures in relation to the number of perfectly satisfactory performances.

Thus, some people have wondered whether this manner of construction was not tainted by some sort of original sin to which such failures would have to be attributed.

To this specific question the answer must be an emphatic negative. Our knowledge of reinforced concrete, while doubtless it can still be enlarged, is nevertheless now generally sufficient so that any competent designer can produce with absolute safety not only building frameworks, but even bridges of very large span, of the type which would formerly have appeared to be the exclusive preserve of structural steel.

It would be just as wrong to give up the use of reinforced concrete because of a few accidents as it would have been to abandon structural steel construction after the collapse of the Quebec Bridge or any of the other disasters of which the memory is gradually fading.

Some may ask, however, why it is that if the properties of reinforced concrete are sufficiently well known accidents still occur from time to time.

I would reply that they can happen just as aeroplane and automobile accidents happen even with the best pilots and drivers, or just as there are still train wrecks even though everything appears to have been foreseen and checked. Accidents happen because unfortunately, as in every branch of human activity, there are fortuitous circumstances, builders or users of varying degrees of competence and sometimes of conscience, and finally, in rare cases, there is actual malevolence.

When we consider the chief serious or not so serious miscalculations that have occurred since reinforced concrete first came into use, we are surprised to discover that the underlying causes are comparatively few in number. Almost always the same errors recur in different forms.

Behind every error we generally discern a lapse of elementary common sense.

Now this common sense, which is one of the basic qualities of our race, has been subject to numerous lapses, which it would be useless to enumerate, in practically all fields since the recent wars.

As long as only fashion and the arts are involved, all that results, as a rule, is a bit of ridicule. However, when the strength of materials is in question the material and bodily consequences may be much more serious.

Too many young engineers, just out of school, believe they can ignore the advice of experienced practical men.

Now, while science can assist and complement experience, it cannot replace it entirely.

In saying that I have no wish to disparage the role of the technical knowledge acquired in the schools. On the contrary, it is my opinion that the studies of an engineer never go far enough, and in particular I deplore the fact that a technician may be handicapped in a project by the inadequacy of his mathematical skill.

However, the training of a man remains incomplete unless he has a clear idea of the relative importance which must be assigned to the various items of information that he has received. In particular he must realize that laboratory results are not exactly the same as field results, and that even the most accurate strength calculations are never more than an approximation of the true conditions.

He must understand that before discarding a practice established by our predecessors one must first have a thorough understanding of the reasons underlying this practice, and this always constitutes a profitable study.

I am above all an advocate of progress and of a constant effort to realize it. But I do not regard as progress the adoption of a method of construction or execution which has nothing to recommend it other than that it is different from conventional modes.

Originality in itself, without regard to any real economic or safety advantage, may indeed make good advertising copy, but it does not mean progress in the true sense of the word.

The training of an engineer should provide a knowledge not only of the best solutions that may be adopted, but perhaps also of practices that should be avoided. Each error in itself may be instructive, though often in an expensive way. Logically, the study of accidents and their causes ought to be part of the curriculum of our schools alongside the study of normal constructions.

The most frequent causes of accidents to reinforced concrete structures can be classified according to the following three main categories:

The design stage;

The formwork and centering;

The quality and placing of the materials.

We shall take up the question of foundations separately, since they have an importance all their own.

## A. DESIGN STAGE

There are three main elements in the conception of a plan:

1. Choice of type of structure;
2. Strength calculations;
3. Structural devices;
4. Precautions to be taken against certain external factors.

### 1. Choice of Type of Structure

In most cases the type of construction, whether residential or industrial, is generally dictated by the local or end use conditions. Thus, it is generally for projects of a monumental nature that a choice must be made between a relatively large number of solutions.

The primary determining factor is almost always the character of the underlying soil.

One of the most frequent errors which results in disappointments consists in the designing of a hyperstatic construction such as a fixed arch or a beam with continuous spans on a compressible soil, on the incorrect hypothesis of rigid supports.

On the other hand, an inadequate knowledge of the phenomena of shrinkage, flow or slow deformation under stress and adjustment has sometimes led designers to abandon the judicious solution of fixed arches in favour of articulated, statically determinate structures which were by no means obligatory.

Inadequate knowledge, or complete ignorance of the effects of oscillation or rough handling in the operation of travelling cranes and the vibrations of looms or machines that produce resonance in industrial buildings,sometimes leads to very serious miscalculations.

The choice of the type of foundation for an insufficiently studied terrain also plays an extremely important part in the pathology of constructions. There are many instances where floors, footings, or piles were apparently supported on a firm bed, but where there was an unidentified compressible layer underneath. In other cases piles have been driven into a soil which creeps laterally because of nearby dredging, etc.

Sometimes all that is needed is a thin, sloping layer of clay in a heterogeneous water-bearing soil to produce sliding and considerable thrust against retaining walls. Inadequate, or even non-existent drainage can also have serious consequences.

Let us consider several types of construction which are among those most frequently leading to ills of a more or less serious nature.

Floors keyed into walls

The evaluation of the degree of restraint enjoyed by floors keyed into walls, which some designers have a tendency to overestimate, also claims a number of victims.

When the ends of the slabs or beams are fixed in the walls, their moment of restraint, which depends on the characteristics of the design, is generally small. Giving them a value somewhat greater than their true one usually results only in a comparatively slight reduction of the factor of safety. In a number of cases, however, such overestimates have been sufficiently serious to produce real mistakes.

A typical case of this kind of error is that of a large building in Paris in which there are floors of 8 m single-bay span resting at both ends on the 0.22 m thick wall sections. As a result of an incomprehensible error these floors were calculated on the hypothesis of absolutely perfect fixity at both ends. Even a cursory examination would have shown that the bending moment at the supports was very small and therefore the moment at the centre of the span was about double the calculated stress. Because of this error the floors experienced a deflection under their own weight of about 1/50th of the span, so that the stress on the material considerably exceeded that allowed by the safety regulations even before the application of any useful load.

Besides this, the designer had included near the supports the reinforcements needed to balance a practically non-existent moment of restraint, so that he had not even saved any steel.

Now, in all cases where the restraint at supports cannot be evaluated with precision it is wise to assume first that the value of this restraint is zero, and then that it is relatively high; the concrete and steel cross-sections are then determined on the basis of each of these extreme hypotheses. However, in order to take into account the fact that the actual situation is somewhere between these two extremes, stress values somewhat above those required by the regulations can be adopted, while at the same time remaining within entirely safe limits.

Flat arches

A very special case is that of a flat arch consisting of a plate with a rectilinear extrados and a slightly arched intrados.

A few years ago the calculation of such plates was the subject of a great deal of discussion among foreign experts. Some regarded them as restrained beams of variable section while others considered them to be true arches with horizontal thrust.

Actually this distinction is practically, if not theoretically, pointless. If it be assumed that the rise of the intrados arch is comparatively high, and the stresses are determined by considering it as an elastic arch restrained at its extremities (i.e. assuming that the supports can resist a horizontal thrust without being deformed), we obtain a pressure line of approximately the shape shown in Fig. 1, the reactions of which slope towards the supports.

As the rise of the intrados is diminished, the reactions R approach the vertical and the horizontal thrust decreases. At the limiting case of rise to zero the two reactions are completely vertical and the horizontal thrust of the arch is zero.

There is thus no abrupt transition from the "curved" function to the "beam" function. For the very flattened members employed in designs of this type the results of the two methods are practically identical.

The above considerations, of course, assume a slab with standard reinforcement. If, as has sometimes happened, the reinforcements are incapable of resisting the imposed bending stresses, the slab will behave in the manner indicated in Fig. 2. At first it will behave like an arch restrained at both ends and will be subject to considerable tensile stresses at right angles to the supports on the extrados and in the centre of the span on the intrados; however, these sections, which are not sufficiently reinforced to withstand these stresses will crack severely. As a result of this self-articulation the slab then tends to become a true statically determinate arch which will hold up or collapse, depending on whether the supports can or cannot withstand the horizontal thrust without moving.

In view of the fact that in contemporary designs supports capable of resisting large horizontal thrusts without deformation are rare, it is generally wise to consider slabs of this type as beams of variable section, restrained at the ends or not, as the case may be.

<u>Single span arches and vaults</u>

The arch or vault is one of the commonest types of bridge used in reinforced concrete designs, since generally speaking it is the most economical. The few arch failures may be classified as follows:

Fixed arches on non-rigid supports. The use of an indeterminate, non-articulated arch when the abutments are not absolutely rigid is always a misconception. It is rare for an actual rupture to occur, since the arch articulates automatically as it cracks, but more or less serious damage may nevertheless result.

The use of provisional articulation or decentering by means of jacks is generally only a partial remedy, since shrinkage and settling of the abutments may sometimes continue for several years before stabilization is complete.

Arches on unstable supports. A typical case is that of a footbridge 42 m in span built over a canal about 20 years ago (Fig. 3). This bridge was supported on slightly inclined piles standing on a bedrock overlaid by an unstable silt. The slope of the reaction R of the bridge did not coincide with the slope of the piles, and since the silt was unable to prevent the rotation of the piles the abutments began separating at a continuously increasing rate as soon as the structure was decentered. The bridge collapsed completely after six days.

In such cases it is necessary either to employ massive, self-supporting foundations or abutments based on piles, the variable slope of which is able to contain the reactions of the arch under all actual load conditions.

Another example is that of a very flat static determinate arch with these articulations, one of the abutments of which underwent a 15 cm horizontal displacement resulting in a lowering of the key of the order of 45 cm (Fig. 4). Besides imparting an ugly appearance, the structure was weakened because of the reduced rise and the fact that the permanent pressure line no longer coincided with the axis of the arch.

These are in reality cases of foundation failure.

## Continuous multiple-bay arches

Arches without tie beams spanning several equal bays with intermediate supports consisting of simple walls or articulated columns able to sustain only strictly vertical reactions, were the cause, in a foreign country, of one of the most serious accidents in the history of reinforced concrete.

Arches of this type had been selected to cover a large reservoir and had to support a load of fill. Each arch had been calculated as a fixed arch on fixed supports at both ends, on the basis of the fact that the spans were identical and supported equal loads, and therefore the horizontal thrusts would be balanced at right angles to each intermediate support. Now, during the filling operation, instead of applying the surcharge of earth simultaneously in a strictly uniform manner to all the spans, the earth was spread only over a few of the arches.

Since the horizontal thrust of these arches was not balanced by that of the neighbouring ones and beyond, these first arches sagged and produced a complete collapse of the assembly in the manner represented in Fig. 5.

Basically, if these non-rigid arches are to be stable, they must satisfy the two conditions represented in Fig. 6. Firstly, there must be outside supports A and B capable of balancing the oblique thrust R without shifting; secondly, there must be a strictly uniform load on all the arches.

For safety's sake, however, one should always consider the possible consequences of failure to observe this latter condition for any accidental reason whatsoever (an error of workmanship, repairs, bombardment, etc.). In various instances the builders had been content to furnish the abutment spans only with tie-beams without paying adequate attention to the strength and rigidity of the end supports.

As a general principle when arches without ties, of unequal span or under different loads are involved, the problem becomes definitely more complex.

Formerly, for masonry bridges, builders were satisfied to consider each arch as being fixed on rigid supports without taking into account the elasticity of the other arches and of the piers. The latter were calculated to withstand the thrust of adjacent arches and these thrusts were determined in the first hypothesis. Although inexact in principle, this method was nevertheless admissible in practice, because of the low surcharges relative to the dead load on the masonry bridges of that time. Actually the calculated strains were too low for the arches and too high for the piers. Certain extrados cracks at the roots, sometimes attributed to other causes, seem to have been a direct consequence of these errors.

With the lighter reinforced concrete bridges and present-day heavy surcharges the problem has changed and must be considered more carefully. Let us consider two imaginary, extreme cases, namely that of a thin arch supported on comparatively rigid piers (Fig. 7) and that of a thick arch on piers articulated top and bottom (Fig. 8).

In the first case, if we neglect the elasticity of the piers, each span can be compared to a succession of independent spans, since each pier can withstand the difference in thrust from the adjacent spans.

In the second case only vertical reactions are exerted on the piers because the horizontal thrust, which is the same for all spans, is balanced by the terminal abutments.

Between these two extreme cases there are an infinite number of possible solutions. If we consider the simplified case of monolithic types without articulation (Fig. 9) where the division into spans and the height of the piers remains invariable, one may vary the rigidity of the various elements as long as one observes in each case the limiting values demanded by the regulations. Greater rigidity of the piers permits the use of lighter arches, and vice versa.

Since the unit costs of the materials employed are different for arch and piers there will be a solution in each case that will give maximum economy after the detailed estimates have been carried out. Sometimes the designer of a project is dismayed by the rigorous calculation of a continuous multiple-bay arch in reinforced concrete and therefore resorts to the simplified rules formerly employed for masonry bridges. This sort of indolence has been the cause of certain serious cracking in reinforced concrete arches built on comparatively high piers which have spread considerably under the surcharge of a single span, in the manner indicated in Fig. 10. Multiple-bay arches can be calculated in a manner that is both rigorous and simple by applying the ellipse of elasticity, either graphically or analytically or with the aid of an integrator. This method has the advantage of reducing the calculation of each span to that of a single span by including at the roots imaginary elastic elements, the deformations of which coincide with the piers of the other spans. Furthermore, the shape and sectional variation of the elements involve no difficulty in practical application*.

When it is a question of determining the dimensions of a structure very approximately, the approximation method based on the following simplified hypotheses may be employed (Fig. 11):

1. The ends of the arches and piers are displaced in a substantially parallel direction, i.e. without rotation.

2. The ends of the arches adjacent to the loaded span are fixed, which is tantamount to considering the work as a 3-span structure.

3. The axis of the arches is a parabola and their moment of inertia varies from key to roots in a ratio $\frac{ds}{dx}$, where ds is the length of an element and dx that of its horizontal projection. The moment of inertia I at a given point in the arch is then equal to $I = I_1 \frac{ds}{dx}$, where $I_1$ is the moment of inertia at the key.

4. The pier cross-section is constant over the entire height.

Using the following nomenclature:

a - semi-span of an arch;

f - theoretical axial rise;

h - height of piers;

$I_2$- moment of inertia assumed constant;

P - surcharge of a single span assumed uniformly distributed over the horizontal, the following is obtained:

The thrust R at each root of the loaded span breaks down into two forces, the horizontal thrust $R_1$ balanced by the adjacent arch

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* See "Théorie générale de l'arc élastique continu sur appuis rigide" by Henri Lossier in collaboration with A. Paris, Bulletin technique de la Suisse Normande, and "Les viaducs en arcs à plusieurs travées solidaires" by Henri Lossier, Institut technique du bâtiment et des Travaux Publics, June 5, 1941.

and acting at two-thirds the height of rise f, is given by

$$R_1 = H_1 = \frac{45\ I_1}{8 \cdot a \cdot f^2}$$

and $R_2$ passing through the centre $G_2$ of the adjacent pier, the horizontal component of which is given by

$$H_2 = \frac{12\ I_2}{h_3} .$$

The displacements of the tops of piers determines the secondary stresses on the loaded arch by reason of the elasticity of the whole assembly.

With these simplified considerations we have a means of comparing approximately and very rapidly various solutions with a view to making a basic choice, reserving the rigorous calculation for the plan finally decided upon. The effects of shrinkage, flow and thermal variations will not differ substantially from those pertaining to a single arch unless the spans are unequal.

Statically determinate beams

Either simple, or cantilevered statically determinate beams are necessary wherever there is a danger of substantial settlements of the supports, unless the settlements are exactly known in advance and are taken into account in the calculations. If there is a danger of such settlements being great enough to affect the appearance of the structure or to interfere with traffic then jacking devices should be provided so that at any time the deck can be brought back to its normal position, if possible without interrupting the passage of vehicles and pedestrians, although the traffic may be reduced if necessary for the time being.

Generally it is sufficient to provide recesses at right angles to the piers or abutments for the introduction of jacks and shims with removable platforms for the operators (Fig. 12).

In certain silty and clayey soils over comparatively great depth, settlements sometimes of the order of 1 m have occurred. However, in the case of structures of several spans with truly stable supports continuous decks are preferred, thus avoiding the many disadvantages of joints.

Continuous-span beams

Reinforced concrete continuous beams have been the subject of many discussions among specialists. Some are of the opinion that they really ought to be calculated as homogeneous beams, i.e. according to the classical formulae based on the theory of elastic deformations. Others believe they should be treated by special empirical methods involving only partial continuity.

Generally speaking, beams calculated and constructed on the basis of complete continuity are less susceptible to cracking than those calculated on the basis of partial continuity. Actually, since their reinforcements are determined so as to resist the tensile stresses both at right angles to the supports and in the sections between them the standard fatigue values of steel and concrete will only be exceeded slightly, if at all at any given point.

Partially continuous beams have frequently been calculated by the simplified formula $\frac{pl^2}{10}$ for the bending moment between the supports, only half of the rods determined in this way being retained over the supports. Under load the parts over intermediate supports tend to be subject to strains greater than those for which the reinforcement provides. As the result of the consequent excessive deformation the distribution of bending moments is modified and the weakened support areas seek relief at the expense of the stronger sections between them. A new state of equilibrium is then reached (sometimes at the cost of extrados cracking over a support) which tends automatically to approach the hypothesis that has been made without really endangering the actual safety except in the case of extremely variable positive and negative live loads frequently repeated.

Hyperstatic structures, like living organisms, can be said almost universally to possess the faculty of adapting themselves to any abnormal conditions that may be imposed on them as though to postpone as long as possible their ultimate failure. In practice the designing of a continuous span beam on the hypothesis of partial continuity does not result, in principle, in a pathological case, provided:

1. The relative weakness of the support areas is compensated by a corresponding excess of strength in the bays .

2. The difference between the values assumed for the moments and those regarded as occurring in practice remains within the limits normally accepted among engineers.

A few cracks that do not endanger the stability of the structure are generally the only detrimental consequence of this distortion of the classical theory. The main causes of miscalculation to which continuous-span beams are susceptible can be summed up as follows:

(a) Vertical displacement of the points of support. These may be permanent or elastic.

If permanent, defective foundations are almost always involved. Elastic displacement occurs in structures built on tall piers where the beams rest on sleepers, e.g. girders supported by bridge pieces, structures on floating supports, etc.

If sufficient account is not taken of these support displacements in the plans, as has often been the case, the neglected secondary stresses may cause the standard strains to be exceeded and results, if not in accidents, at least in more or less serious cracking and the reinforcement, moreover, will be poorly distributed with respect to the actual functioning of the structure.

Let us consider, for example, the case of a constant section beam resting on flexible piers of equal height equally distributed (Fig. 13).

Nomenclature:

I - moment of inertia of the beam;

E - coefficient of elasticity of the concrete;

l - distance between piers;

v - vertical displacement of the top of a pier under a unit reaction;

$c = \frac{E \cdot I \cdot v}{I^3}$ the characteristic of the system.

Let a single force P be applied at the centre of a span and let M be the bending moment under the load, $M_1$ the bending moment over each of the adjacent piers and $A_1$ the reaction of each of the latter. The following figures are obtained for various values of the characteristic c which, other things being equal, is proportional to v, i.e. the elasticity of the piles:

| C | M/Pl | $M_1$/Pl | $A_1$/P |
|---|---|---|---|
| 0.0 | + 0.171 | - 0.079 | + 0.600 |
| 0.1 | + 0.233 | - 0.017 | + 0.474 |
| 1.0 | + 0.370 | + 0.120 | + 0.319 |
| 2.0 | + 0.434 | + 0.184 | + 0.276 |
| 5.0 | + 0.539 | + 0.289 | + 0.225 |
| 10.0 | + 0.637 | + 0.387 | + 0.192 |

The graphs of Fig. 14 clearly show that the stress variations are particularly high for small values of c, the case where it may be dangerous not to take them into account.

(b) Resistance of piers to bending. When the beam is restrained on the piers the latter oppose the angular variations of its axis, thereby resisting bending stresses that are sometimes quite considerable. This reduces the transmission of the beam moments from one bay to the next.

Actually, structures of this type act as ordinary continuous beams if the bending elasticity of the piers is infinite and as a succession of fixed beams if this elasticity is zero, with an infinite number of intermediate cases.

The chief errors leading to miscalculations are almost always due to underestimation of the stresses due to shrinkage and thermal variations, on the one hand, and the effect of braking of vehicles or travelling cranes, on the other.

In addition, the reinforcement joining piers to beams has sometimes been inadequate for producing perfect liaison between them.

Vierendeel beams

Except for the rare cases of erroneous calculation most failures in structures of this type can be attributed to a poor distribution of reinforcement at right angles to the joints with outward tensile forces insufficiently balanced by stirrups.

In most cases the bending moments and shearing stresses due to the deliberate omission of diagonals have been underestimated.

Examples of uprights with shearing cracks due to insufficient reinforcement are rather frequent (Fig. 15).

Vierendeel beams must always be very carefully designed.

Bow-strings

The most difficult point to deal with in the case of bow-strings is almost always the joining of the arch to the tie-beam. We shall deal with this matter below.

Riprap dam shields*

The impermeability of riprap dams is generally assured by a shield of either reinforced or bituminous concrete or one of clay. In the first case the shield may either be placed against the upstream wall or sunk vertically into the riprap itself (Fig. 16, a and b).

Other things being equal, the latter type of shield is always in a more favourable position as far as the effect of riprap settlements and conservation of the concrete is concerned and is then protected from the direct effect of the sun when the dam is only partially submerged.

On the other hand, this type of shield increases the stress on the foundation soil.

Moreover, a serious accident has emphasized the advantage of hollow shields open to internal inspection by which it is possible

* See "Riprap dams" by Henri Lossier, Génie Civil, October 25 and November 1, 1930.

to guard against certain local damage without the necessity of fully or partially emptying the reservoir.

This statement, however, is subject to qualification in the light of a very considerable miscalculation which the author was required to remedy. Riprap is subject to settlements of between 0.3 and 2.7% according to observations on existing dams and as a consequence of this the upstream walls undergo deformations, the stresses of which are transferred almost fully to the shields (Fig. 17). If the latter is not sufficiently flexible to withstand these deformations without considerable tensile stresses in the concrete, cracks that endanger the impermeability and the preservation of the reinforcement may then occur. Thus, in order to make these hollow shields that are open to inspection as flexible as possible it has been suggested that they be built in the form of two walls which are united at the base and are joined elsewhere by struts leaving to each of the walls its own elasticity.

Finally abrupt variations in the longitudinal profile of the base of a shield should be avoided.

## 2. Strength Calculations

An excellent mathematician may still be a poor engineer, for in this field, as in many others, pure science is of little avail unless backed up by sound common sense.

We are confronted with two main elements, namely reality, which is unique and independent of our ideologies, and theory, which is human in origin and of arbitrary character and which seeks to give to reality a simplified and comprehensible form on the basis of which it is possible to reason and to formulate practical conclusions with a minimum risk of error. No theory coincides exactly with reality and the divergences between the two are the principal cause of most of the false reasoning indulged in by certain theoreticians. Who among us have not known such people enamoured of abstractions who, by identifying the approximate picture of theory with the object itself which is the reality, pursue the course of their reasonings indefinitely long after they have gone beyond the limits of their applicability.

In the case of structural steel the functioning of most structures depends on fewer factors than in the case of reinforced concrete, for structures in the latter are subject to the dual effect of monolithism and of variations in time of the properties of certain of their elements (shrinkage and flow of the concrete, relaxation of high-strength rods). This is why designers employing reinforced concrete are particularly exposed to the maltreatment of inveterate theoreticians.

Let us take as a first example the elementary case of a simple slab furnished with parallel, equally spaced ribs (Fig. 19).

How often have we not been told that a slab of this type must be calculated by strict application of the classical formula for the continuous beam of constant section resting freely on fixed supports? Yet actually (a) the slab does not have a constant section because it generally possesses a double reinforcement at right angles to the ribs and a single one between them and this has some effect on its elastic properties; (b) it does not rest freely on its supports because it is integral with its ribs which resist torsion; (c) its supports are not fixed because the ribs bend unequally under the action of the variable surcharges and moreover their flexibility decreases towards their extremities; (d) and finally the slab resting equally on the walls acts more like a monolithic plate.

The obvious differences existing between the required calculation and reality in such a simple, well-known case show how illusory may be the results of certain theoretical calculations which are unsuitable for quite common structures.

As a second example let us take the case of a bow-string bridge comprising (Fig. 20) an under-reinforced concrete arch and an over-reinforced tie-beam.

Shrinkage and flow, or slow deformation, of the concrete under stress have a much greater influence on the linear variations of the arch than on those of the tie beam. Thus the way in which the structure functions will change as the results both of the surcharges and the dead load from the time of its construction and during the years that follow. In other words, its functioning will vary in time within limits which, in the case of large structures, it would be wise to attempt to evaluate.

Single-span arched bridges have also suffered at the hands of bad theoreticians. I shall cite only a single example, which I experienced at the beginning of my career. This was a case of a competition abroad for a reinforced concrete bridge comprising several arches of equal span resting on very high piers.

Believing I was on the right track, I had calculated the structure on the basis of chance loads, taking into account deformations of the piers and arches by the method of the ellipse of elasticity. Compared with the approximation method, which consists in considering arches as being restrained on thick supports in all cases, this method, incontestably more accurate, led logically to a reinforcement of the arches and a lightening of the piles.

Now, the foreign professor who was responsible for checking the calculations violently opposed what he called a "heresy". When I attempted to demonstrate the principle with the aid of a scale model containing very deformable, elastic elements, he declared that "he would never believe that a model test could be used against the method which he had been teaching for many years".

This honourable professor is an example of a pathological technical specialization, and I try myself to avoid similar blind spots in dealing with my younger colleagues.

Let us now consider the coefficient "n". The value to be given this equivalence coefficient has been the occasion for much spilling of ink and for numerous long discussions.

This, let us not forget, is a coefficient by which the steel section must be multiplied in order to simulate equivalent, imaginary sections of concrete.

At the beginning there were at least two antagonistic, or at any rate diverging doctrines. Hennebique assigns an arbitrary useful strength value to concrete of 25 kg/cm$^2$, and to compressed steel of 1,200 kg/cm$^2$, which with certain reservations would yield an equivalence coefficient of 1,200/25 = 48, a record figure, to the compression.

The orthodox theoreticians, on the basis of the fact that concrete has a mean coefficient of elasticity of the order of 200 metric tons per $cm^2$, i.e. one-tenth that of steel recommended, on the other hand, a value n = 10, which became the basis of many regulations.

While this value may sometimes result in needless outlays for compression reinforcement, it must be admitted nevertheless that it has never resulted in any accident.

Later, in order to take into account various factors, especially the role of concrete in the tensile zone of the section subject to bending, the number 15 was adopted, without taking into account the formula deduced from actual tests by Mr. Caquot, which introduces in a logical way the reinforcement percentage.

This fixed number 15 is currently being employed.

I have often wondered how so many engineers can get enamoured of this coefficient n. Let us now consider the normal evolution of a compression unit in use. At first it may act more or less as though its reinforcement was comparable to concrete, their section being multiplied by $n = \frac{E_s}{E_c}$, where $E_s$ and $E_c$ are the respective coefficients of elasticity of the two materials.

Subsequently, under the dual action of hardening shrinkage and flow, the concrete as it gradually contracts will partially escape its initial state of stress, transferring its load to the reinforcement which is of substantially constant properties.

In other words, the coefficient n will increase from a virtual value of approximately 7 to 10, to one which in some cases may be more than three times this value.

Is it logical, under these conditions, to campaign vigorously, as I have seen done many times, in favour of a given fixed value which can only correspond momentarily to reality, and which, moreover, generally has little effect on the economy of the plans, except for the case of compression reinforcement, which, moreover, is rapidly being abandoned?

Certain theoretical questions even today are still the subject of serious discussions among builders and engineers who have to apply the official rules.

Now although the centenary of the introduction of reinforced concrete has already been celebrated one must nevertheless realize that there is still no practical agreement among the rules applied in various countries. What is even more serious is that the requirements of these rules sometimes vary even within a single country, depending on the administration imposing them.

This applies particularly to the figures on so-called "cracking strength of concrete due to shear".

The limiting stresses assumed in the calculation of reinforced concrete structures are of two kinds, namely those relating to the strength as such with respect to standard loads and surcharges, and those which are intended only to prevent the cracking of the concrete and are designated by the term "cracking strength". The former pertain to the working of the reinforcement either in tension or compression and of the concretes in compression, shear, cohesion, etc. The latter apply particularly to the cracking of the cladding of deformed rods or ties in hooped elements and the so-called shearing cracks in arched members, which are the subject of our observations here.

In Fig. 21 let

$T$ - total shearing stress in a section

$z$ - the lever arm of the resisting elastic couple

$b'$ - width of rib

$t_b = \frac{T}{b' \cdot z}$ - theoretical shearing stress of the concrete

$A$ - cracking stress or value of the theoretical shearing stress $t_b$ which must not be exceeded if cracking of the concrete is to be avoided.

One must thus have

$$t_b = A;$$

the stirrups and hooked bars being calculated, moreover, without discounting the actual tensile strength of the concrete and a shearing stress.

Now, depending on the country or the administration, the prescribed values of A may vary widely, other things being equal. Certain regulations neglect the role of hooked bars or inclined stirrups while others introduce the spacing of these elements in relation to the height of the beams and still others as a function of the rod diameters, etc.

Some regulations pay special attention to sections subjected simultaneously to bending moments and maximum shearing stresses.

Furthermore, some regulations relating to the reinforcements which join slabs to the ribs of T sections appear to conform poorly to the experimental results.

Without stressing the point too greatly, it must be recognized that in addition to a pathology of reinforced concrete there is also a pathology of official rules which is often troublesome to builders subjected to obligatory inspection.

This situation appears to be a temporary one. Contacts between engineers even of distant countries are becoming more and more frequent and cordial, and experimental programmes are everywhere in progress. This must inevitably lead to the harmonization of points of view that are now divergent.

Statically determinate and hyperstatic structures. If the calculation of general stresses in statically determinate structures (bending moments and shearing stresses) presents no practical difficulty, this cannot be said for hyperstatic structures the functioning of which depends on their elastic deformations. Let us consider, for example, a continuous T-section beam (Fig. 22). Over the supports the wide flange will be subjected to tensile stresses, while the narrow web is compressed (a). At the centre of the spans the reverse will be true (b). Despite the compensating effect of the reinforcement the coefficient of angular deformation of the sections will not be constant over the whole length of the beam. The variations of the moment of inertia to be introduced into the calculations, which are of considerable magnitude in large structures of widely varying section, is more or less uncertain. Shrinkage and flow become more and more important in the course of time.

Some builders consider only the whole or overall concrete section without taking the reinforcement into account. Others differentiate, from the standpoint of elasticity, between the concrete under compressive stress and that under tensile stress, taking into account reinforcements, etc.

In the absence of systematic, long-term experiments there will always be some gaps in our knowledge. Thus, despite the conditional and restricted assistance afforded by adaptation phenomena, one should be particularly cautious in calculating hyperstatic structures.

In certain cases, especially for thrust beams, it is wise to vary, within logical limits, the ratio of the moments of inertia of the various parts, so as to take into account increases of stress that may result from the uncertainties of the hypotheses contemplated.

A few words now on the theoretical accuracy of strength calculations. Some theoreticians try to work out their strength calculations to an impressive number of decimals without troubling themselves about the extent of their conformity to the realities. Now, although the experiments of the Austrian engineers, in particular, have demonstrated that the theories of the restrained elastic arch had practical justification not only for metal structures, but also for arches of concrete, reinforced concrete, brickwork and rubble work, nevertheless other tests, too often passed over in silence, have been much less conclusive.

This was particularly true in the case of certain reticulated structures for such massive works as dams, turbogenerator pedestals, etc. Divergences of the order of 20 to 30% and even more are not exceptional.

We have many times had to argue this point with engineers who demand of themselves, and what is worse, of us also, excessive arithmetical operations which, from the practical point of view, constitute pure illusions.

Some people, with more reason of course, place unlimited confidence in calculations based on the mathematical theory of elasticity. Even there, however, the basic hypotheses sometimes differ very appreciably from reality. For one thing, reinforced concrete is not a homogeneous material in all directions, and for another thing the errors of these hypotheses are compounded along with the standard factors.

To sum up, it must be realized that in most cases the behaviour of reinforced concrete cannot be theoretically defined with very great accuracy even by our most orthodox methods of strength calculation. It would therefore seem useless to get involved in interminable calculations and to seek an excessive numerical accuracy, or at any rate one very much beyond the practical margin of error which one may normally expect in any given case.

Hennebique has built numerous structures by applying a method which is sometimes regarded as heretical, not without reason of course, particularly since this method leads to two unequal components, tensile and compressive, in the balancing of a bending moment.

Nevertheless, no accident can be attributed solely to the use of this method, thanks in part to the splendid practical common sense of its author, and thanks also to the adaptation phenomena which compensate, although only in certain instances, some of the mistakes of the builders.

Research is now going on all over the world with a view to evaluating with certainty not only the stresses occurring under normal service conditions, but above all the breaking strength of reinforced concrete sections, which is of much greater importance from the practical point of view. This research will certainly contribute to the better proportioning of structures from the standpoint of their true safety.

I believe that any engineer worthy of the name must have sufficient mathematical training to be able to handle all the current theoretical difficulties. Once more, however, this training will be of no great value if he does not at the same time have as a guide and moderator one of the greatest qualities that a man may possess, namely, common sense.

Model tests

If all the linear dimensions of the concrete and the diameter of the reinforcements are reduced by the same proportion, no matter what, the theoretical stresses remain the same, provided the load per square metre remains constant. The bending moments, like the resisting moments, are then proportional to the cube and the shearing stresses and thrusts, like the surface sections, to the square of the dimensions.

In practice, it is obviously wise to control the gradation and the cement proportion in such a way as to obtain the same strength and workability of the concrete in the model as in the full-scale work. For gantries and thrust beams, a collection of flexible plates with rigid joints enables one to find the position of the bending points with the aid of a simple cursor.

By varying the elasticity ratios of the various parts of the model the order of magnitude of the errors deriving from the hypotheses and their effect on the results obtained can be determined.

## 3. Structural Devices

We shall consider a number of typical structures.

Silos

The most frequent failures in silos are the tearing out of the discharge openings and the cracking of the vertical walls.

Failures of the first type are rarely due to the sections of the suspension bars being too small, but most often to poor anchorage of these bars in the junction at the bottom part of the cylindrical silo wall.

We cannot overemphasize the importance of this anchorage. The rods must be generously bent back into the junction. In practice the action of a silo almost always departs more or less from the simplifying hypotheses assumed in the strength calculations, because this action varies greatly with the time. The only sure datum is the weight of the material to be stored, which remains constant for a given dead load after deductions have been made for the effect, if any, of desiccation, chemical transformation, etc. Actually, this weight is balanced by the following factors: (1) the friction against the vertical walls and (2) the strength of the "discharge opening".

If one of these factors decreases, the other increases, and vice versa, since their sum is constant.

Now, at the beginning the stored material is still not compact and its angle of internal friction is relatively small. Its thrust against a vertical wall is then at a maximum, as is the part of its weight supported by the friction against this wall.

The pressure against the unloader opening is then at a minimum. In the course of time the material settles down, its angle of internal friction increases, the lateral pressure decreases and the load on the discharge opening increases. This phenomenon is sometimes intensified because of the polishing of the vertical wall by the movement of the grains, some of which, moreover, have a lubricating effect. This is why it is frequently found that the cracks in the vertical walls appear shortly after the silo is put into service, while dislocations of discharge openings take place somewhat later.

Cracks occur sometimes also as a result of the opposition to the free play of linear variations due to hardening shrinkage and thermal variations. This case is rather frequent in cylindrical cells which are touching each other and which for this reason are subject to rather intense secondary bending stresses.

Finally, at the time of loading or unloading, eccentric dynamic forces are produced especially at the root or in the base of the discharge openings which should be taken into account by suitably increasing the theoretical forces.

Tests on models, while they give useful hints in principle in this connection, are generally much less accurate as far as their numerical results are concerned, especially when materials are involved whose properties are modified under heavy pressures, as is particularly the case for certain grains. Only tests on full-scale structures are then of unquestionable value.

To sum up, the best theories for the calculation of silos inevitably contain rather large gaps. Thus it is wise to heed the teachings of experience in order to rectify them in their results.

Where grains are to be siloed the seeds of which may be damaged by any roughness or excess of pressure the walls should be made as smooth as possible and all cross bracings or other internal parts, even with rounded edges, should be avoided.

In using prefabricated elements, which often join poorly with the parts cast on the site, care must be taken to fill all gaps or cracks in which weevils or other parasites might find shelter.

When the sidewalls of silos have horizontal corrugations the resistance to the descent of the siloed materials tends to the formation of "arches" and development of the dynamic effects resulting from this during emptying operations.

The corrugated elements may then be subjected to torsion stresses which it would be dangerous to underestimate. One may then, at some inconvenience depending on the case, prevent these phenomena by inspection or by means of devices installed permanently in the cells.

Reservoirs

Most reservoir leaks occur at the corners of rectangular tanks and at right angles to the junction of the floor with the walls. In the first case it is sufficient to ensure the joining of the walls by heavily reinforced chamfers (Fig. 23) fitted with braces "a" to balance the thrust in the direction of A.

In the second case the secondary stresses should be taken into account which are due to the obstacle presented by the stiffness of the floor to the free expansion of the vertical walls at their base, which tend to become deformed in the manner represented in Fig. 24.

The theoretical determination of these secondary stresses does not present any special difficulty.*

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* See: "Le calcul des reservoirs circulaires" by Henri Lossier, Génie Civil, July 30, 1910.

In the case of cylindrical reservoirs exposed directly to the sun, the tank, heated on one side, tends to assume an elliptical form, whereas the floor, being in contact with the ground, remains substantially circular. This increases the tendency towards separation of these two units, so that it is logical to reinforce their dowel rods more than the respective reinforcement rods of each, or to protect them by insulating devices.

Some builders reduce or eliminate the positive contact between the floor and the walls, separating them by an impermeable, elastic joint. The walls can then expand freely at the base, either finding support on struts or rollers or resting on a layer of plastic material. In the latter case the following two precautions are absolutely indispensable: First one must choose a product which will retain a constant plasticity; second the plastic layer must be protected against penetration by concrete parts of the wall in the course of execution, since this would produce welded points which might interfere with the normal working of the device adopted.

Horizontal cracks are often observed in reservoir tanks which are difficult to explain theoretically. They are generally due in part at least to differences in shrinkage between wetted parts and parts which are not in contact with the water.

In order to prevent these cracks it is always necessary to provide vertical bars in a proportion of approximately 0.5 to 1%.

<u>Supporting beams of overhead travelling cranes</u>

These beams frequently suffer damage, the principal causes of which are generally as follows:

1. Faulty estimation of the stress increases due to shocks and vibrations.

2. Inadequate resistance to lateral stresses.

These stresses are often greatly aggravated either by the malfunctioning of the equipment and their swaying back and forth or by rough handling on the part of the operators. One frequently sees loads picked up which are not placed directly below the normal position of the hook. This results in horizontal stresses which the crane transmits to its track beams.

In designing track beams it is always wise to foresee the worst as far as the functioning of the machinery and the handling by the personnel is concerned.

In many cases the welding of some at least of the rail joints helps to suppress the shocks due to rollers passing over these joints.

We should point out that posts supporting roofs over travelling cranes and the roofs themselves sometimes give way from the dynamic effects after a certain number of years of service.

Machine pedestals

The foundation pedestals of machines may be subject to intense dynamic stresses which must be taken into account.

The typical case is that of a turbogenerator for which the static loads are at present increased by 400 to 500% for this purpose.

In particular, one must be on guard against the so-called "resonance" phenomena which occur when the rpm of the machines coincide with either the vertical or horizontal natural frequencies of the pedestal.

Now, the calculation of these frequencies is often very problematical. On the one hand, pedestals of reinforced concrete have massive and complex forms which scarcely permit careful definition of the problem, whereas, on the other hand, the value of the coefficient of elasticity of the concrete is not a constant in time.

This is why, unless preliminary measurements have been made on models that are really comparable in all respects to the real structures, it is wise, even if the results of the frequency calculations are favourable, to assume the possibility of resonance occurring either on starting up or in the course of operation.

In order to make provision for this possibility without disturbing the machine it is possible to put bars in the pedestal with a view to adding on other elements (slabs, beams, etc.) which could be executed, if necessary, without interrupting the use of the machine.

It is quite possible, also, to provide beams or posts in the form of partially joined elements capable of being made completely independent or being connected up positively in the course of operations so as to vary their rigidity as required.

Generally, of course, a slight modification or addition to a pedestal is all that is required in order to get rid of certain resonance phenomena.

In other words, in the absence of precedents or tests on really comparable models the results of theoretical frequency calculations of reinforced concrete pedestals must be accepted with a great deal of reserve.

It may be mentioned that Electricité de France at our suggestion is examining its turbogenerator pedestals in order to determine what variations the properties of the concrete may undergo in the course of service.

The method of auscultation chosen is based on the velocity of propagation of sound in the materials.

The question of resonance also arises in many cases of industrial equipment where vibrating machines or looms are employed. Wherever the direct mounting of equipment on reinforced concrete may run the risk of producing such phenomena it is wise to consider the use of shock-absorbing or insulating devices in the form of cork, rubber, springs, etc., in cooperation with qualified specialists.

## Rectangular slabs

Rectangular slabs of very large dimensions frequently develop extrados cracks more or less at right angles to the bisectors of the corners, as shown in Fig. 26. These cracks are due to negative bending moments which alone can interfere with the upper diagonal reinforcements, as Mesnager has shown in his excellent study of thin rectangular plates.

## Tensile or outward thrust stresses

Tensile or outward thrust stresses are one of the causes of very serious accidents.

When a rod under tension is bent or curved (Fig. 27) a tensile force is produced in the direction V and tends to cause the concrete to burst towards the outside while at the same time detaching the rods and hence breaking the structure in the zone involved as a result of bending.

Thus special yokes capable of balancing the total stress passing outside should be provided and should be placed sufficiently close together in order to restrict the bending of the bars between the points of support which they provide for them. The use of crossed bars does not always avoid the danger sufficiently (Fig. 28) or eliminate the need for special yokes.

A curvilinear ribbed casting under a compressive stress is similarly subject to thrust forces passing to the outside which must be balanced by means of yokes and by increasing the number of reinforcement rods between the ribs (Fig. 29). This is a precaution that is often neglected in the case of ribbed arches where some of the ribs are on the extrados and some on the intrados side.

Retaining walls

Retaining walls are subject chiefly to overturning and sliding, and both these phenomena may occur simultaneously. A frequent cause of such phenomena is the inadequacy of the drainage devices, tending to increase often very considerably the thrust that the walls must resist.

I shall leave aside the question of incorrect estimation of the earth pressure due to an improper application of classical theories, as this would go beyond the scope of the present paper. We may simply note that when a soil is not homogeneous and especially where there are layers of clay, the results of the theoretical calculations must be interpreted very conservatively, referring wherever possible to comparable cases.

At all events, very special care must be taken in studying the slope of the foundation footings from the point of view of the possibility of sliding.

When a retaining wall stands on really stable ground the resultant of the loads and thrust may safely produce pressures at the base which intersect towards the outside edge. However, if a foundation soil is compressible the wall in time will "nose over", which might cause a great deal of trouble. In that case it is advisable to give the foundation footing a shape such that the resultant R of loads and thrusts will pass either through its centre of gravity G or better still, slightly behind the centre of gravity (Fig. 30). Since the true value of the thrusts is almost always more or less conjectural, the stability of a wall may be verified by combining the most unfavourable extreme hypotheses.

Mistakes are most frequent in retaining walls built on clayey ground and for walls of a great relative height. They are often due to the adoption in the a priori calculations of too high an angle of internal friction or too high a cohesion value.

## Bond

Bond is a phenomenon that is underestimated and misunderstood by too many builders. It depends on several factors, in particular the following:

1. A generally weak "gluing" effect;

2. Friction due to the pressure of the concrete against the steel as a consequence of the hardening shrinkage. This pressure is a function, in particular, of the thicknesses of coating, the neighbouring reinforcement rods, connecting rods, etc.;

3. A moulding effect produced by the roughness and surface irregularity of the rods.

Apparently this latter effect is often the preponderant one, because rods which are strictly cylindrical with polished surfaces will slide in the concrete under comparatively weak forces.

This is what justifies the adoption of "deformed" or twisted rods or the like when it is a question of high strength steels of which the bond values are particularly high.

As early as 1905 the author proposed the use of devices of this kind together with the welding of oblique yokes onto the main rods. However the regulations at that time prevented us from realizing the advantages effected by the corresponding savings in steel.

On several occasions failures have occurred with the current procedures as the result of inadequate bond.

The most frequent causes of such failures have been as follows:

(a) Reinforcing rods of large diameter placed too close together and inadequately covered.

(b) Rods grouped in bundles, which builders had considered in their calculations as independent reinforcement from the point of view of bond, whereas only the outside circumference of the bundle could actually act as such (Fig. 31).

As a result of this the ends of tie-beams of arched roofs have come loose, resulting in partial or total collapses of the structures.

With deformed, twisted, or similar rods the covering thicknesses should be increased as well as the length of splices, because of the thrust to the outside exerted on the concrete by reason of the sliding stress of these rods.

Finally, the ratio of the bond value to the concrete compression value tends to decrease with certain high strength cements, which, moreover, have other disadvantages with respect to shrinkage and susceptibility to cracking.

Joints of rods

When the rods from the mill cannot be delivered in a single length, joints have to be provided for.

Certain foreign regulations require these joints to be made either by welding, or with the aid of threaded sleeves in order to ensure continuity of the strength without allowing for the effect of the concrete cover.

In most cases this restriction seems excessive in practice, since it is possible to obtain joints which behave quite normally simply by crossing or overlapping the rods for a sufficient length.

Three devices are currently being used, the joints being, of course, always alternated (Fig. 32). These three devices are as follows:

(a) The ends of the rods are made to overlap for a distance equal to about sixty diameters.

(b) The ends of the rods, furnished with hooks, are overlapped for a distance of about thirty diameters.

(c) The extremities of the rods, without hooks, are simply placed end to end, the total number of rods being increased so as to produce overlapping joints.

This last method, which is used at the present time in France for the tie-beams of large vaults and bow-strings, has the advantage of furnishing a uniform profile for concreting without the local crowding of metal which occurs with the first two methods.

Accidents due to rupture of joints generally occur as the result of one of the following two causes:

(a) The rods do not overlap for a sufficient length or are accumulated in a given zone where the excess of metal does not allow the concrete to ensure the required bond.

(b) The spacing at joints made by placing the reinforcement rods end to end is insufficient to ensure continuity of strength from bonding of the metal to the concrete.

A typical example of case (a) is that of an arch of 45 m span subjected to vibrations which suddenly collapsed after several years of service owing to the rupture of a tie-beam without giving any visible sign beforehand of the imminent danger, such as the appearance of cracks.

Ruptures of the joints are especially to be feared because they almost always occur suddenly and without warning.

Anchoring the ends of vault ties or trusses

Several serious accidents have happened owing to the imperfect anchoring of ties at their extremities. Fig. 33 and 34 represent several defective arrangements which have resulted in failures of this type.

Fig. 35, on the other hand, shows the abutments of the Lucien-Saint bridge in Tunisia, a bow-string of 92 m theoretical span, the end anchorages of which were the subject of a particularly careful study.

Buckling of plates between the ribs of arches

In many arched roofs the plate is deformed by buckling between the ribs in the direction of the directrices thus abdicating at least partially the role which it should assume in the strength of the unit (Fig. 36).

There are two possible causes, which may also combine to produce this result. The plates may have been too thin to begin with, and secondly the shape may be inappropriate or may have been poorly realized in execution.

If it can be said that some plates whose theoretical buckling strength appeared doubtful have nevertheless behaved well it is almost always because the form work had been executed with particularly great care. On the other hand, as many tests carried out on thin arches have shown, a slight initial deformation is generally sufficient to start the buckling. This phenomenon is not always immediately perceptible at the time of stripping, but shows up frequently a few months later as soon as the flow of the concrete becomes accentuated. It is wise, therefore, to keep within safe limits in reducing the thickness of plates, especially as the saving in quantity of material achieved may be more or less wiped out by the additional labour required and by certain dangers. One should also impress on the foreman the necessity of extreme care in the execution of the form work.

Construction joints in concrete

Poor construction joints may produce cracks in parts that have standard reinforcements and may result in failure in zones where the strength of the concrete in tension is discounted. The greater the difference in age between the old and the new concrete, the higher the losses of strength.

Even in joints inclined 45° relative to the direction of the tensile stresses and where water has been applied liberally before repairing, these losses are generally of the order of 30 to 40% after a week and 80% after a month.

Reinforcements to resist shearing stress

While ruptures due to bending moments generally give warning long in advance by very obvious deformations, those resulting from shearing stresses are almost always sudden and unexpected, like those of posts that are not hooped, and this makes them all the more dangerous.

It is always unwise, except generally for slabs or other massive elements executed without concrete relief joints, to leave out any secondary reinforcements on the grounds that the theoretical stress of the concrete will not reach the so-called safety limit.

In particular, there may be unintentional, or poorly placed relief joints, or secondary stresses due to the shrinkage, the free play of which is hindered either by reinforcements or by other resisting elements and which the designer cannot always predict with certainty.

Generally, therefore, from the point of view of safety the careful study of the shearing strength is more important than that of any other strength value. One should proceed on the basis of the most unfavourable hypothetical stresses which can possibly occur.

Besides problems of strength as such, problems arise with respect to cracking due to tension in the concrete. This cracking, which is but little alleviated by vertical stirrups or stirrups placed perpendicular to the axis of the members, is influenced much more effectively by slanted stirrups or hooked bars, provided, however, that the spacing of these bars is limited not only in relation to the height of the beams, but also to their diameter and even in certain cases, in absolute value. Grids with welded or solid intersections (expanded metal, etc.) are generally very effective from this point of view.

Certain devices comprising only hooked bars without stirrups must be summarily rejected (Fig. 37 and 38).

So-called "floating" rods (Fig. 39), which can have a favourable effect from the standpoint of cracking, must not be discounted in the strength by reason of their possible failure in the vicinity of breaking loads.

## Compressed frames not wind-braced and girders of polygonal plan

The lateral buckling strength of frames which are not mutually wind-braced (Fig. 40) may be estimated by various theoretical formulae in current usage. With reinforced concrete, however, it is always wise to verify or supplement the results with a consideration of the casting deformations at a value considerably above the standard allowances and then to determine their incidence on the uprights and bridge pieces which resist the buckling by their transverse rigidity.

This is a simplified calculation by which one may rapidly ensure that no serious error has occurred in the application of formulae that are sometimes quite complex.

The thrusts to the outside at changes of direction of frames with polygonal plan resulted in a particularly serious accident in the early days of reinforced concrete and must always be very generously resisted (Fig. 41).

## Antifriction plates

In some edifices in order to avoid the use of struts or rollers antifriction plates are provided in order to take up the linear variations due to shrinkage and thermal effects.

In small-span structural steel edifices the reactions of supports are small between the passage of vehicles and the friction resistance is easily overcome. In reinforced concrete structures with their higher dead loads the same thing does not always apply and abutment separations are sometimes observed (Fig. 42), especially if the antifriction plates through lack of maintenance have been welded by rust. On light piers, indeed, these plates only provide poor articulation.

Defective struts

Certain builders in order to gain height use struts, the faces of which do not have the same centre of curvature (Fig. 43) and whose working produces variations of level on the one hand and horizontal stresses on the other. These devices should be rejected in most cases.

Effect of shrinkage and temperature variations

The shrinkage of the concrete during hardening is responsible for many minor mistakes, but rarely results in a serious accident.

It is evident a priori that if relatively old elements are combined with a new concrete the latter, during shrinkage, will subject these elements to compressive stresses and will produce shearing stresses between the two materials. Thus, surface coatings or pavings without joints or with insufficiently plastic or elastic joints, when placed on reinforced concrete floors have sometimes lifted, glazed elements have burst and the uprights and cross pieces of façades have cracked because the brick fillings did not have sufficient play, etc.

Since dry air shrinkage can be reduced but not eliminated it should always be taken into account. This can be done in various ways, notably the following:

1. By providing shrinkage joints which separate a structure into several independent sections from this point of view. The distance between these joints is not a constant; it depends especially on the characteristics of the structure. For contemporary shells it is frequently given as 20 or 30 m, but this distance may possibly be much greater if flexible or articulated uprights have been provided, as has been done in a number of modern hangars.

The Biscarrosse hangar, for example, which we designed in 1939 and which was 150 m long by 60 m wide, covering an area of almost a hectare, had a roof which was freely expansible in all directions and contained no joints.

Such arrangements have been realized for covering reservoirs of large area, the fixed point upon which all the expansion arrangements are based being a tower or a self-supporting pylon placed in the centre.

2. In some cases the initial action of shrinkage is eliminated by furnishing provisional joints or "field joints", i.e. by separating the concreting of certain elements by as long a time as possible.

In monolithic structures of large relative height, e.g. bridges of double "T" section beams or tubular section beams, the appearance of cracks is sometimes noted which completely traverse the vertical web and stop in the vicinity of the flanges. These cracks are more or less vertical towards the centre of the span and are slanted in the vicinity of the supports where the maximum shearing stress occurs. If the tension flanges are themselves joined by a horizontal, under-reinforced slab, the latter may also show transverse cracks.

Finally, more or less inclined shearing cracks may appear near the transition between flanges and slabs.

These are all shrinkage phenomena which are particularly frequent when fast-setting cements are employed.

In fact, over-reinforced flanges restrain the free shrinkage of pours in the longitudinal direction, and the latter, under-reinforced, ordinarily crack in the manner indicated schematically in Fig. 44.

In cases of this type it is wise to observe the following rules as far as possible:

1. Avoid using fast-setting cements ("high-early") as these are generally more friable and subject to greater shrinkage than ordinary cements;

2. Avoid abrupt changes in the percentage of reinforcement and the use of rods of large diameter clad in concrete of relatively small thickness;

3. Never go below 0.5%, and if possible never below 1% in reinforcing elements which may be subject to tensile stress, even when reinforcements placed elsewhere may theoretically, by themselves, compensate the total bending component.

4. Wet the concrete during the first days of hardening. The reinforcements do not always eliminate the danger of cracking; however, they restrict the opening of the cracks and hence also the danger of oxidation of the metal although these possibilities are sometimes increased in number.

To sum up, failure to take adequate precautions against the effect of shrinkage results in cracking, scaling and leaks in reservoirs and canals. These are no doubt regrettable, but rarely threaten the actual stability of the structure.

However, just as a flush on a person's skin may, as far as a layman can tell, indicate either a serious illness or a mild discomfort, so the appearance of a crack in concrete is almost always a source of anxiety to the user of a structure, because it may indicate a lack of strength.

Shrinkage is therefore the cause of many controversies which are often unjustified and to which certain insufficiently specialized experts have sometimes attributed a disproportionate importance.

In certain regions differences of thermal expansion may produce cracks in façades not exposed to the sun, especially if the heated side is slightly longer than the opposite side. This applies even to buildings which do not exceed 25 m in length.

Generally speaking, cracks tend to appear at the re-entrant corners of openings and façades.

When terraces or roofs include beams of large relative span differences of temperature produce temporary positive or negative curvatures depending on the sense of these differences. This may have serious effects on the permeability, and in certain cases heat insulation devices should be provided.

In caisson sections it is wise to provide holes in order to equalize temperature and humidity differences between the interior and the outside air.

4. Precautions to be taken against certain external factors

Reinforced concrete is sufficiently important in the domain of construction to be content with its own true qualities, without claiming advantages which it does not in fact possess.

While it resists fire better than most other materials, nevertheless experience has shown that it has practical limitations of considerable magnitude in this respect.

Similarly, its freedom from maintenance costs, which is again one of its highly valued qualities, is only absolute in certain very specific cases.

Let us consider, in order, the following external agents:

(a) Corrosive water; rain water.

(b) Fire.

(c) Earth tremors.

(d) Electrolytic effects.

(a) Corrosive waters and other corrosive substances. Generally speaking all sulphates are injurious to concretes, especially the following:

sulphate of lime;
sulphate of magnesium;
sulphate of soda;
sulphate of potassium;
sulphate of ammonium, etc.

The most dangerous of these, generally speaking, is sulphate of lime carried in ground water and which may originate from plaster, smoke, etc.

Waters containing as little as 0.2 g sulphate per litre are already harmful. Above 0.5 g per litre their destructive effect is generally a certainty.

Apart from drainage and the removal of streams or sheets of sulphated water in order to prevent their contact with the concrete if possible, their effect can also be minimized by using the following cements, in order of decreasing efficiency:

metallurgical, supersulphated cements;
some aluminum cements;
cements with high slag content;
pozzuolana cements;
ferrous cements.

The effect of sea water, which is at once mechanical (wave action), physical and chemical, is particularly important in so-called tidal zones, i.e. between high and low tides. Under these conditions cements having a low free-lime content (metallurgical-supersulphated-pozzuolana cements, high slag metallurgical cements, etc.) are then to be recommended.

Moreover, the corrosive action of very pure water on mortars of manufactured cement has long been known. The rupture of a reservoir in the Ardennes some years ago is a typical example of this effect. Chemically pure water had produced holes of very small diameter in the floor and the water seeping through under pressure undermined the soil, causing the rupture of the floors in two compartments of the structure.

However, builders do not appear always to have paid sufficient attention to the effect of rain water on ordinary constructions.

During the last few years I have been asked to examine a number of reinforced concrete roofs of various ages between 18 and 30 years which appeared to be in a decidedly alarming state, several of them having already undergone partial collapse.

After eliminating the cases of mere cracking due to the absence of shrinkage joints or the use of initially too porous concretes, poorly clad reinforcements, exposure to acid smokes or vapours, etc., the phenomena found were generally as follows:

Arched roofs. In certain parts the concrete has been hollowed out, a considerable amount of the mortar having vanished. The reinforecement is attacked, sometimes to the point of being eaten right through by rust. The anchorages of tie-beams at right angles to gutters, have in several cases given way for this reason.

The parts not exposed to the action of the rain, such as the abutments which transmit the thrust of the arches at right angles to the ridge turrets, are in good condition, as are all the interior parts of the buildings.

However, the uprights and cross members of façades exposed to weathering show hollowed sections and bursts in the concrete due to expansion of the oxidized reinforcements. One finds the familiar whitish lines in the vicinity of the cracks.

Stepped roofs. Phenomena similar to the above, often aggravated by the flatness of the slope, permitting less than normal drainage of water even in the vicinity of the gutters.

Reticulated structures. Thin squared members, struck on several faces by the rain, as well as the gutters are most dangerously attacked.

All the cases considered have to do with roofs on which there has been no maintenance of the initial waterproof coating.

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Actually, the action of rain water is merely a special case of the effect of pure water on cement, which was pointed out as early as 1896 by Stutzer and which, by 1917, had attracted the attention of a few engineers. Consider, for example, the Portland cement pipe laid in 1913 at Vannes which was rapidly disintegrated by the granitic water of this region.

Such events are all the more surprising when it is remembered that the same mortars behave quite normally in the presence of slightly calcareous water.

To explain this difference it is assumed that the calcareous constituents of the cement which are dissolved or hydrolysed by the pure water give, with the bicarbonate of lime found in ordinary natural water a carbonative lime which forms in the pores of the mortar, sealing them and stopping the attack.

In the case of rain water, the possibility of carbonation is linked to the action of carbon dioxide, which all water contains in quantities that are too small to convert the carbonate into the soluble bicarbonate state.

Thus, the possibility of a concrete or mortar being affected by rain water depends on two main factors:

1. Its density, since it is clear that a compact mortar washed by the rain will always be less susceptible to change than a porous one that can be progressively "leached";

2. The conditions of surface carbonation. This becomes the more effective, from the point of view of mortar protection, according as it applies to less soluble calcareous compounds, as Le Chatelier has pointed out.

This carbonation, a function of rain water which always contains a small quantity of free carbon dioxide, possibly borrowed from the atmosphere, varies in its processes and circumstances with the climatic conditions, especially with the frequency of the rains to which the concrete is subjected during its lifetime.

The poorer the cements are in lime, the more effective will be the carbonation. From this point of view, all other things being equal, the artificial cements, which have the highest mechanical strength values, are also the most susceptible to change.

Pozzuolana, metallurgical and certain aluminum cements are among those least susceptible to change.

It is also true that the longer the concrete is exposed to the air before it experiences rain, the better will be its behaviour.

With regard to reinforced concrete, it should be pointed out that the carbonated layer loses its alkalinity and no longer ensures the protection of the reinforcements against oxidation. If the latter are in a carbonated zone they then tend to become oxidized and the rusting produces the familiar swelling which may dislodge the concrete.

To sum up, as the studies of Anstett, Lafuma, Cocagne, etc. have established, rain water because of its very purity may be considered as an agent that is harmful to reinforced concrete structures, especially roofs and the façades of buildings exposed to the rain.

In most cases, therefore, in addition to the ordinary precautions (shrinkage joints, compact concrete, reinforcements given a sufficiently thick coating of concrete, etc.) a durable waterproof coating should be applied and should be regularly inspected and repaired when necessary.

Although the slowness with which these phenomena manifest themselves generally releases the builders of their ten-year responsibility, the durability and safety of the structures may be nonetheless seriously menaced in the course of time.

Certain organic substances, for example sewer waters, milk, whey, mazut, etc., can attack concretes. Mineral and vegetable oils, except for mazut, cider, wine and formol are relatively harmless.

In each separate case, the choice of cement and the possible means of protection must be studied in detail.

On an impermeable surface free of cracks the effect of plants generally need not be feared.

Let us repeat once more: no matter what the substance one wishes to protect, maximum density of the concretes must be aimed at by means of proper grain-size selection and high compaction.

(b) Fire. While undoubtedly reinforced concrete behaves better than most other materials in the presence of fire, nevertheless its resistance has limits which, although wide, are still imperative.

Under increasing temperature concretes do in fact progressively lose their interstitial water.

Above approximately 100°C, cements also begin to lose their water of crystallization.

Moreover, the nature of the aggregates plays a dominant role. Aggregates made from igneous rock are generally preferable from this standpoint to calcareous aggregates.

Reinforcements generally lose all useful strength in the vicinity of 500 to 600°C. This temperature range corresponds to the initial failures of unclad metal structures.

In fact, the mechanical strength values of concrete quite quickly become modified by any lasting elevation of temperature. The tensile strength is more severely affected than the compressive strength.

Certain tests showed the following decreases of compressive strength after an exposure of four to five hours of a concrete consisting of calcareous aggregates:

At 400°C: 35%;

At 900°C: 65%.

With igneous rock aggregates these figures were reduced to 10% and 40%, respectively.

It should be noted that laboratory tests carried out on small-scale models generally give less favourable results than are encountered in actual fires lasting four to seven hours. The transmission of the heat to the interior of a structure is of course retarded by its more massive character compared with the laboratory models.

In relation to the cladding of reinforcements, a thickness of 20 mm resisted a temperature of the order of 1000°C for two hours, while a thickness of 50 mm resisted for about twice this time.

In the course of my examinations I have found, for reinforced concrete structures of standard design, that if the temperature has not exceeded 400°C and the fire has not lasted longer than six to seven hours, the structure can be saved with the aid of some local repairs or restorations. If the temperature has risen to about 600°C the structure, although seriously compromised, will not collapse as would a metal structure exposed to the same conditions. In the vicinity of 800°C and up, rupture may or may not occur, but in any case the structure is almost always ruined.

Among the fires which have resulted in the complete collapse of a structure, we may cite the burning of a liquid fuel depot. The uprights, which were reinforced with poorly clad rods of large diameter, yielded suddenly under the force of the hoses, causing the death of several firemen who had placed too much confidence in the reputation of reinforced concrete for resistance to fire.

To sum up, structures which may be subjected to extremely high temperatures in the case of fire should, as far as possible (in addition to taking other known precautions) be built with aggregates of igneous rock and the reinforcements should be of small diameter, well embedded in the concrete mass.

(c) Effect of earth tremors. Experience shows that monolithic reinforced concrete constructions have definitely higher strength in this regard than those in adequately reinforced masonry.

Generally speaking, tremors with accelerations below 25 mm/sec$^2$ and a propagation rate of 2 mm/sec are scarcely more dangerous than wind stress.

Above these figures special measures must be considered.

Unless curves derived from direct local observations are available it may be assumed that apart from catastrophic events an acceleration figure of the order of 3000 mm/sec$^2$ in the horizontal sense with a maximum propagation rate of 250 mm/sec will serve as a basis.

Several methods of calculation have been proposed to determine the stresses on building frames subjected to these tremors. These methods can be found in our principal publications.

In some countries, notably in Italy, standards have even been published.

In this field it is obvious a priori that the most exhaustive theoretical calculations, like the empirical rules, can only give us the orders of magnitude which should lead for the sake of caution to the adoption of reduced working stresses.

From the structural point of view it is wise to avoid structures of large relative height in relation to the base, as well as slender units with poor resistance to torsion, large cantilevers, heterogeneous foundations on loose ground, etc.

The structures must be monolithic, stiffened and wind-braced in all directions.

(d) Electrolytic effects. In certain parts of a structure in North Africa close to the sea a considerable decrease of strength in the concrete was noted with the presence of ferric salts resulting from the electrolytic decomposition of the reinforcements.

At right angles to various nodal points in the grids of rods the reinforcements had actually disappeared completely and the two stumps forming part of the same rod were broken off (Fig. 45).

The structure was exposed to the action of sea water at high tide and the presence of electrolytes in the concrete had apparently promoted its decomposition. The origin of this effect appears to reside in the attack on the reinforcements.

Probably the lighting system had not been inspected for many years and leaks of current in the presence of moisture were set up which promoted this electrolysis.

## 5. Miscellaneous

### Buildings with inclined uprights (Fig. 46)

For the sake of certain technical requirements some builders have on occasion employed posts which were more or less inclined from the vertical.

The inclination of these uprights produces horizontal forces on the ties so that special precautions have to be taken in order to balance the resulting secondary stresses and especially the twisting moments. For this purpose one may employ strong interior partitions, yokes, rigid patterns, etc.

A small building in which these precautions were insufficiently observed collapsed sideways.

### Miscalculations due to flow

Flow, or the slow deformation of concrete under stress in the course of time, the principle of which has only been known for a comparatively short time and the mechanism of which still requires clarification, several years ago resulted in numerous difficulties in the use of a number of hangars of large span fitted with sliding doors under cantilever roofs.

The doors had been given insufficient clearance in order to take into account the considerable increase in the deflection of the girders in the course of time. They therefore jammed, making the sheds temporarily unusable from time to time.

Builders, duly forewarned of this phenomenon, now allow a margin based on experience and these miscalculations now occur less and less often.

The same applies to the determination of the initial camber to be given to bridges, especially arches and bow-strings.

It is only a few flattened arches with keyed joints which now show an angular deformation of early origin, giving them a simply disgraceful appearance from failure to observe this precaution (Fig. 47).

In certain structures which depend strictly on geometrical shape for their stability, flow has resulted in secondary stresses of a more or less serious character, and in at least one case has led to the collapse of an unfluted cupola.

Reinforced concrete boat hulls

Among the many river and sea barges built towards the end of the First World War there have been relatively few failures except for collisions and shipwrecks.

We may, however, recall the following few examples: (1) the breakup during operation of a 45 m Seine barge of reinforced slag concrete, probably due to a local quality failure of the concrete; (2) the breaking in two in the centre of a 70 m barge of ordinary reinforced concrete while it was being loaded. This was due to a flagrant inadequacy of longitudinal reinforcements. During the inquiry a barge of the same series which had been selected for testing broke open in the same way and under approximately the same load; (3) cracking due either to an insufficient number of reinforcements to resist the shearing stress, or to secondary stresses resulting from the differences of linear variation of the submerged and above-water parts.

The latter danger always requires extra reinforcements, particularly so-called secondary or linking rods. Trellices or stirrups at 45° in both directions have given decidedly better results in this respect than vertical and horizontal rods.

Actually, reinforced concrete boat hulls for which the designers had taken the trouble to give standard forms have generally behaved well from the point of view of their resistance to shocks and ease of repair; however their high dead weight considerably reduces their popularity in normal times.

There are still those, however, who advocate them for harbour service vessels which have to be ballasted, for example the pontoons of dredges.

Dangers of certain combinations

It would seem obvious that if two parts of a structure, each of which is stable in itself, are joined positively together then the whole structure should be equally stable. However this is not always the case.

Let us consider the example of a shed containing a self-supporting arch V and a rather high wall plate R designed to withstand only its own weight and that of the weather board (Fig. 49).

When the two parts V and R were joined together in the execution the arch, after removal of the forms, exerted a horizontal eccentric force against the beam which caused lateral overturning by torsion and then rupture.

This is the classic problem of associating functional parts with poorly harmonized deformations.

Concrete surfaces of roads and landing strips

Concrete surfaces of roads and landing strips may be, depending on the requirements, of non-reinforced concrete, reinforced concrete, prestressed concrete, poststressed or self-stressing concrete (expansive cements).

They are subject to a variety of stresses. Apart from the effect of the live loads which they must withstand while resting on a more or less elastic or plastic soil, they are also subject to shrinkage, variations of ambient temperature, variations in the soil moisture content and the atmospheric humidity, direct insolation, etc.

The chief known failures are as follows:

(1) breaking as a result of bending or shearing under the wheels of vehicles, trucks or aircraft;

(2) cracking due to shrinkage and thermal variations;

(3) local heaving during a severe increase of temperature.

In attempting to guard against the latter two effects at present gaps or joints are provided which divide the surfaces into panels of limited dimensions (5 to 15 m), as was done for railway tracks until just recently.

Now, whether it is a question of roads, landing strips or railway tracks, expansion joints always present a number of disadvantages, with respect both to the stability of the subsoil and to the wear and tear on vehicles and the comfort of passengers.

The desire to get rid of these joints or at least to reduce their numbers no doubt followed closely upon their introduction.

The welding of streetcar rails, and more recently those of railroads are a manifestation of this general tendency.

In France the S.N.C.F. is welding its rails in 800 m lengths and considers that quite apart from savings realized in road maintenance this welding represents an outlay that is absorbed within a few years.

In connection with concrete surfaces this question is only just starting to be considered.

Expansion joints from the point of view of load resistance

At right angles to a joint (Fig. 50) the local unit pressure on the foundation soil on passage of one axle can in certain cases attain a value of 4 times the pressure that it undergoes elsewhere. This may sometimes produce progressive dislocation of both the subsoil and the surface itself.

Various devices have been realized or contemplated with a view to reducing this effect.

Dowels (Fig. 51) or steel rods, at least one end of which slides freely in a pipe or insulating envelope in order to permit the longitudinal free motion of the joints, merely constitute, in the vertical direction, a semi-articulation of an often precarious functioning.

The pressure on the soil can often attain as much as twice that supported at mid panel (Fig. 50) and the rigidity of the rods sometimes produces secondary bending stresses that lead to cracking. Finally, if the dowels undergo appreciable angular deformations, the elastic limit of the metal having been exceeded may restrict the longitudinal linear variations. More or less semi-articulations can also be realized by other means, notably by mortising (Fig. 52).

Some of the devices have a tendency to produce partial continuity while allowing for the free play of the linear variations, thereby reducing the effect on the soil of the angular variations of the surface at right angles to the joint (Fig. 53).

Finally, we may mention longitudinal reinforced concrete beams underneath the joints (Fig. 54).

Any of these types of devices, which may vary in detail, can reduce the disadvantages of joints without, however, eliminating them altogether.

Some engineers, on the other hand, believe that while natural cracks in a cover impart a bad appearance by reason of their more or less irregular shape, in practice they are less troublesome than real joints, provided they remain "blind" or are filled by injection of a bituminous product in order to keep out the water.

Surfaces without joints*

Actually at the present time this involves only the reduction of the number of joints rather than their complete elimination, which is considered elsewhere.

Prestressing by means of wires or cables artificially tensioned

* See "Dangers of heaving in runways and on railways", as presented by Robert Levi and Perrin under the guidance of Henry Lossier. Report of 25 September, 1947 of the Institut Technique du Bâtiment et des Travaux Publics.

with jacks already appears to be giving interesting results. Full-scale laboratory tests based on the utilisation of the energy of expansive cements, which were started several years ago, are being methodically pursued.

A considerable reduction in the number of joints on load and runway surfaces even now appears to be a practical possibility. However, time alone will tell what the pathological results may be.

Calculation of the stresses resulting from the application of live loads, from trucks, tanks, or aircraft must take into account not only the stiffness of the surfacing material and the coefficient of elasticity of the subsoil, but also the dynamic stresses from rolling, taking off, landing, etc.

Various methods of calculation, sometimes very divergent, have been proposed.*

In my opinion only experimental tests can at present be considered really satisfactory from the point of view of safety.

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* See "Course in reinforced concrete", part 2, by Professor A. Paris.

## B. EXECUTION

The chief mistakes which can be attributed to the execution as such of a design are generally due to the following factors:

quality of the materials;
preparation;
centering or falsework;
form-stripping operations.

### 1. Quality of Materials

The quality of the reinforcement steel is very rarely involved in failures. Any deficiencies which originate in the factory may indeed reduce the safety margin, but are generally never sufficient to result in rupture unless other factors intervene simultaneously.

The quality of aggregates, apart from a lack of strength, can only result in serious failures if their chemical composition is incompatible with that of the cement, or with the subterranean waters, or the water of mixing.

Actually, some pebbles or gravels may even have a slightly lower strength than that required for the concrete without the latter becoming deficient, because the pressure of the mass around these aggregates increases their strength in the same way as any multi-axial stress acting on friable materials will generally do.

This has been discovered especially in Morocco, where the quality of aggregates frequently worries builders.

Certain porous aggregates are capable of absorbing as much as one third their volume in water. This should be taken into account when calculating the amount of water of mixing to be used. In all cases the aggregates should contain no gypsum, mica, coal, organic matter, etc. The content of clay and fine materials should be less than three percent by weight. The aggregates should be derived from inert rock which have no effect on cement and are not themselves affected by water, air or frost. Shales, feldspars and soft limestones must not be used.

In general, marine sands can be used if they are clean and contain no shells. These produce certain efflorescences, but have no disadvantage other than the poor appearance they impart.

The water of mixing may sometimes result in mistakes, but this problem is so well known among engineers that such occurrences can only be accidental. If such mistakes do occur they are due either to ignorance or negligence.

The water of mixing, of course, must not contain organic matter or materials in suspension amounting to more than 2 g per litre, impurities in solution to more than 15 g per litre, and must not be contaminated by industrial waste.

The most important factor is the quality of the cements.

In normal times the cement manufacturers enjoy the advantage over most other industrialists of having practically no foreign competition, largely because of the high cost of transporting this heavy material. Internal competition, moreover, is at present greatly restricted by the tendency of demand to exceed supply.

Under these conditions it is only natural for manufacturers, urged on by both public and private clients, to concentrate on increasing their output rather than improving the quality of their cements.

Another factor which has a definitely unfavourable effect on certain concrete construction work is the excessive importance attributed to binders of high initial strength.

There are of course many cases in which rapid hardening may be advantageous, by permitting an early removal of forms, but this advantage is sometimes more than offset by the troubles resulting from this property.

While most of these products are true rapid-hardening cements, occasionally an exception is encountered.

If speed of hardening is attained merely by finer grinding of materials carefully sorted and mixed, greater shrinkage may indeed occur, but will generally remain within very acceptable limits. However, the use of certain additives which promote initial hardening less expensively cannot always ensure the constancy of the results obtained over a period of time. It may also have an unfavourable effect on shrinkage.

Calcium chloride, in particular, strongly increases the compressive strength after 7 and 28 days, but scarcely alters the final strength values. An addition of 1.75% by weight of cement of this product may double the air shrinkage; an addition of 5% can increase it by as much as ten times.

Now, one of the greatest fears of builders who employ reinforced concrete is the fear of cracking. A crack in itself, however, has never killed anyone. It is only dangerous, in fact, in two main cases: (1) if it is wide enough to allow the oxidation of the reinforcement; (2) if it is not traversed ty rods capable of resisting by themselves all the tensile stresses.

In most current cases oxidation of the metal does not occur, except in the presence of aggressive gases or vapours, unless the width of the crack exceeds 0.5 mm at the surface. Below this figure cracks generally end before they reach the reinforcements.

However, cracking is never a favourable phenomenon and since, superficially at least, it is a cause of anxiety to the user of a structure, the builders are correct in attempting to avoid it. The cracking of the cement will be reduced: (1) according as its tensile strength is greater, or more accurately, as its ratio of tensile to compressive stress, an index of its friability, is higher; (2) the smaller its normal shrinkage; (3) the less intense its temperature rise, especially during the start of hydration. The latter applies particularly to large masses.

The ratio of tensile to compressive strength is generally smaller for high early cements than for the others, and their shrinkage is almost always greater. Hence the systemmatic search for cements of high initial strength, which may be justified in exceptional cases, is nevertheless frequently an error for most current reinforced concrete structures. It is better to be content with binders that merely assure the compressive strength taken into account in the calculations, while giving the best results as far as tensile strength is concerned with the minimum shrinkage and temperature rise.

A cement of high tensile strength will almost always show good compressive strength as well, but the reverse of this statement does not hold.

The crushing of clinkers at an excessive temperature, and too short a storage time in certain plants in which storage capacity is scarcely adequate for the normal daily production, may be the underlying cause of certain mistakes. It is being investigated at the present time whether, side by side no doubt with other factors, this inadequacy is not instrumental in producing intensive shrinkage and the phenomena of "false setting" which are sometimes encountered in some contemporary cements. However that may be, the quality of certain binders appears to be partly responsible, at least, for the excessive shrinkage and cracking which sometimes results either in the temporary interruption of the use of certain structures showing anomalies from this point of view until a complete examination can be carried out, or the breakage of glass elements embedded in reinforced concrete arches or other structural parts, etc.

Among the unexpected phenomena encountered during the past few years in the course of structural tests we may mention here that of residual deflections of a decidedly abnormal nature after removal of the load for the first time. In several cases all that has been needed has been the successive application and removal of the load several times in order to make the structure function normally again. With respect to the incidents produced by certain aluminous cements, which have been dealt with by various authors elsewhere, I shall confine myself to pointing out that the structures executed according to my plans with a water-jacket manufactured alumina cement have never resulted in any miscalculation. Such structures include many piles driven into sulphated terrains or into the sea, the Lucien-Saint bridge in Tunisia, a bow-string of 92 m span with reticulated arch, which for more than 20 years has held the world span record for structures of this category.

The mistakes which I have had to deal with in my capacity as an expert have always involved aluminous cements produced by other methods. This is a simple fact from which I shall take care to draw no conclusions which would not be within my competence.

To sum up, I cannot emphasize too strongly the necessity for builders in reinforced concrete to verify the quality of the cements delivered to them as far as possible before using them, taking special care to ensure that (1) the tensile strength is up to standard; (2) the shrinkage is normal; (3) there is no danger of "false setting"; (4) there is no excessive heating. There are rapid and simple methods for making this verification which can be carried out on any building site.

It should also be borne in mind that when a failure is due to a cement it is often almost impossible for the builder to furnish an absolute proof of this a posteriori. Too many factors of production, conservation and preparation can be invoked by the supplier in order to cast doubt on the contention.

In all circumstances the quality of a cement must always be verified before it is used.

Some additives, such as plasticizers, air entrainers, etc., do not always react in exactly the same way in the presence of cements of different quality. Thus in case of doubt it is wise to carry out preliminary tests of sufficiently long duration under conditions simulating their practical utilization.

## 2. Preparation

Even with excellent materials a contractor can still make poor concrete.

### Gradation of the aggregates

The gradation of aggregates has long been a subject of numerous controversies, and still remains so.

One of the most disputed problems, of course, is that of a choice between a continuous and a discontinuous gradation. In fact, too many authors have been armchair gradationists. The assumption made by some at the outset that the aggregates are spheres or cubes of given dimensions, whence the proportions to be employed in order to realize certain conditions are deduced, is incorrect for the very simple reason that in reality these aggregates have random shapes.

There are limits to the reduction of the proportion of voids, for with a proportion equal to zero the absence of cement would result in the disappearance of all strength. In fact, in every individual case, except where rocks are being crushed, one is confronted with aggregates determined by the local resources which must then be used under the best possible conditions, taking into account the cross-section, the dimensions and arrangements of the reinforcements in the concrete structural parts to be executed.

Some gradations which give excellent compressive strength in the laboratory, and which are excellently suited to unreinforced concrete structures such as dams and road surfaces, are of little use for reinforced parts because they give concretes that are difficult to work and which do not envelop the reinforcements effectively.

The value of the reference curves recommended by certain experimenters is in general limited to cases similar to those contemplated. In other cases they can only be considered as starting points from which additional control experiments must be undertaken in one direction or another.

To sum up, the study of gradation is above all an art of adaptation on the spot, and each case must be considered separately, approaching as far as possible the actual conditions of the structure, expecially as far as the workability of the concrete is concerned.

The gradation must result in simple rules that can be easily understood and applied by a good building foreman, since it frequently happens that supply difficulties necessitate a change of aggregates and hence of mix proportions in the course of operations.

The mixing water

The adverse effects of an excess of mixing water on the shrinkage and the initial strength values are too well known to require comment. The effect of the chemical composition of the water is equally well known.

Compaction

Compaction produced by vibration or internal vibration can give excellent results, but only in the hands of highly skilled operators. When badly applied these procedures often lead to very dangerous segregation phenomena. In several cases we have had to destroy completely structural parts that had been vibrated or internally vibrated indiscriminately. With contractors who had not had experience in the use of these methods it is often preferable to employ ordinary compaction without the aid of machinery.

## 3. Centering or Falsework

These units must be both strong and rigid, as indicated above. In rivers or streams where the floods are intense, special care has to be taken to protect the piers against undermining and the action of floating materials.

In some regions the flood waters carry branches, vines and grasses which can transform a falsework, if its submerged parts are too close together, into a veritable dam which then prevents any free flow of the water.

It is wise, therefore, to reduce the number of piers below the high-water level to a minimum and to shield them with a facing of smooth panels to which the floating materials cannot cling.

In some regions, especially in North America, several centerings have been carried off by floods for the reasons described above.

There are a number of devices for getting rid of all temporary supports in midstream. The most important ones are as follows:

By casting, i.e. by constructing the entire structure at once or in units as an extension of its final positioning then pushing or pulling it across the required spans using, if necessary, removable downstream cutwaters and ballasting devices. In certain cases a floating equipment can be used either for placing the prefabricated units or for setting up a falsework supported only on end piers and abutments.

By using a movable falsework assembled on one bank and then put in place by casting above the first span, using the abutment of the neighbouring pile for support. The formwork is then suspended from the falsework. After hardening and stripping the first span of the deck the latter is used as a casting bed, the movable falsework is thrown over the second span, and so on (Fig. 56).

By cantilevering, a method which consists in using the parts already built to support a movable apparatus having a cantilever which can be used as a platform for the execution of subsequent stages (Fig. 57).

## 4. Form-Stripping Operations

This operation underlies the great majority of known reinforced concrete accidents, accounting for from 60 to 90% of them, depending on the country.

Several main errors should be borne in mind. First of all the form may not have sufficient rigidity and the structure will acquire its deformation during concreting; this may dangerously modify its static equilibrium. A notable case of this occurred with a roof on which the curved girders overturned a wall on which they were supported so that when the centering was removed the wall broke and the whole structure collapsed (Fig. 58).

The same thing happened in a thin arch the centre line of which had been deformed as a result of poor formwork and differed considerably from the designed theoretical curve. This produced unforeseen bending moments and the structure broke. The shores were either lowered or raised indiscriminately and this resulted in instantaneous secondary stresses for which the structure had not been designed.

In the case of comparatively very flexible structures, for example cantilevered ones, it is essential to have an exact programme of centre stripping so that at all times the reaction of all the supports is in harmony with the progressive deformations.

Fig. 59 shows the roof of the Stade d'Honneur of Casablanca with 34 m overhang. In the course of centre stripping the elastic rises varied from 0 to 160 mm. The falsework was removed before the concrete had hardened sufficiently.

Thus it is a matter of elementary caution always to proceed as follows even in the main parts of a structure:

(a) Before removing the falsework check the strength of the concrete on the on-site control specimens.

(b) Make sure that there is no anomaly in the structure, or cracks or abnormal deformations in the formwork, etc.

(c) After conditions (a) and (b) have been met the centering or the shore heads should be lowered only about 1 to 3 cm below the reinforced concrete, where they will be left in place so that they can support the weight of the structure should the latter yield for any reason.

The so-called "saw cut" method, which consists in notching either a hollowed-out key or the top or bottom of the shores so as to bring about their gradual failure, may give good results provided it is carried out systematically (Fig. 60, a and b).

(d) If no unfavourable circumstances are noted after twelve to forty-eight hours, depending on the individual case, the centering or falsework can be removed with full confidence.

## C. VENEERS ON REINFORCED CONCRETE

Various failures are due to differences of thermal expansion or of shrinkage between reinforced concrete walls and their veneers, especially the following: pavings without sufficient joints have lifted off due to the compression stresses produced by the shrinkage of reinforced concrete floors.

In structures which undergo considerable temperature variations fired clay veneers have been loosened. These veneers generally have a much lower coefficient of thermal expansion than concrete and a considerably higher coefficient of elasticity, so that any change in temperature, even if uniform for both materials, produces, in effect, shearing stresses allowing their plane of contact to which may be added other stresses originating from the hardening shrinkage of the concrete (Fig. 61).

In such cases the veneers should be applied with extremely adhesive products, or they should be furnished with dowels or other anchoring devices extending into the concrete (stainless steel wires, etc.).

## D. CRACKS

A crack is almost always the cause of anxiety, at least temporarily, for the contractor. Moreover, the inadequately documented opinions of experts from time to time brought technical and legal consequences out of all proportion with the true import of the facts, which still further increases the fears of builders in the presence of cracking above and beyond any actual technical danger.

In designing in reinforced concrete one should normally abstract the tensile strength of the concrete in all parts where cracks are likely to occur. If these appear the structure is simply realizing the hypotheses made by its designer. Strictly speaking, no accident is involved here.

In principle, a crack is only dangerous in the following two cases: (1) if it is wide enough to permit oxidation of the reinforcements; (2) if it is not crossed by rods capable of resisting, by themselves, all the tensile stresses imposed on it in its plane.

In most cases oxidation of the metals scarcely ever occurs in the absence of aggressive gases or vapours unless the width of the crack at the surface exceeds 0.5 mm. Below this figure the cracks usually end before reaching the reinforcement.

In case of doubt there are various methods of measuring the effective depth of a crack, especially the following: (1) An alcoholic solution of phenol phthalein is injected with a Pravaz syringe into the crack. This solution has very low surface tension and acts as a colour reagent in alkaline medium. It turns pink on contact with freshly broken concrete. (2) The tendency of sound to to propagate in a straight line is employed to find the shortest route from a transmitter to a receiver in a homogeneous material. Fig. 62 shows that if we know the velocity of sound in the healthy material a measurement made in the cracked region will immediately establish the depth of the crack with the aid of an elementary geometrical construction. In a composite material, of course, a more refined interpretation is needed. Certain laboratories in France specialize in measurements of this kind.

## E. FOUNDATIONS

Leaving aside the problem of reinforced concrete as such, the chapter on foundations is perhaps the most important one from the point of view of pathology.

Until the last century the number of different types of foundations was very limited and they were generally designed by experienced specialists who had long been familiar with the regions in which they were working. Accidents, therefore, appeared to be comparatively rare. It is possible, however, that they were merely less advertised since, although every accident is extremely instructive to anyone who knows how to interpret it, there is never any urge on the part of the person responsible for it to talk about it, and this may explain certain gaps in our information.

It is unfortunate that the curiosity which people often show for things that do not concern them is not always exercised where architects and engineers are concerned in connection with foundation soils. In fact, a considerable proportion of the miscalculations that do occur are the results of inexcusable ignorance about the precise nature of the soil and subsoil on which the structure is to be supported.

During one of my inspections, for example, I was surprised to see the Court expert, although he enjoyed a certain fame, poke into a piece of ground that he was observing for the first time with his cane and declare with authority that "this soil will support a load of 4 kg per $cm^2$ with perfect safety". On digging the excavation I discovered that the ground in question, 150 m further down, was resting on a silty layer several metres thick without firmness. Many other incidents of a similar nature could be cited.

Unless he knows the ground completely no engineer can rightly judge of a foundation before he has taken a sufficient number of borings to an adequate depth.

Perhaps it is because the soil is not transparent, a psychologist once said, that too many engineers have the wrong idea about foundation problems and are thus led directly into errors that are often difficult to correct.

Above ground it may be possible, given due caution and common sense, to indulge in certain constructional acrobatics without serious danger. With foundations, however, there are generally too many unknowns and the risks are too great to justify any such audacity.

We shall now consider a few typical examples of accidents which could have been prevented.

## 1. Surface Foundations

This is the designation we give to shallow foundations generally.

When the ground is stable the question of the permissible pressure can be settled simply by a direct loading test. It often happens, however, that a top layer of firm ground itself rests on one of lower strength. A local test will not always disclose the danger because the test load is then distributed by the top layer over a larger area of the subjacent layer. In other words its intensity is not as great as that which will result from the structure itself. Therefore, boring is always necessary, even on ground which the builder professes to know well, because the nature of the subsoil may vary from place to place.

Our foundation soil laboratories are now able to determine with sufficient accuracy, on the basis of suitable samples, the time effect and order of magnitude of the soil settlements that can be expected now and even in the future, for there are of course settlements which become stabilized only after several years.

Now, the danger from settlements derives above all from the fact that they have not been foreseen. In certain regions, especially in North Africa, it is frequently necessary to build surface foundations on compressible or non-stabilized soils because the very strong layers are inaccessible by normal foundation methods. These conditions can be dealt with in various ways. For a large building in Tunisia, for example, a basement caisson was first provided, permitting the removal of a weight of ground equal to that of the building and with the same centre of gravity so as not to modify the general equilibrium of the subsoil. However, as heave of this hollow base layer could still take place in the course of construction, sandbags were furnished and used on several occasions as a ballast which could be moved as required.

It may be stated emphatically that large settlements of the order of one metre are entirely possible on certain ground.

When we were obliged to put in the foundation of the O.C.P. on a terrace in Casablanca which had just been finished and was not stabilized, we adopted two devices:

(a) We built the half-sunk repair galleries of short reinforced concrete sections joined together by special joints which would guarantee both strength and impermeability under quite high angular deformations (Fig. 63).

(b) We fitted the foundations of the supply tower with a lever arm by means of which the tower could be straightened up as required with the aid of screw jacks (Fig. 64).

The ground settlements actually did show differential subsidinces exceeding 1 m, without causing any difficulty.

In several installations we have employed posts that are freely articulated at the base in reinforced concrete boxes integrated into the foundation soil. Very simple jacking devices are used as required until the movements are practically stabilized.

This was done with a quay cover at Sfax, the various parts of which could undergo a mutual displacement without producing secondary stresses (Fig. 65).

A particularly complex case is that of an installation in a western port. The ground here consisted of a top layer of fill underlain by a thick layer of silt which is subject not only to large settlements but also to horizontal displacements due to river dredging operations, necessitating controls in both directions. The latter are realized by providing adequate play for the bases of the posts in boxes of the type described above. When this ground had to support a considerable load for the first time, the silty layer, compressed between the fill and the firm soil, poured into the river and broke the reinforced concrete piles of a loading platform. The silt had to be dredged out, the bed of the quay had to be reconstituted with sand and then the platform was rebuilt with thin piles driven into the firm soil in case the phenomenon should occur again at a greater distance.

For contemporary buildings, whether residential or industrial, the compressibility of the foundation soil is no obstacle to their erection. It is merely an additional datum of the problem to be solved and should be taken into account particularly by observing the following general measures:

(a) As indicated above, if possible excavate (cellar) a quantity of earth weighing approximately the same as that of the building with the centre of gravity situated along the same vertical line so as not to modify the equilibrium of the subsoil.

(b) The initial level of the building will be the final predicted level increased by the probable settlement predicted from the study of the ground.

(c) In calculating the various parts of the framing, basement floor, lining, internal and external partitions, posts, floors, etc., allowance should be made for the most unfavourable differences of level and inclinations resulting from this study.

(d) Outside windows, bow-windows, stair wells, etc., should be continuous with the framing and given cantilever support without direct transmission to the ground.

(e) Roadways, sidewalks, gutters, etc., should not have any rigid connection with the structure. Coverings to ensure impermeability of joints should be flexible or fixed only to one of the contiguous parts.

(f) All pipelines - water, gas, electricity, heating, etc., must be capable of withstanding without damage the predicted changes of level between the structure itself and the ground on which they rest. This condition can be realized by various methods, especially by the use of flexible or articulated devices.

(g) Filtering wells or piles - sand, pebbles, etc., - in certain cases will hasten the stabilization of the ground.

When foundations are being built for buildings involving refrigerating installations special precautions have to be taken if the soil is water-bearing.

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In this connection it is interesting to cite the case of an industrial building in the Paris area. This building included a central section which was higher than its two lateral wings (Fig. 66). These wings broke away from the central part. The foundations of the wings were blamed and were reinforced, but nevertheless the movements continued and increased.

It ultimately became clear that there was no settlement on the part of the lateral wings, but rather the central part was gradually rising with a continuous motion. The cause turned out to be a poorly insulated cold chamber kept at -30°C which was freezing the soil water and causing the central part of the building to heave from the well-known expansion of ice.

The reconstruction operations required special precautions. In order to avoid an inverse movement due to the thawing of the soil, which might have produced more damage than the 7 cm or so heaving that had already occurred, the central part was stabilized in its new position by digging wells below the frozen layer, after which measures were taken to produce gradual thawing.

In certain mining regions or areas containing old quarries, sudden settlements sometimes occur without warning. Especially in mining regions settlements sometimes vary in direction, in the course of time, as exploitation of the mines proceeds. This requires variable jacking devices for statically determinate structures.

A large number of borings to sufficient depths is absolutely essential under these very dangerous circumstances.

On clayey ground in hot climates, under buildings or the pavements of roads or runways, water sometimes accumulates and produces heaving due to the swelling of the clay.

## 2. Pile Foundations

Rather a large number of accidents or incidents have occurred in connection with pile foundations.

Many causes of these have been distinguished and they can be classified generally as follows:

(a) Insufficient knowledge of the nature of the soil down to the depth of penetration of the piles;

(b) Insufficient knowledge of the soil below this level;

(c) Errors in design;

(d) Errors in execution;

(e) Accidents or unforeseen circumstances;

(f) A change in the soil properties in the course of time;

(g) Special piles;

(h) Improper application of the methods of calculating the strength of piles.

(a) Insufficient knowledge of the nature of the soil down to the depth of penetration of the piles. In the great majority of cases the soils are not homogeneous. It is therefore always necessary to take a sufficient number of soundings in order to determine the variations of level, thickness and characteristics of the various layers encountered.

Soundings which are used only to determine these layers in general terms, e.g. sand, marl, clay, etc., are almost always inadequate, because soils grouped under the same name may often in practice have very different properties from the point of view of construction. What has to be known accurately are the special characteristics, e.g. the angle of internal friction, cohesion, etc. Testing for these properties demands the use of impermeable samplers by which specimens of undisturbed soil can be obtained for laboratory testing purposes. In certain cases soundings may be supplemented by cone penetrometer tests which give a direct and separate measure of the lateral friction resistance and the so-called point resistance.

Some of these apparatuses can be used to evaluate the resistance to withdrawal of anchor piles, a value which is too often overestimated by designers.

Of course, the results obtained with these apparatuses, which are generally of small relative diameter, must be properly interpreted before being applied to the piles themselves.

The driving in of bars can also yield useful information for comparison purposes.

In the absence of an accurate knowledge of the soil characteristics, the application of static formulae, even the best of which involve uncertainties, is likely to yield results of more or less precarious values.

In some cases scale model tests of piles can provide a useful guide for builders, provided they are correctly interpreted.

(b) <u>Insufficient knowledge of the soil below this level</u>. We are here touching a sensitive nerve in the pathology of foundations, because it has been a direct cause of many miscalculations, especially on the banks of the Seine and in North Africa.

Consider a single pile supporting a load of 50 tons, the tip of which is supported on a layer of gravel A, 2 m thick, resting in turn on a weaker soil B. Assuming arbitrarily that the load is distributed over a cone of 3 m diameter at the base, the pressure transmitted to soil B will be of the order of

$$\frac{50{,}000 \text{ kg}}{70{,}000 \text{ cm}^2} = 0.7 \text{ kg/cm}^2.$$

However, if in applying the results of the driving of a single pile we have (Fig. 68) a very large number of piles each supporting 50 tons and spaced 1 m apart from axis to axis in both directions, the load transmitted to soil B will become approximately

$$\frac{50{,}000 \text{ kg}}{10{,}000 \text{ cm}^2} = 5 \text{ kg/cm}^2,$$

i.e. about seven times that of the first case. To ensure that the strength of a group of n piles will be approximately n times the strength of a single pile it is absolutely necessary that the pressure transmitted to the subsoil shall not exceed the safe load on the latter.

On the banks of the Seine, where the compact limestone is generally covered with several alternating layers of various nature (clay, mud, gravel, fissured or thin limestone, etc.) many mistakes have been made which could have been avoided completely by a more thorough knowledge of the subsoil.

In North Africa an important structure involving piles cast in the soil, the first one of which showed excellent strength when tested initially under direct load, underwent very considerable settlement when all the piles were loaded. A few metres only below the strong layer in which the piles were lodged was a muddy layer which had not been revealed by the two shallow soundings.

(c) Errors in design. We shall cite three rather frequent cases of error:

The first concerns a reinforced concrete bridge of very large span. The foundation of each pier comprised a group of hooped concrete piles of square cross-section with 0.40 m side, separated by a totally inadequate space of equal size in both directions. These piles had been driven into a clayey-sandy soil.

The bearing load P per pile had been calculated by various static formulae and it had been assumed that the total strength of the group of n piles would be equal to n . P. When it was time for centre stripping the foundation began to yield under the load. When the settlement had reached 10 cm without showing any signs of stabilization, the jacks had to be reinstalled so that the bridge would again rest on its centering. A very costly reinforcement of the foundations then had to be undertaken.

Actually, the group of piles, being too close together, had acted as a single piercing element, so that the friction of the ground took effect only against the outside piles, i.e. with very reduced intensity (Fig. 69).

For a foundation in a clayey soil the builder had provided for reinforced concrete piles of octagonal section of 0.30 m inscribed diameter and 35 m long, i.e. too slender. When driven with a 6 ton ram the piles "whipped", enlarging their holes and considerably reducing the lateral friction against their shaft (Fig. 70).

As the piles were loaded too quickly, before the ground had had time to reestablish contact with the shafts, the latter sank abruptly by almost a metre under the standard load that had been provided for but was moreover not confirmed by the results of the last tallies of blows.

Slender piles should always be avoided regardless of the nature of the ground.

The third case has to do with a quay wall comprising a slab of reinforced concrete resting on three rows of piles, two of which were driven vertically and the third at an angle (Fig. 71). Under the thrust E of the soil the system of piles yielded and the wall collapsed.

In principle, if the force E were rigorously invariable in its position it would merely be necessary to drive in piles parallel to the force, their elastic centre being then directed against the latter (Fig. 72). In reality, however, supported soils may be subjected to variable surcharges on the one hand, while on the other hand the evaluation of soil thrust both in direction and in absolute value almost always involves some uncertainty. It is wise, therefore to employ a system of piles having three different slopes (Fig. 73). Using the quick, if inelegant, trial and error method, or any other method, a distribution of piles is found by which one utilizes as far as possible the available load on each row, because no row, except possibly in the case of unusual soils or particular structures, should be subjected to a dangerous lifting stress. These conditions should be verified, of course, under the most unfavourable conditions with respect to the action of the dead and accidental loads of the structure.

Most of the methods for determining the distribution of loads between the different rows of piles (Westergaard, Nökkentved, Verdeyen, etc.)* assume that the piles are comparable to rods articulated at both ends; they allow for their elastic deformations. Now, in reality, piles are generally partially fixed at their crowns and sometimes in the soil as well, so that they are subject in some degree to bending moments and shearing stresses. Nevertheless, various comparative studies appear to have demonstrated that it is generally possible to disregard these in the calculations provided, however, one does not come too close to the limits of the strength of the materials.

(d) Errors in execution. The use of too light a ram, the weight of which is definitely less than that of the piles being driven, falsifies the application of the so-called "Dutch" formula and leads to results which err by excess, i.e. in the direction of a reduction of safety.

The static formulae are based on the assumption that the ram is applied in free fall, a condition which it is wise to implement at least for the last volley of blows.

In steam drivers the final speed of fall may be reduced by too slow an evacuation of the steam or by incorrect operation. These reductions of speed, which normally amount to between 10 and 20%, may sometimes go as high as 50%.

Friction against the guide-poles, although generally small, must always be taken into account when the piles are driven at an angle.

The use of an 8 ton driver for piles 30 m long and 0.50 m in diameter, weighing 15 tons each, had very serious consequences in a harbour installation.

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* See: La mécanique du sol et fondation, by Verdeyen.

The application of the Dutch formula without any allowance had led to the conclusion that all the piles had found bedrock, as the refusals practically cancelled each other out at the end of the operation.

Now, a few years later, considerable settlements began and borings down to bedrock, when compared with the driving records, showed that all the piles were "floating", their tips having stopped some 2 to 3.50 m short of the bedrock level.

In a factory in Normandy, piles had been cast in the soil with a metal pipe, the tip of which was improperly sealed so that when the concrete was introduced into the water-filled pipe it was washed out; the bottom of the pile thus became nothing more than a well filled with pebbles.

Under the load of the building the lateral pressure of the pebbles compressed the silty clay through which the piles passed and the latter gradually subsided without shocks as they flattened out. After a year the building had undergone lateral warping to the extent of 0.26 m, i.e. there was a settlement of 0.04 m on one side and 0.3 m on the other.

To remedy the situation a completely new foundation had to be installed, then the structure had to be raised with jacks and stabilized. This operation was carried out, moreover, without interrupting the operation of the plant and without any incident resulting from it.

On an Algerian farm, piles of the same type had been made using soft chalk as the aggregate which when pulverized in the rammer would not allow the cement to penetrate. When a pile broke in testing before it had received its normal load, reinforced concrete piles had to be driven between the cast piles to guard against the latter's deficiencies.

In several instances reinforced concrete piles have quickly become useless either because the cements employed were unstable in the presence of chemically pure, saline or sulphated water, or especially because of the lack of compactness on the part of the concrete or because of reinforcements being too close to the surface.

In tidal river ports disaggregation or decomposition phenomena generally begin to show up in the tidal zone as a result of the mechanical action of ebb and flow of the water which enhance the effect of acids or other industrial products that rise to the top because of their low density.

(e) Accidents or unforeseen circumstances. In a mining area the foundation of a building had been constructed on piles driven into a fill of poorly quenched slag. A spontaneous combustion took place which reduced the strength of the concrete and reinforcements to the point where considerable settlement took place within less than two years. After shoring up the building the slag had to be removed by stages for a certain distance around it down to the good soil, then a new foundation was installed, the slag was replaced by sand, and finally the structure was raised and repaired.

The following relates to a foundation on piles for an industrial installation in a neighbouring country. Due to accidental circumstances the superstructure was not built until several years later. Fortunately, before resuming the work the builders had the good sense to test a pile by direct loading. This showed a comparatively small strength.

What had happened was that the foundation had been built on the site of a former sulphuric acid factory and the impregnated soil had caused the disintegration of the concrete and reinforcements of the piles. As in the previous case the piles and earth had to be removed down to normal soil, protection had to be provided against neighbouring ground and a fill of sand was necessary before proceeding with new foundations.

It cannot be too strongly emphasized that one should always enquire into the history of the ground on which a structure is to be built.

In another instance several reinforced concrete piles of a structure had shown refusal at lengths considerably less than those of the neighbouring piles and then had undergone settlements several years later. Investigation showed that these piles had broken in the ground during ramming and their initial resistance was due to the lower part functioning as a distributing member (Fig. 74). Their subsequent failure resulted from the oxidation and rupture of the reinforcements joining the two pieces at right angles to the break.

(f) A change in the soil properties in the course of time. Some changes are of an accidental nature. Such is the case of a foundation executed in a normally dry clayey soil where the later rerouting of a stream for a canal which was not sufficiently impermeable moistened this soil and after a few months produced disastrous settlements. Apart from truly accidental cases, the pathology of pile foundations owes a great deal to clayey and silty soils. A few examples will indicate the scope:

When piles are being driven it is sometimes found that the apparent resistance, as evaluated by a dynamic formula, gradually decreases right up to the end of the operation. After a shutdown of a few days, however, an often substantial increase of resistance is noted. Actually the adherence of the soil to the piles, partially destroyed by driving (whipping, vibration, etc.) has been reestablished by the subsequent consolidation of the clay. The increase of friction resistance in certain clays may be as much as 50% after two years. The reason for this increase appears to be first the consolidation of the soil due to the centrifugal motion of the ground water producing an overpressure in the vicinity of the piles and secondly self-consolidation due to thixotropy.

In other cases, however, it has been found that piles which have shown the desired refusal subsequently sink owing to a decompression of the soil.

In soils which have not yet been consolidated, fills, for example, settlement produces negative frictions against the shafts of the piles which adds to their load and can produce settlements after driving or drilling.

Piles cast without the use of lost forms sometimes moisten and soften the clay, whereas driven piles of porous texture pump water from the soil and hasten its consolidation.

Generally speaking these are instantaneous phenomena, but some of them have resulted in errors.

It may be concluded from what has been said above that it is always wise in the presence of clayey and silty soils to conduct careful laboratory tests with a view to estimating the extent to which settlements may possibly be aggravated in the course of time. In assessing the test results a count must be taken for the method of extracting bore samples, and whether the tiles are to be installed by drilling or driving.

With cohesive soils it sometimes happens that some of the soil gets firmly attached to the piles during the driving operation, thus increasing their inertia and falsifying the results obtained from the Dutch formula, with a consequent reduction of safety. Several serious errors have been traced to this cause.

(g) Special piles. Anchored piles which are subject to tensile stresses at top and bottom have given way in several cases because their strength had been overestimated. Some builders have assumed, in fact, that the resistance of a pile to withdrawal is a constant fraction of its resistance to sinking. This hypothesis may be reasonable for piles for which the latter value is due solely to friction against the shaft, i.e. where the tip rests in a layer of low relative strength. However, one need only point to the case of piles resting at the tip on bedrock covered with liquid mud. Here the resistance to withdrawal would scarcely exceed the weight of the pile itself. It is obvious, therefore, that the assumption of a constant relationship between withdrawal and sinking cannot be generalized. Direct tests or adequate inspection of the soil are indispensable in all cases.

A number of engineers recommend the use of cone-shaped piles in order to obtain results superior to those of cylindrical piles. Model tests carried out by the author on various substances representing a homogeneous soil, sometimes powdery and sometimes cohesive, have led to the following conclusions* (Fig. 75).

The resistance to sinking of conical piles is less than that of cylindrical ones of equal volume. In no case did it exceed that of cylindrical piles of equal contact area.

If a pile of any shape be joined to a surface slab, the same tests have shown that the total strength of the assembly is less than the sum of the two parts considered separately.

Screw piles, which exist in various types both in metal and reinforced concrete, are particularly advantageous for floating foundations or anchorages.

In many cases disappointment could certainly have been avoided by the use of direct or model tests.

(h) Improper application of the methods of calculating the strength of piles. Whether piles are installed by drill, by screw jacks, pile driver or screwing there is no method of calculating their strength which gives accurate results in all cases. In practice, soils that remain homogeneous over the entire depth involved are very rare and ground water levels are almost always variable.

In our opinion the best so called static formulae can be safely applied only in soils which are stable over a period of time and the results of direct tests can be extrapolated only to very similar cases.

In the absence of these tests, obtained by direct loading and extended until practical stabilization of settlements, these formulae must only be applied as estimates in preliminary planning or for the purpose of determining the initial length to be given to the test piles.

The "dynamic" formulae from the outset appear more trustworthy because they take certain experimental factors into account. Nevertheless they, too, must be applied with discretion and the final

* See Le Génie Civil of 12th September 1932, "Influence de la forme sur la résistance des pieux flottants" (Effective shape on the strength of friction piles), by Henry Lossier.

tallies of blows must always be realized with free fall of the ram, as detailed above.

Some real progress has been made, or is on the point of being made, in taking into account the elastic and plastic refusals, measurements of rate of sinking, etc.

In principle real safety will always require the following: (1) direct loading tests of sufficient duration; (2) examination of the soil below the tip of the piles extended over a wide area.

For relatively high, monolithic buildings of several storeys the loads tend to become distributed over points where the foundations are strongest or most rigid. This should be taken into account especially for the corner uprights, which by this reason may support loads greater than those resulting from the regular calculations.

Finally, the amplitude of the settlements under normal load and especially the inequalities of settlement between various piles must be taken into account without any optimism in studying the structures to be supported.

In many cases the question of shocks transmitted to buildings close to the sites of pile driving operations will arise in certain soils. Precautions must then be taken to avoid unjustified claims on the part of the owners of these buildings. First of all, before starting the driving operations, the state of these buildings should be examined for existing cracks, etc. Secondly, the intensity of soil vibrations should be measured at various distances from the point at which a tested pile is driven.

In the vicinity of Bordeaux we made measurements of this kind with a specially designed Philips electromagnetic vibration tester enabling us to measure the amplitude and frequency of the vibrations. In order to obtain a quantitative estimate of the danger from the vibrations for nearby buildings we resorted, in the absence of a French documentation, to the plan laid down by Schultze-Muhs*, by which we were able to define the zone in which

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* See Bodenuntersuchungen für Ingenieur Bauten, page 48.

buildings were endangered, where we then proceeded to install piles by drilling instead of driving, or to take special precautions.

### 3. Compressed Air Foundations

Accidents occurring in compressed air foundations are comparitively rare because these structures are almost always executed by specialized and experienced firms. The following, however, have occurred:

Tilting and even overturning of caissons, either due to poor workmanship or to inadequate study of the stability conditions in the course of various phases of sinking, taking into account the effect of ships passing; sudden drops on passing through a soft or muddy layer; tilting or displacement in the course of service, etc.

We shall therefore cite the case of a type of caisson used, where all precautions were successfully taken in order to avoid these various dangers. The problem was to lay five piers which would support the rail of a very long gantry on the banks of the Seine. From top to bottom the soil composition was as follows:

5 metres of fill;
11 metres of more or less liquid mud;
3 metres of sand and gravel;
4 metres of fissured chalk;
23 metres of compact chalk.

The following measures were taken (Fig. 76):

(a) To prevent the crumbling of the fill towards the river, which had already occurred near the installation, from shifting the piers, the width of the latter was reduced to 2.50 m in the unstable, muddy zones so as to perform the function of a cutting blade where the need arose. Furthermore, the foundation, comprising a caisson of 8 m · 14 m base was cut away in the direction of the river.

(b) To enable the workmen to take cover in the case of a sudden collapse during the sinking, the specially designed caisson (Fig. 77) was given an inside height in its central part of 5.60 m.

(c) Above the cutter of the caisson a horizontal, open framing was provided, the entrances to which could be closed independently of each other by movable, reinforced concrete tiles. By this means it was possible to vary the resistance of the soil from one side to the other so as to correct any leaning of the pier in the course of its descent.

(d) In order to limit the possibilities of sudden settlements during passage through the muddy layers the initial weight of the caisson was reduced to a minimum by the use of a double wall construction.

The space between these walls was filled with reinforced concrete as soon as the firm layers were encountered, so as to increase the strength of the working compartment.

## F. REINFORCEMENT OF STRUCTURES

We shall examine successively, with examples, the following items:

- mass concrete plates;
- floor beams;
- trusses (arches and tie-beams);
- uprights;
- circular reservoirs;
- bridges;
- underpinnings.

Mass concrete plates. Generally speaking, accidents due to the rupture of mass concrete plates are rare, since these elements have the benefit of both large sections and reinforcements of comparatively small diameter. This tends to protect them against danger from excessive beam and bond stresses.

In case of weakness they can be reinforced either by a coating of concrete with means of joining the two concretes, or, if one does not wish to increase their weight too much, with the aid of additional ribs.

Floor beams. The reinforcing devices differ, depending on whether the weakness comes from the longitudinal rods, the stirrups, the concrete or from several of these elements at once.

In the case of the longitudinal rods only, the method of cutting out the concrete so as to introduce additional rods alongside those already there, followed by reconcreting, can scarcely be recommended unless the structure is first unloaded with the aid of jacks during the hardening process and highly expansive cements are employed in order to reconstitute more or less the normal initial state.

It is also possible to introduce artificial stresses opposing the regular ones by using hard steel wires stressed with the aid of jacks, keys or by any equivalent procedures (Fig. 78).

In an eastern industrial installation where only the stirrups were weakened we fitted the beams with frames which were placed under tension with the aid of threaded bolts (Fig. 79). Tests to rupture on a reinforced beam completely justified this procedure.

Where the concrete alone is concerned we generally have recourse to artificial prestressing by means of elements pressing against the face under tension, or by pulling on the opposite face, or by external hooping.

Finally, if the weakness is due to several of these causes at once a post-stressing device can be used comprising oblique rods producing an artificial deflection in the sense opposite to that of the normal deflection of the structure, as explained in the section on bridges. Obviously, of course, it is also possible to add more beams, but generally speaking this is an inelegant solution.

In cases where looms or vibrating machines produce resonance phenomena even when shock absorbers are employed (cork, rubber, springs, etc.), sometimes only a slight modification of the floors or uprights is sufficient to suppress these, for example by the addition of a few cross-braces, the bolting down of a joint, etc.

In more serious cases inclined struts may be needed to counteract the horizontal displacements.

Trusses. The arches or upper chords of the trusses of arched roofs are usually reinforced by hooping with metal frames placed artificially under tension, or by encasing in reinforced concrete which either completely or partially envelopes them, or by interior sheathing, or finally by adding an auxiliary flange A (Fig. 80).

The use of an expansive concrete cement is recommended here in order to compensate in whole or in part for shrinkage during hardening.

Tie-beams, like braces, are generally reinforced by wires prestressed with the aid of keys or jacks on the tie-beams or side faces.

We may cite the case of the tie-beam of a bow-string truss of wide span the longitudinal reinforcements of which had been erroneously sectioned at right angles to the facing of the main supporting uprights without being bent back so as to provide an anchorage (Fig. 81).

The concrete had to be dug out for about 6 cm, then the end of each rod was threaded, iron supporting plates were inserted and screwed down with nuts.

In the case of Vierendeel beams in which either the cross bars or side bars are inadequately reinforced against bending, it is sometimes possible to add compression diagonals loaded with the aid of flat jacks or expansive cement keys E (Fig. 82).

Uprights. Uprights often have to be reinforced when additional height is added to a building that had not been provided for in the original plans. It is almost always carried out by hooping with steel hoops placed artificially under tension and then covered with a coating or with a complete reinforced concrete encasement, which offers the double advantage of adding its own strength while at the same time increasing that of the original upright by the overall hooping effect (Fig. 83).

Circular reservoirs. Vertical cracks in stressed zones are generally due to excessive stresses in curved reinforcements, which often may aggravate the shrinkage. Concrete can then be injected into the cracks from the outside while the reservoir is under load in order to open them up, or added external hoops can be placed under tension on the structure or an interior plastic coating can be applied. Certain special coatings, which give excellent results on thick-walled, rigid reservoirs are not so effective on comparatively elastic structures because of the comparative brittleness of these coatings.

Horizontal cracks may be due either to shrinkage, incomplete filling of a reservoir or the effect of the floor which prevents the deformations that the walls would otherwise undergo.

Post-stressing in the vertical direction is possible, but sometimes difficult to realize.

Generally speaking, after injection of the cracks the structure is protected by means of a fill against the effects of outside variations of temperature and the often more serious effect of solar radiation, which tends to pull it into an oval shape, thereby setting up secondary stresses in the vicinity of the floor.

In hot countries reinfroced concrete reservoirs are often protected against the direct effect of the sun by the provision of a brick facing, leaving a small space between the two walls.

Bridges. Massive arches can sometimes be reinforced by construction of a new arch against the intrados and placing the latter under artificial compressive stress with the aid of screw jacks or expanding keys (Fig. 84). Caisson arches are reinforced in the same manner, except that here the new structure is placed inside the caissons and is therefore invisible from the outside (Fig. 85). The functioning of some arches has been improved with the aid of compensating keys placed under load by means of jacks or by expansion at the key and at the abutments, in the manner indicated in Fig. 86.

An operation of this type was realized for a caisson arch of 100 m span (Fig. 87 a and b), where one pier of the arch had been accidentally displaced before the keying, and this resulted in a wide crack at right angles to one of the roots. By the careful insertion of an expansive cement key the normal functioning of the structure was completely reestablished.

Bow-strings, comprising arches, tie-beams and braces, are dealt with in the same manner as the trusses of arched roofs.

Box beam bridges of constant or variable height, with one or more spans, generally lend themselves well to einforcement based on adjustable post-stressing, a principle which we applied, for example, in the reconstruction of the bridge over the Seine at Villeneuve-Saint-Georges (Fig. 88). This consists in placing suspension bridge cables inside the caissons and loading them by means of screw jacks. They are arranged so as to reduce or completely annul the bending moment and shearing stresses due to the dead load (Fig. 89 and 90). Reinforced concrete or cast steel rods take up all the friction stresses produced during the tensioning of the cable (Fig. 91). These devices are permanently accessible to inspection and adjustment, so that any aggravation due to increases of useful load, as sometimes happens on our roads and railways on account of the use of continuously heavier vehicles, can be taken into account.

## G. REPAIR OF WAR-DAMAGED STRUCTURES

The problem of repairing breaches caused by bombs, shells or mines generally means reestablishing the former state of stress in the elements which have to be rebuilt.

Obviously, with ordinary structures this can be done by putting the entire construction under centering and balancing its own weight completely with the aid of jacks so that it can be repaired in the absence of all initial stress. This, however, is sometimes a difficult and expensive procedure.

For structures of reinforced or non-reinforced concrete the problem is more complex, for here it is a question of reestablishing the original stresses in the parts that were formerly compressed and at the same time taking into account the hardening shrinkage of the new concrete.

Among the various possible solutions - filling up of joints, screw jacks, etc. - the use of expansive cement seems particularly well suited to the purpose because it requires no special mechanical equipment, but only the action of the material itself.

From among the various cases dealt with by this method, we shall describe two typical examples:

(1) The repair of the Épinay-sur-Seine bridge of the S.N.C.F., a masonry structure with elliptical vaults.

(2) Repair of the Moscou bridge near Montereau (S.N.C.F.), a reinforced concrete beam with one-piece spans and caisson sections.

All these cases were dealt with using either slightly expansive or very expansive cements, or jacks, or both simultaneously.

Repair of the Épinay-sur-Seine bridge (S.N.C.F., northern sector). The masonry bridge for standard double track crosses the navigable arm of the Seine between Gennevilliers and Épinay on the line from Saint-Ouen to Ermont. It has three spans with intrados elliptical arches of 38.50 m open span and 1.35 m thickness at the key.

During the aerial bombardments of 1944-1945 the bridge was hit several times as shown in Fig. 92 and 93. In addition to various damage of a minor nature requiring no special repair measures, the span on the Gennevilliers side was pierced by a bomb about 6 m from the key in the direction of Épinay and approximately 1.50 m on the upstream side of the axis of the structure. The breach was small on the extrados but widened out toward the intrados where its apparent diameter was of the order of 4 m.

Taking into account the dislocation zones surrounding the breach, it appeared that the vault was practically sliced through over more than half its width on the upstream side. Since the navigation conditions were unfavourable for the construction of a temporary scaffolding in mid-river in order to carry out the necessary repairs, the erection of an upper scaffolding supported by the extrados of the arch itself from which the working platform could be suspended, had to be adopted (Fig. 94).

The two problems which presented themselves were then the stability of the arch during the repair operation and the method of repairing the breach.

Stability of the arch. The use of an arch which had been sliced through over more than half its width as a support for the repair operations might have been successful provided the various parts of the masonry were strong enough to resist the stress dislocations due to the presence of the breach. In view of the uncertainty of this, however, there was a serious risk involved and this solution had to be abandoned.

It was therefore decided to begin by partially reestablishing the static continuity of the arch at right angles to the breach by furnishing it with three partial, temporary, lower arches of reinforced concrete which had been subjected to precompression and which transmitted their thrust into the interior of the arch on both sides of the damaged zone.

As Fig. 95 and 96 show, these temporary arches have several radii of curvature and are placed at intervals of 1.75 m from axis to axis. They are about 22 m in length, 0.50 m wide and their height varies from 0.50 m in the centre to 0.70 m in their embedded sections. They are placed 0.10 m below the intrados to the right of the breach, in order to provide room for planks and shims, and by reason of their shape they tail into the bridge arch gradually at either end. Their reinforcements are strengthened in the vicinity of the breach in order to take account of any upward thrusts that might occur there as the result of the precompression. Anchorages A, which are longitudinally elastic, prevent lateral buckling of the arches. The loading of each of the arches, which might have to be varied in intensity during the repair operation, was accomplished with the aid of a screw jack having a minimum of 100 t force acting in the direction of the key of the bridge arch and which of necessity remained under stress until the repairs were completed. If the thrust to be exerted had been constant, the use of expansive concrete would have been the simplest solution.

The repairing of the breach. When the three partial, temporary arches had been installed and placed under load, the breach repairs were carried out in principle by the method applied to the Poix viaduct, that is to say by successive execution of five rings 1 m wide each, carried out in the order indicated in Fig. 96. The keys of expansive cement concrete measured 0.60 m to 0.70 m long and were separated by temporary joints about 10 mm wide, whereas the rings were jointed at their central parts.

Fig. 97 shows the repair concrete, the expansive key and the device for wetting the latter.

The repairing of the Moscou sur l'Yonne bridge near Montereau (S.N.C.F., southeast section). This structure was put up in 1941. Situated on the line from Flamboin to Montereau, it supports a standard double track and crosses the Yonne by four spans of 26 m and 26.4 m from axis to axis of the piers. Each track rests on a half deck of reinforced concrete of single span and having a box section of variable height.

During their retreat in 1944 the Germans tried to destory this bridge with charges placed in the upper part of the deck on the first span and above the first pier at the left bank and above the central pier. Serious damage occurred, but the deck held (Fig. 98). The damage was of two types:

(1) In the left bank span: breaches in the concrete deck beneath the track; breaches in the lower concrete deck; exposure and deformation of upper rods (Fig. 99); exposure of lower rods beneath the completely dislodged concrete;

(2) Above the piers: breach in the concrete deck below the track; upper rods exposed and deformed (Fig. 100); cracks in the ribs and supporting devices (struts).

Finally, part of the overhanging sidewalk was demolished.

The methods of repairs. The various parts of the structure were repaired in the following fashion:

For the breaches in the concrete decks and various cracks the edges were recut in order to remove dislocated parts from them and then the reinforcements were straightened or complemented and the missing section was restored with ordinary concrete or expansive cement concrete, depending on the individual case. The first method was applied to the downstream half deck, the second being used on certain parts of the upstream deck. Wide cracks were drawn together and small ones filled by injection either with ordinary cement or expansive cement. There still remained the exposed, deformed reinforcements. The longitudinal deck reinforcements consisted of rods 40 mm in diameter not hooked and uncrossed, i.e. with end-to-end joints, the number of rods conforming to the requirements of this method of achieving continuity. These rods had been bedded in the three 0.50 m ribs and the 0.65 m one (bridge side), their basic axial interval being 80 mm in both directions (Fig. 101).

These reinforcements were disposed at five levels over the intrados of the shore span, and five and three levels over the extrados, respectively, on the lateral and central piers.

Two difficulties were encountered. On the one hand there was the practical difficulty of straightening the parts of the 40 mm rods which were severely deformed by the explosion of the charges and which had been subjected to a partial hammer hardening. Secondly there was the inadequacy of the space available between the existing rods for the insertion of additional reinforcements under normal conditions.

Moreover, the cutting and removal of the bent parts of rods and the restoration of their continuity by the insertion of welded sections was a delicate and precarious operation under the circumstances.

Finally, the plan of reinforcing the deck by means of cables inside the box decks, which were then to be put under a variable stress by means of screw jacks, had to be discarded a priori not only because of the relatively high overloads, but also because of the excessive delays which would have occurred in waiting for the special parts required for this solution.

The simplest method, therefore, was to take advantage of the fact that each half deck had a caisson section for the accomodation of additional reinforcements between the ribs, that is to say in the extrados and intrados plates, which were reinforced as required for this purpose.

Naturally, of course, the concrete slab sections were given a sufficient thickness of concrete to ensure perfect cladding of the added longitudinal rods. Furthermore, transverse rods of suitable shape were placed in such a way as to transmit the stresses of the longitudinal rods to the ribs (Fig. 101).

The question arose as to what importance should be given to the installation of new longitudinal rods with reference to the deformed reinforcements that they were intended to assist. The complete removal of the latter seemed, on the evidence, to be unnecessary in view of the fact that they had already withstood the dead load of the bridge.

Furthermore, the various possible theoretical considerations were all more or less questionable. The solution which was adopted, therefore, was simply to attribute to the added rods the total maximum tensile stress, but assuming a unit stress that would exceed 25% of the standard official stress. There could certainly be no danger involved here.

Ties and stirrups were added in order to balance the stresses passing to the outside from the retained rods at right angles to their deformations.

The work schedule. Traffic was first reestablished on a single track on the upstream half deck over which metal beams were laid directly on a temporary pier beneath the left-bank span while the deck itself supported the track over the other spans. The repair work was then begun on the downstream half deck, which was partially shored up by the same pier, while the measures described above were being applied.

For the sake of appearance the repairing of the damaged part of the pedestrian walk on the overhang required brackets without extensions embedded in the interior of the concrete deck. Anchorages of various types, several of which involved the use of expansive cement, were employed.

## H. UNDERPINNING

We shall consider the following two examples: The Palais Rihour at Lille, and the Ministère des Colonies in Paris, both of which involved the use of expanisve cements.

Palais Rihour. This involved repairing the wall of an historic building, the Palais Rihour at Lille, the foundations of which were showing signs of weakness. The system selected was the Méga pile designed by the Franki Company.

It will be recalled that this pile consists of precast reinforced concrete elements which are assembled at the site. They are sunk with a jack. The difficult part of the operation after the pile has been sunk to the proper depth, is to place it under load beneath the wall in such a way as to assure its full contact with the wall. This is generally accomplished by using a special U-shaped piece, a jack, shimming devices and finally by shimming with the caulking chisel below the beam.

The use of expansive cement considerably simplified this operation.

A removable metal mould M was placed on top of the last section of each Méga pile (Fig. 102). The mould was fitted with a hopper E, for pouring in the concrete, and with a vibrator V.

After a light reinforcement had been placed in the mould the latter was filled and vibrated until the laitance of the concrete flowed out between the mould and the beam. After setting and removing the form the poured section was kept wet for the number of days required in order to obtain the initial desired thrust against the wall of approximately 20 tons per pile.

This method, compared to the first one, enabled us to obtain greater solidarity between the crown and the body of the pile and to realize a saving of about 20 kg of very intricate reinforcement per element.

Fig. 103, 104 and 105 show the various phases of the operation.

The holes visible in Fig. 105 permit the wetting water to penetrate into the interior of the expansive concrete. Actually, there is no expansion until the wetting is begun a few hours after the setting of the cement.

If this wetting is suddenly stopped in the course of expansion, the expansion itself will cease altogether in twenty-four to forty-eight hours, after a 10 to 40% elongation, beyond the value already reached, depending on the quantities employed. Thus the termination of wetting is a simple and effective means of controlling the expansion at the site.

The Ministère des Colonies. Fig. 106 is a diagram representing the underpinning of a wall.

On top of every footing of non-reinforced concrete we built a crown of expansive concrete 1 m in height. Wetting holes A, B and C, each 30 mm in diameter, placed at 20 cm intervals, were made with the aid of temporary steel bars placed in the concrete and withdrawn at the time of setting. They were connected up to a supply of water maintained by means of a simple fillet gutter. The force of expansion assured the loading of the wells under the wall without the aid of jacks.

It may be noted in passing that there are many possible applications of these principles in mining construction in filling around the vaults of galleries and tunnels.

## I. RULES FOR THE USE OF REINFORCED CONCRETE

Generally speaking, rules are not the progenitors of progress. On the other hand, they cannot be charged with having caused accidents. Their purpose is to avoid errors of doctrine and their effect is to stabilize for the moment the rules of application of a material until another rule can be formulated which will take into account any progress that has been made since the preceding one was drawn up.

Now, if a rule remains in force for a long time and the progress of a science is rapid, the result may be a more or less serious lag in the practical application of the new knowledge. This is the pathological side of the question.

There are two possible solutions. On the one hand there may be frequent revision of the rules in order to keep them constantly up to date, or on the other hand they can be rather liberally formulated to enable builders to keep up with technical progress.

The first of these two solutions has been generally adopted abroad. The second one, on the other hand, inspired the "Circulaire française" of 1906 which was not replaced until 1934, i.e. twenty-eight years afterwards. Since it authorized stress values which were no longer fixed, but were proportional to the real characteristics of the materials, builders and industrialists were thus encouraged to search for products of higher quality, and hence towards the realization of bolder structures. It is due in part to the liberal character of the Circulaire of 1906, inspired by the genius of Considère, that French builders have for many years held a distinct lead over some of their foreign colleagues who were too narrowly restricted by more rigid rules. Although known and proposed almost fifty years ago, the use of bars with a surface opposing their slippage in the concrete has in particular been abandoned for several years solely because certain rules did not permit builders to take advantage of them even though conclusive tests had made these apparent.

While any good rule may help to prevent imprudent procedure, it should never be allowed to detract from the initiative of a good builder or to impede progress.

Finally, in any country the situation should be avoided where different administrations each adopt a rule which while good in itself agrees only partially with general prescriptions laid down in other similar rules.

Present-day knowledge enables us, broadly speaking, to bring the conceptions of various administrative divisions of a single country into harmony with each other. The same should apply to the rules applied in different countries where the authorizing committees too often work in isolation and in more or less complete ignorance of what is going on in neighbouring countries.

## J. A FEW EXAMPLES OF FALSE REASONING

(a) Calculation of stirrups. Some engineers assign to the stirrups only the excess of the shearing stress, or more exactly the tensile stress, which the concrete is able to balance. In reality, however, vertical stirrups really become effective only after the concrete has cracked. They must therefore be capable of completely replacing the concrete, resisting, by tension, all the tensile stresses of the concrete acting over the entire area occupied by a given crack.

(b) Consideration of concrete under tensile stress. In special cases structures which are executed without shrinkage joints and which always function in isolation the tensile strength of the concrete may be taken into account, but this must always be done with great caution.

In other cases where shrinkage joints may be employed, or where there are liaison stresses with other parts, (shrinkage, temperature, fitting stresses, etc.) the reinforcement must be capable by itself of absorbing all the tensile stresses.

To act otherwise involves the risk of reducing the margin of safety of a structure by increasing the stress on the steel in zones where the tensile strength of the concrete is partially or completely absorbed by the action of secondary stresses or by the effect of continuity solutions.

(c) Shearing strength of the concrete. So-called "shearing cracks" in concrete are actually tensile cracks.

Some builders believe that if the secondary reinforcement is capable of balancing the shearing stress, the shearing strength of the concrete is of no importance from the point of view of the total strength. In reality, the opening up of all cracks should be restricted so as to avoid oxidation of the reinforcements by outside agents.

Vertical stirrups are of little help in preventing cracks.

So as to be able to assume "strength values" which certain rules fix at up to half the value of the compression in various cases where oblique rods are employed, the spacing of these rods should be limited as a function both of the height of the beams and of the diameter of the rods themselves, sometimes even in absolute values.

Special measures may be considered in zones where maximum bending moments and shearing stresses occur simultaneously. They must always be based on the results of really conclusive experimental data and not on questionable theoretical considerations.

## K. FIRST STEPS TO BE TAKEN IN INVESTIGATING AN ACCIDENT TO A STRUCTURE

The most urgent matters confronting an expert in the case of an accident are the interrogation of witnesses and the examination of the ruins.

(a) Interrogation of witnesses. Anyone who has ever interrogated the witnesses to any event will know that each one sees it from a different point of view.

They will also know that between a first deposition and following ones given by the same witness differences will arise which will be more or less pronounced depending on whether the witness has been subjected in the meantime to the influence of other witnesses and possibly, indeed, to certain pressures from outside.

The initial interrogations should therefore be begun as far as possible immediately after the accident, at least as far as the main points are concerned. Delays can only hinder the acquisition of an exact picture of the facts.

(b) Examination of ruins. If, say, in a building of post and beam construction a given part is brusquely removed so as to produce a partial or total collapse of the structure, then each of the other elements will occupy a definite, predetermined position of fall. There is thus a direct relationship between the arrangement of the parts that have collapsed and the identity of the one which produced the failure.

From this it may be concluded in principle that the examination of ruins by an experienced expert should reveal the elements, the failure of which was responsible for the collapse of the structure. Every expert, therefore, should familiarize himself with these cause and effect relationships, especially with the aid of suitable models.

In practice, however, such studies often present difficulties and sometimes even problems impossible of solution. For example, where there have been casualties certain parts may have been displaced by the rescue operations. If a number of ceilings of a building have collapsed they may be superimposed on each other in such a manner as to render all examination very hazardous. Finally, reinforced concrete structures are in general more unfavourable from this point of view than steel constructions because of their fragmentation in the case of collapse.

Nevertheless, a visual examination by an expert of the ruins of a structure will almost always give him much more valuable information than he could obtain from the best photographs, even a great many of them taken in strategically chosen places. Such photographs are nonetheless indispensable in all cases.

It is for these reasons that this careful personal examination appears to me to be a matter of prime importance. Moreover it is one that frequently goes unrecognized.

Without wishing to minimize the utility of all the other aspects of a thorough investigation, it may still be said that the intelligent observation of ruins has more than once pointed the way from the very start to the means of finding a solution for even the most complex of problems.

## L. THE MISDEEDS OF CERTAIN ENGINEERING EXPERTS

In addition to failures attributable to the errors of the planners or of the contractors on the one hand, and to major forces on the other, we have on more than one occasion had to deplore the aggravation of damage produced by the initiative, and sometimes the lack of initiative, of the experts called in to advise on the measures to be taken in order to restrict the damage.

When the designated expert is a specialist of incontestable reputation who has proven himself and who enjoys complete independence from interested parties, his intervention can only have fortunate results. This is generally the case.

It sometimes happens, however, that investigations are entrusted to inadequately specialized engineers who, no doubt in good faith, believe themselves capable of solving any question whatsoever.

I am sometimes surprised at the inadequacy of the documentation placed at the disposal of the officials who must choose the experts and at the thoughtlessness of those among the latter who will accept a commission which they are in fact incapable of carrying out properly. The results may be delays or errors in the making of decisions, the incurring of needless expense, or even the aggravation, sometimes in a dangerous manner, of the original damage.

In several publications and lectures I have already cited a number of examples of this nature. I shall reiterate here a few typical cases, which I have disguised somewhat so as not to expose the responsible parties.

(a) A factory building had been very seriously cracked because of the failure of the piers on which it was founded. The concrete of these piers had been decomposed by aggressive waters from a nearby chemical factory which had not taken the precaution of channelling them. What did the expert do? He ordered the restoration of the foundations without taking any steps to remove the cause of the trouble. After this work had been carried out by underpinning, that is to say under the most difficult of conditions, he came to the conclusion that the building itself would have to be torn down, a possibility that he had failed to consider up until that time. Thus, due to the intervention of "professional", six months and several millions of francs were lost at a time when the franc still stood at a high rate of exchange.

(b) When the strength of a reinforced concrete building in the north was questioned by a rival engineer, the owner became worried and called in a specialist - in structural steel. After examining the plans and making calculations this expert declared that it would be necessary to carry out a partial demolition of certain parts so as to add additional reinforcements. Unaware, no doubt, that refilling with green concrete in a concrete that had already hardened is rendered largely ineffective by reason of the shrinkage (a phenomenon that is unkown in structural steel work), and despite the protests of the contracting architect and the builder, the expert was told to proceed with these "reinforcements". A few months later the structure, having been weakened by the partial demolitions of concrete to which it had been subjected, cracked severely in several of these "reinforced" parts and had to be reconsolidated by more appropriate methods.

(c) An entire factory comprising several buildings built on a hill became subject to progressive cracking. For four years a "expert" had local repairs carried out, without in any way retarding the dislocation processes to which the buildings were subject. When a specialist was finally called in he discovered that a subterranean stream was undermining the foundation soil so that the latter was undergoing a general sliding process. He had the spring tapped and immediately the ground began to stabilize. However during the four years of neglect the damage to some of the buildings had gone so far that four of them had to be torn down completely and rebuilt.

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It would be altogether wrong and unjust to conclude from the foregoing that a large number of experts do not know their job. The great majority of them are unquestionably competent and conscientious enough to accept only commissions for which they are well qualified. However, just as there are good and bad doctors of medicine there are also good and bad engineers and one must know how to choose between them. It would no doubt be sufficient if the officials responsible for selecting the experts in each particular case were more fully cognizant of the skill and speciality of each in order to avoid initial errors of orientation which may be prejudicial both to the public interest and to that of the parties directly involved.

## M. PRESENT OUTLOOK FOR REINFORCED CONCRETE

The realization of a world span record for a given type of structure does not by itself prove that its designer overshadows in skill all his other colleagues. In every country today there are a certain number of technicians who, had they been confronted with the problem would under normal conditions have been able to solve it. Very often, in fact, the choice of a project and a designer to realize it is decided on the basis of cost or time. Generally speaking, today's world records are already far surpassed by the available possibilities.

Only rarely does the width of our rivers require spans of the order of more than 250 m, except on the estuaries or in order to cross valleys at high altitude.

From the technical point of view, indeed, the width of spans is limited only by the dead weight and the strength of materials. From this point of view alone, we could contemplate, for example, arches of approximately 1,500 m and bow-strings of 500 m open span, i.e. six and four times, respectively, those of the largest existing structures (Fig. 107).

With regard to heights we may cite the plan of a 2,000 m high tower presented in 1935 by the architect Faure-Dujarric and the author, represented in Fig. 108 and 109. The body of the structure, to be executed entirely in reinforced concrete, had the shape of a truncated cone 40 m in diameter at its peak and 210 m at the ground level. Its thickness ratio, top to bottom was 1 : 12 m. Its foundation, resting on chalk, 20 m deep was designed as a circular footing 400 m in diameter.

This structure supported three platforms at altitudes respectively of 600, 1200 and 1800 m from the ground. The platforms were to measure respectively 357, 400 and 450 m in diameter. Each platform comprised radiating, reticulated metal girders which act as a framework for the main floor, intermediate floors and the roofing, and secondly conical vaults of reinforced concrete resting on metal girders. The latter could also have been executed in prestressed concrete.

This design, originally presented as a defence work for Paris against aerial attack, was later considered for purely civilian uses (sports, aviation, radio, television, astronomy, meteorology, hotels, sanatoria, etc.) in collaboration with the best qualified French and foreign experts.

In addition to stairways and high-speed elevators a helical automobile ramp would lead from the bottom to the three platforms.

In round figures this structure would require 5,000,000 $m^3$ of sand, gravel and stone, 1,200,000 tons of cement, 300,000 tons of round mild steel rods, 300,000 tons of structural steel framing, 4,200,000 $m^2$ of wood and metal falsework, and 3,000,000 $m^3$ earthworks. The building time was estimated at three to four years.

## N. PRESTRESSED, POST-STRESSED AND SELF-STRESSING CONCRETE

Some readers may wonder why we have not considered the pathology of prestressed, post-stressed and self-stressing (expansive cements) concrete, etc., in the present work. There are three reasons for this.

In the first place, the application of these materials is comparatively new and most of their initial difficulties are of a temporary nature.

Secondly they are still being used by, or are under the control of eminent specialists holding patents on them, and this in itself is a guarantee against errors due to ignorance.

Finally, we must have the assurance, and only time can give us this, that these conceptions will not have to allow for as yet unsuspected processes. Only then will it be possible to judge them in a definitive manner.

## CONCLUSION

Although reinforced concrete has been known for about a century, its general application dates back only about fifty years. Nevertheless reinforced concrete is not only assured of a place in the first rank among constructional materials, but appears today to be in a period of full evolution towards new progress.

In studying the pathology of reinforced concrete I had no thought of undermining the fully justified confidence in it that so many engineers have. On the contrary, I have sought to show, in what is perhaps an unusual way, first of all that none of the known miscalculations which have entered into certain of its applications is the result of any fundamental fault of the material itself, and secondly that our present knowledge, although still open to improvement, is already adequate to permit any engineer possessed of a healthy logic and a robust common sense to employ it without incurring any risks other than those due to major forces, risks which are inevitable in all fields of construction. In emphasizing both the pitfalls which must be avoided and the very simple means of doing so with certainty, I have again affirmed my profound faith in this remarkable material.

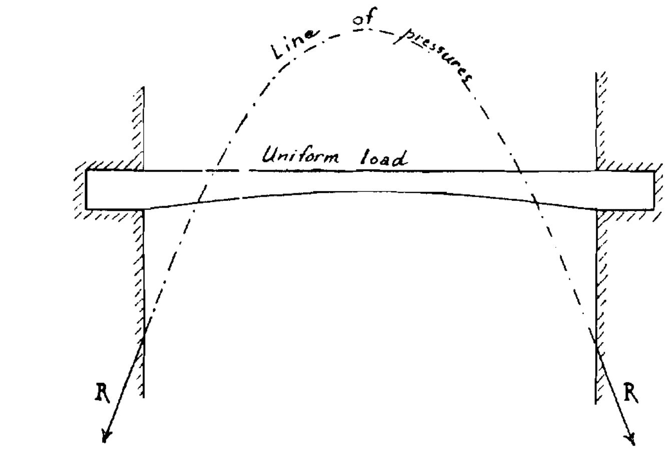
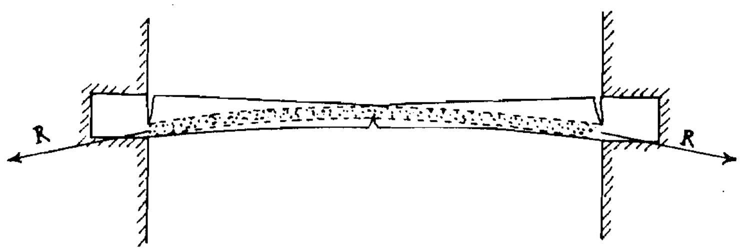



Fig. 1

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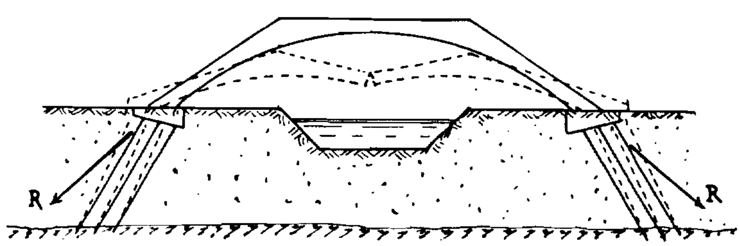


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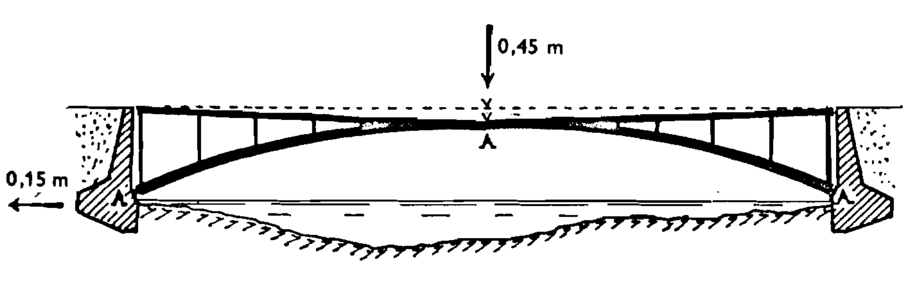
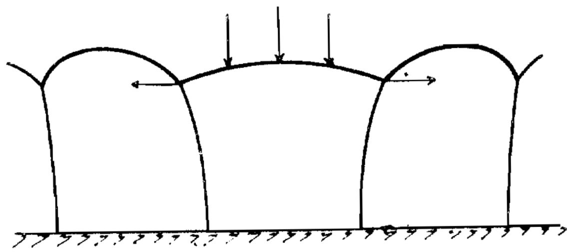



A : Articulations

Fig. 4



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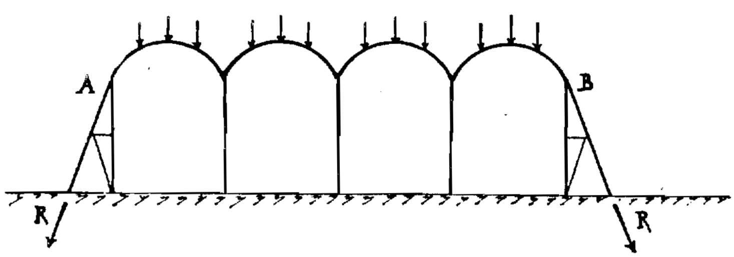


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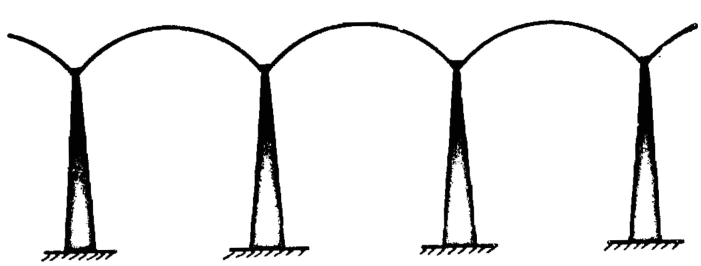

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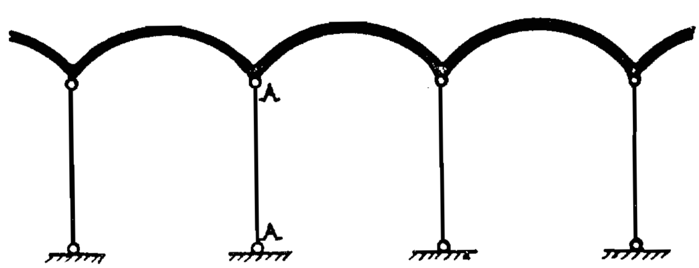
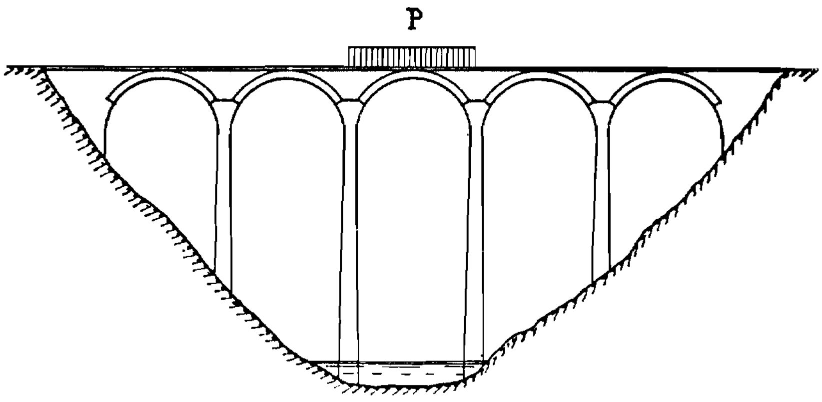



A: Articulations

Fig. 8

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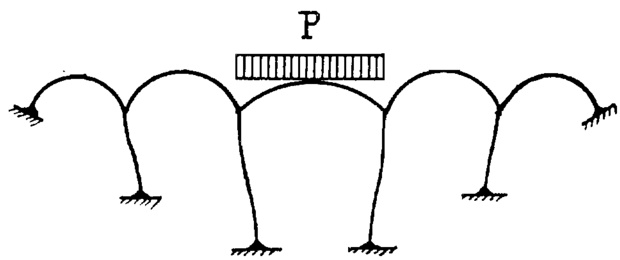


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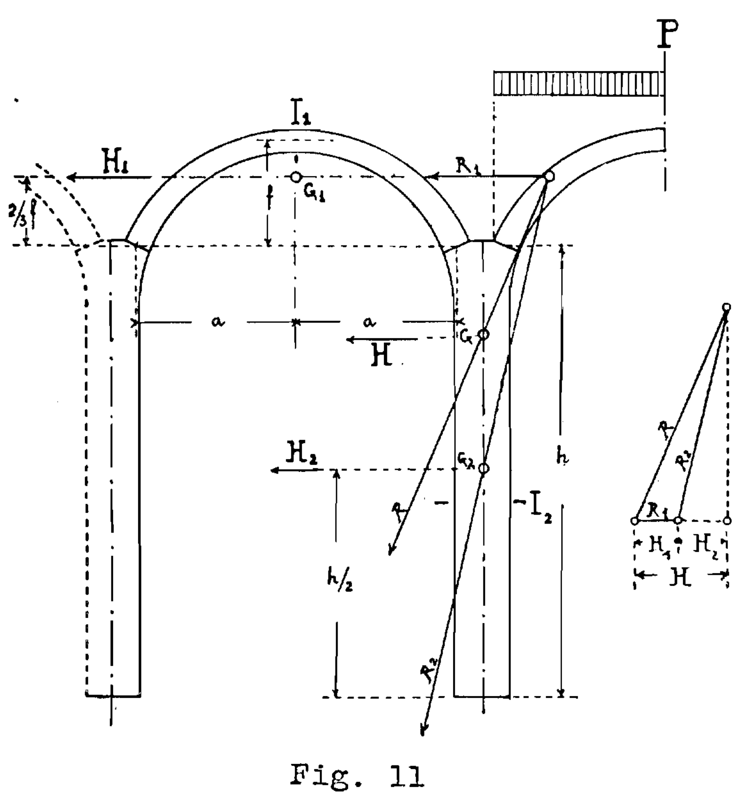


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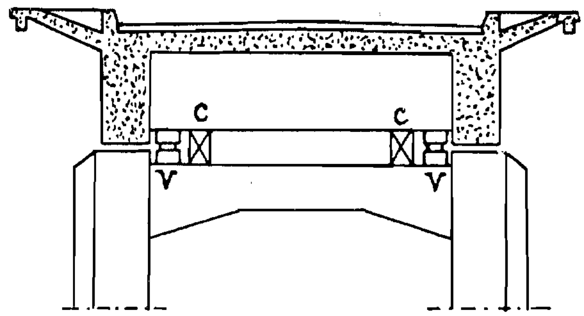



V: Jacks C: Shims

Fig. 12

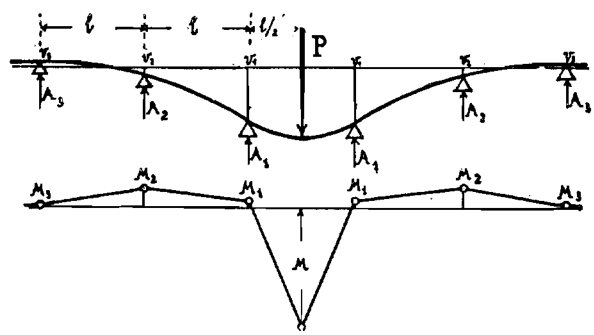



Fig. 13

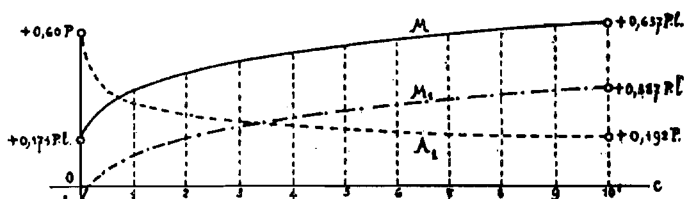
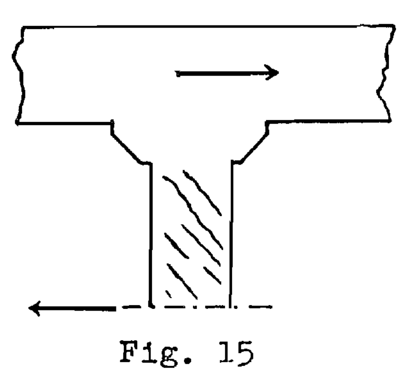



Fig. 14



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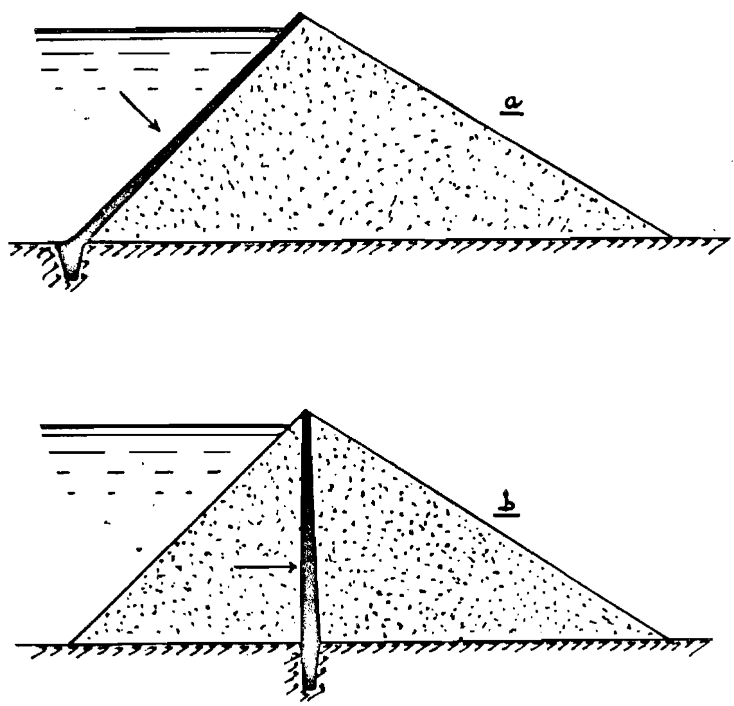



Fig. 16

(a) Shield on upstream wall. (b) Vertical shield

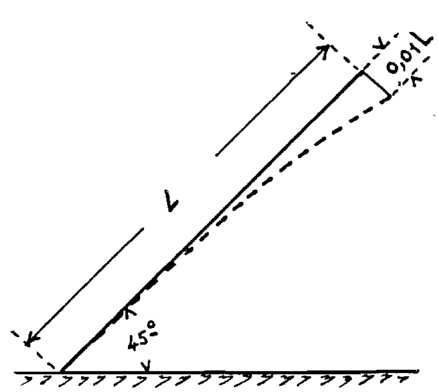
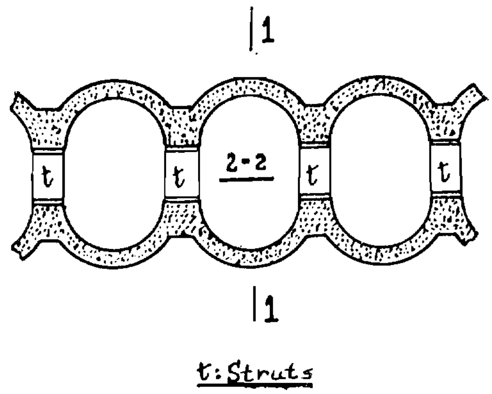



Fig. 17

Deformation of upstream wall
of a riprap dam



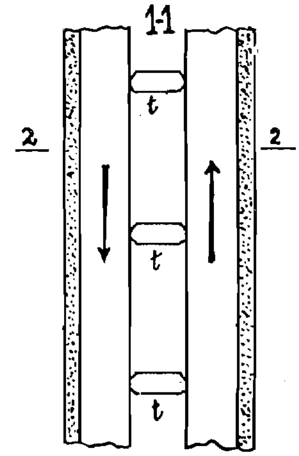




Fig. 18

Hollow shield with independent walls. Henry Lossier type

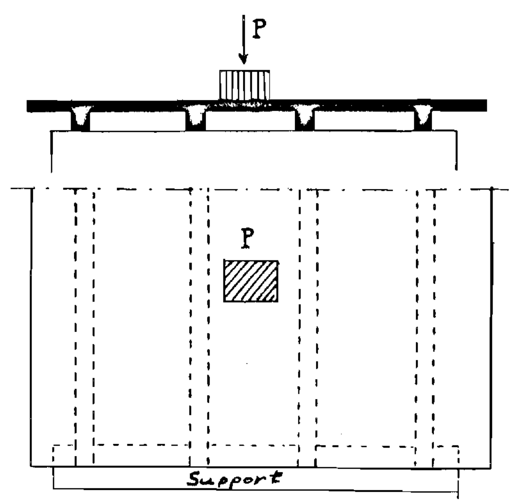



Fig. 19

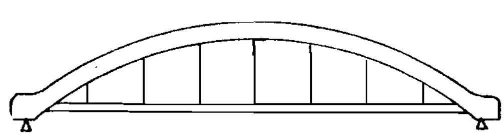


Fig. 20

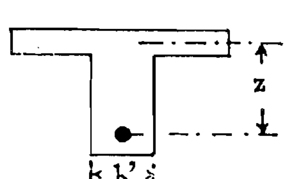



Fig. 21

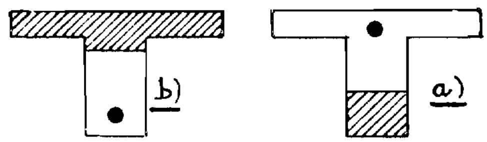



Fig. 22

(a) Section between supports. (b) Section over supports

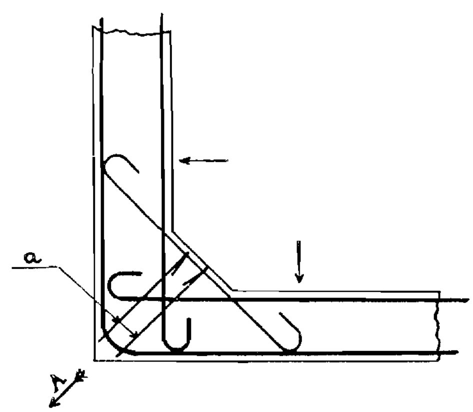
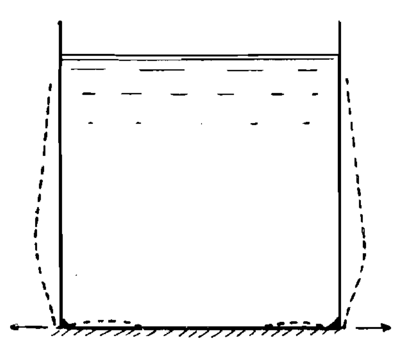



Fig. 23



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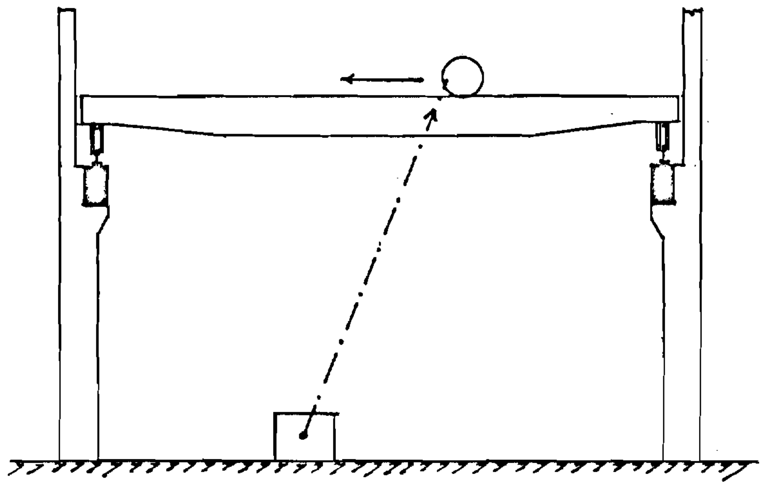

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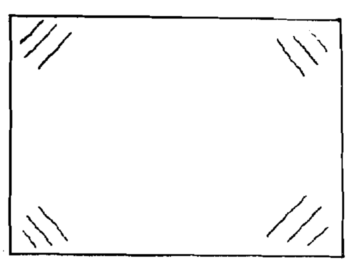

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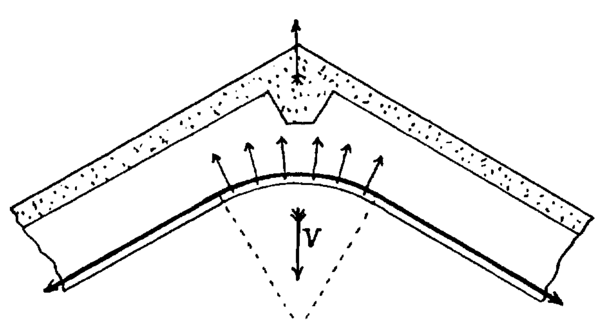


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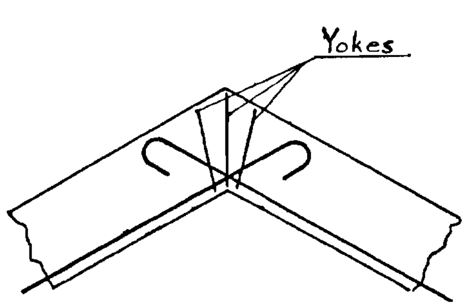


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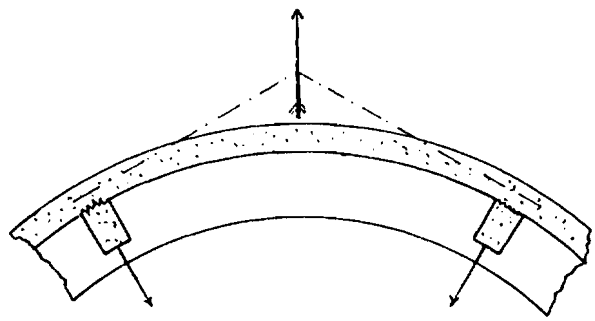

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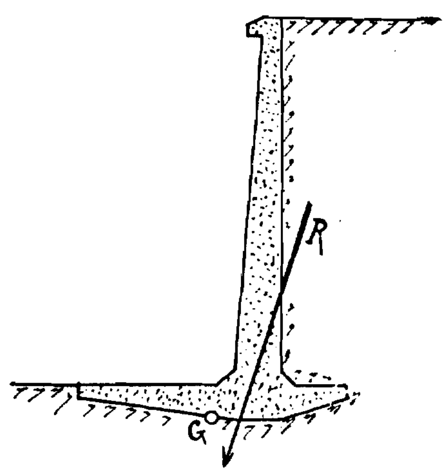
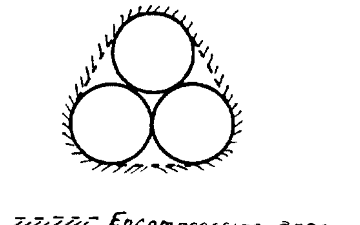



Fig. 30

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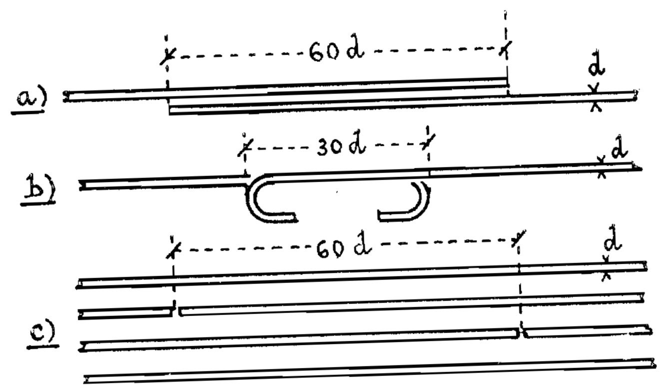


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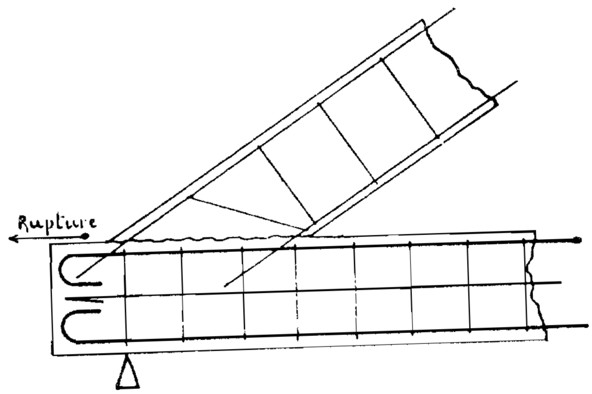


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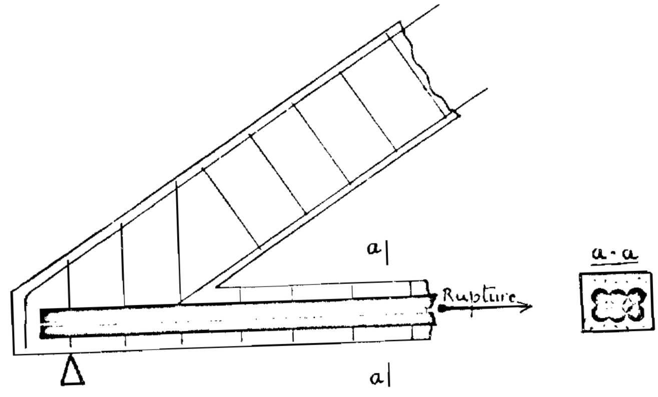


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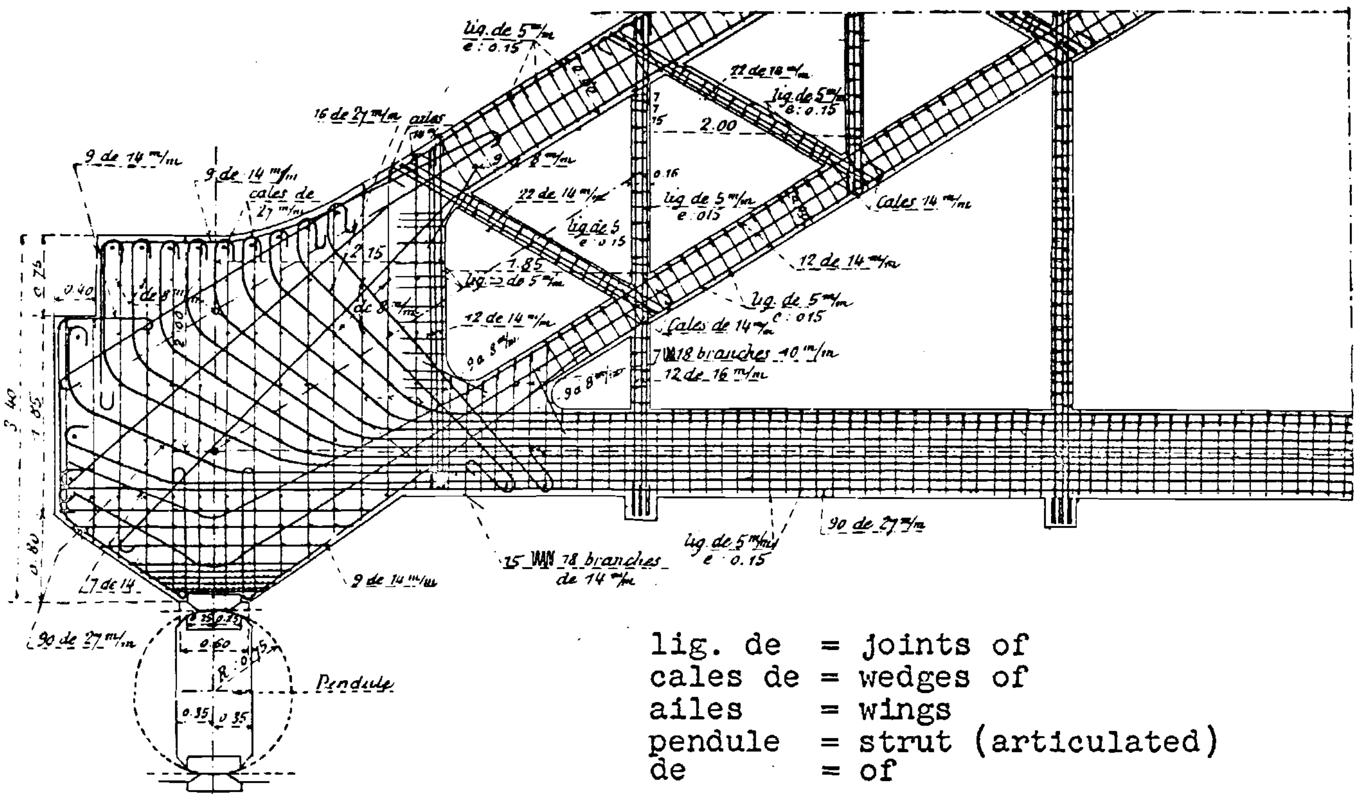
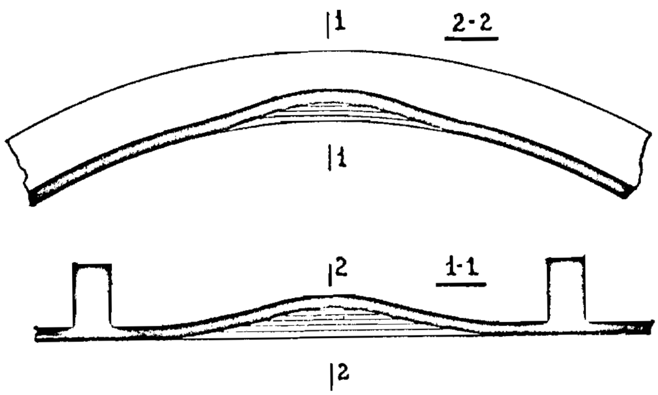



lig. de = joints of
cales de = wedges of
ailes = wings
pendule = strut (articulated)
de = of

Fig. 35

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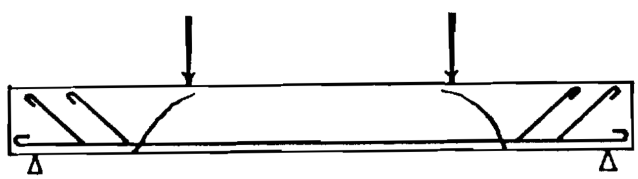

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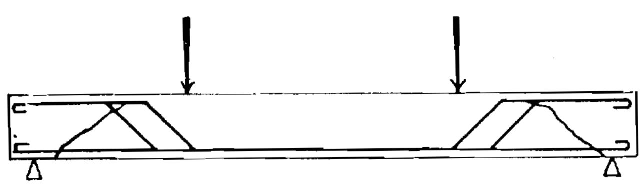

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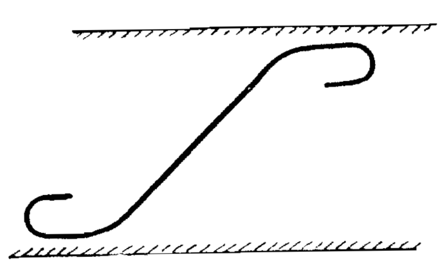


Fig. 39

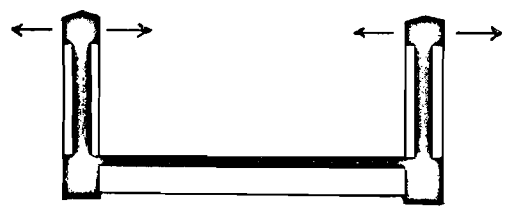


Fig. 40

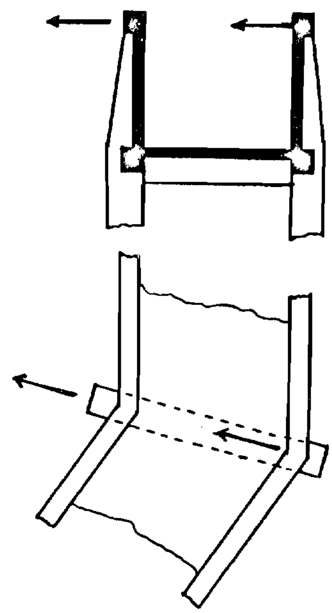


Fig. 41

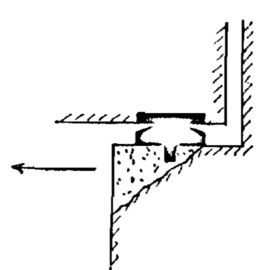


Fig. 42

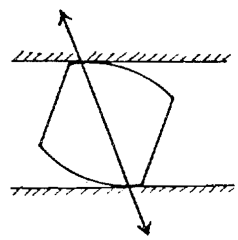
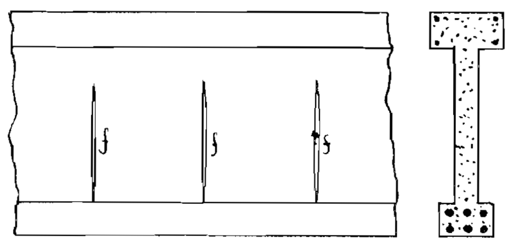


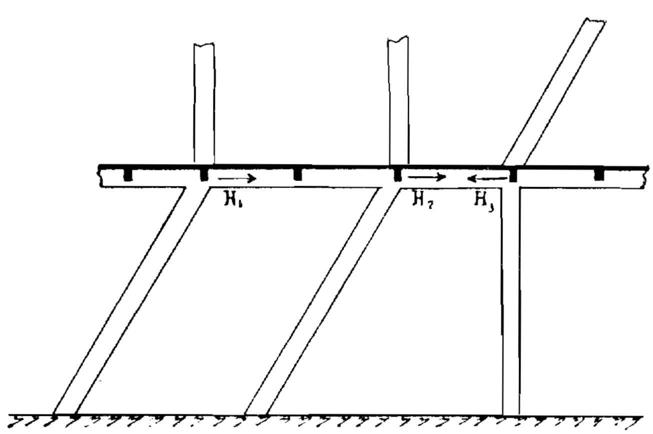
Fig. 43




f: Cracks

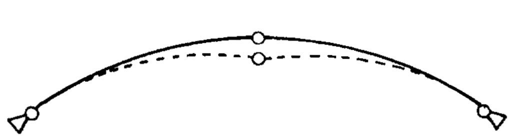
Fig. 44

Fig. 45




$H_1$-$H_2$-$H_3$ : *Horizontal thrusts*

Fig. 46



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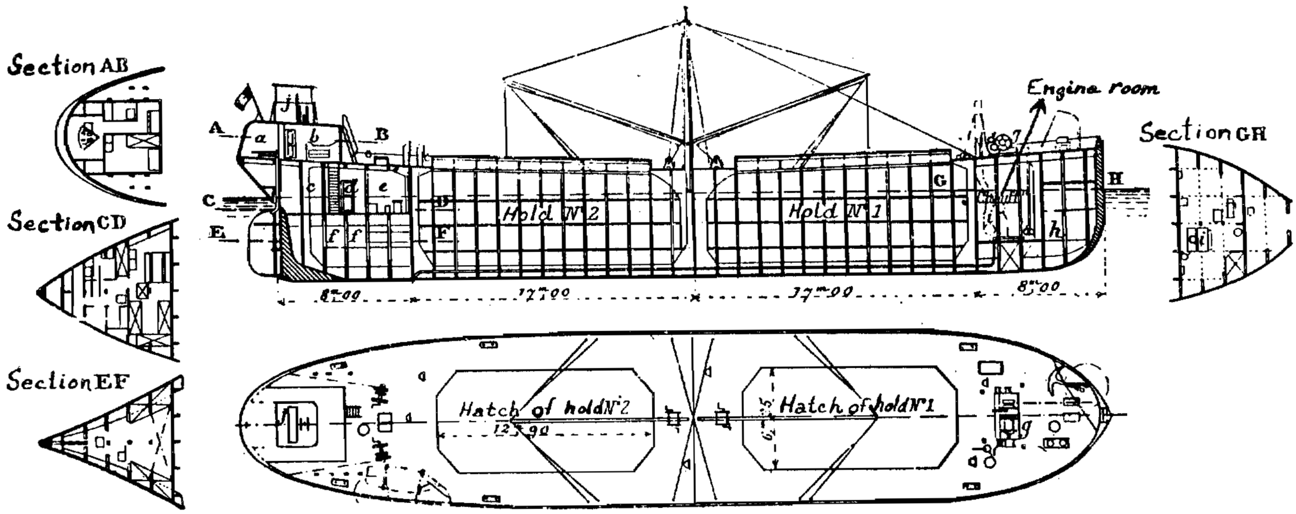
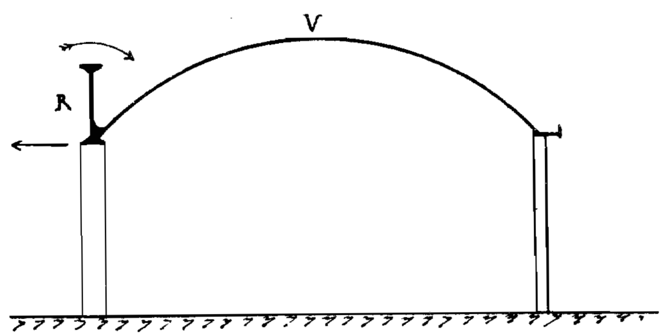



Fig. 48

Henry Lossier marine barge

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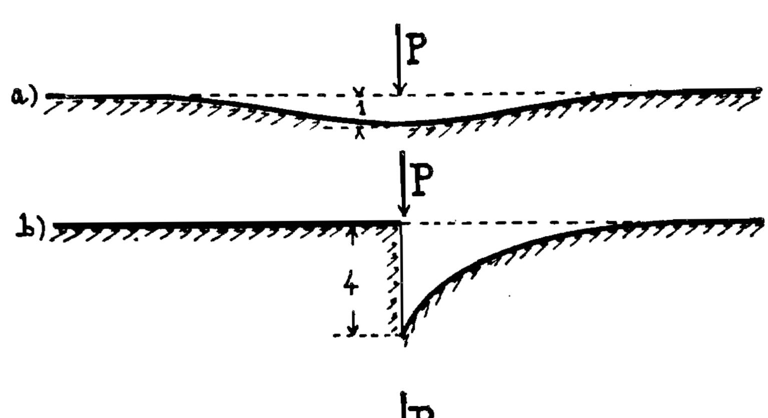



Fig. 50

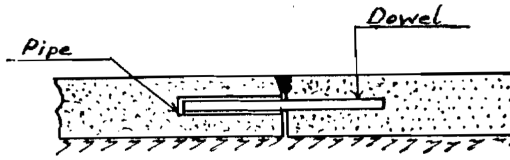



Fig. 51

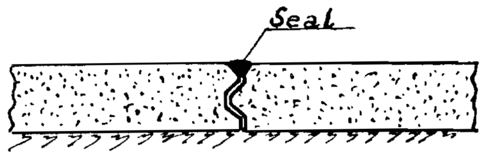



Fig. 52

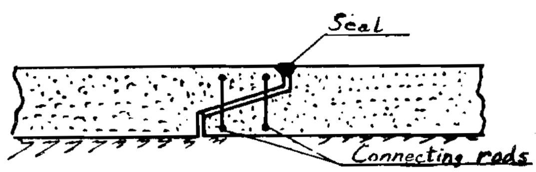



Fig. 53
(Henry Lossier type)

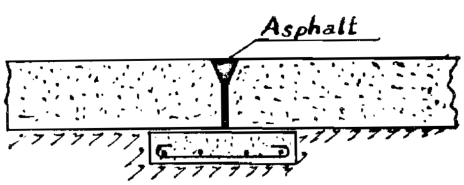
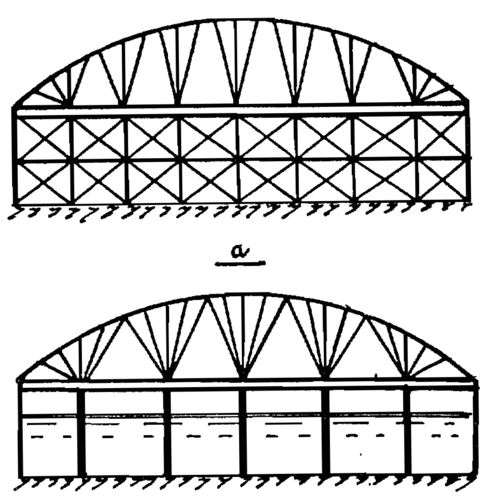



Fig. 54



a

b

Fig. 55

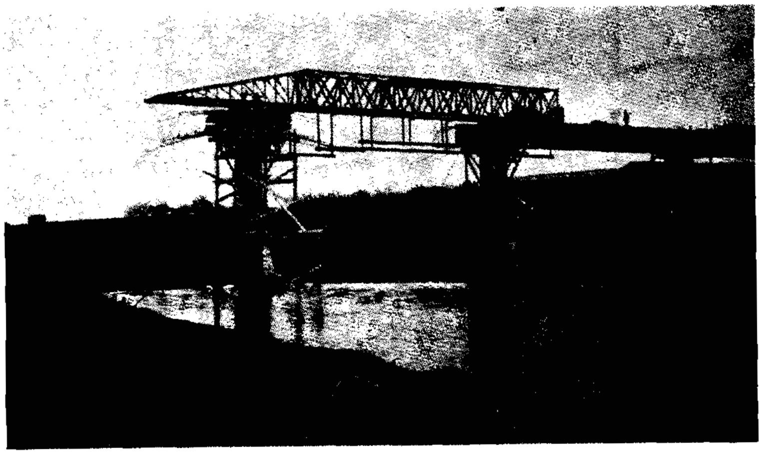


Fig. 56

Bridge over the Sébou R. at Sidi-Abdel-Azia (Marocco).
Henry Lossier method

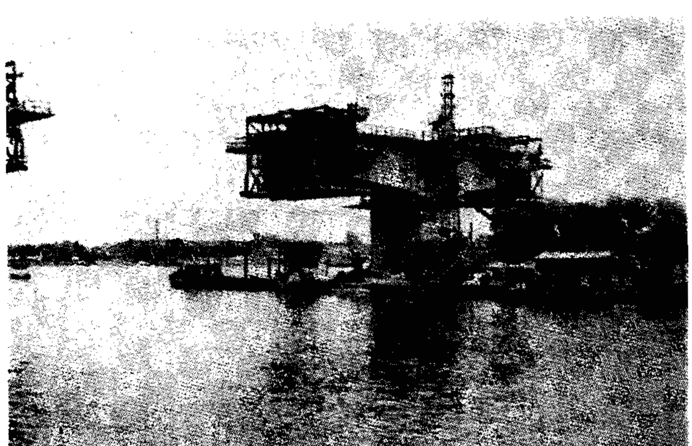


Fig. 57

Execution of the Worms Bridge
on the Rhine

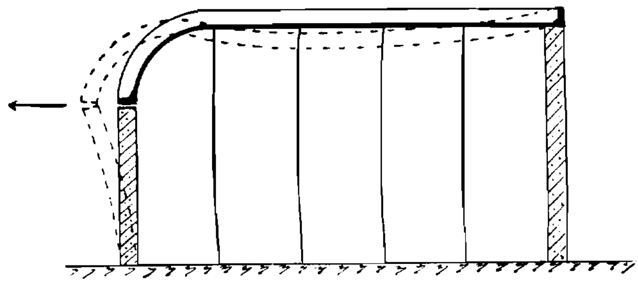


Fig. 58

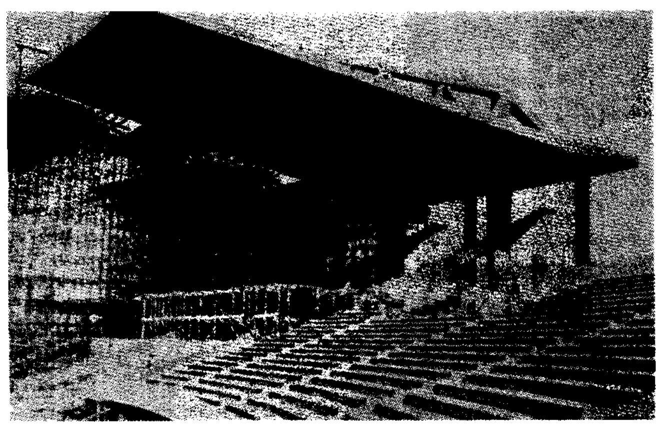


Fig. 59

Execution of the 34 m overhang roof over the Stade d'honneur in Casablanca

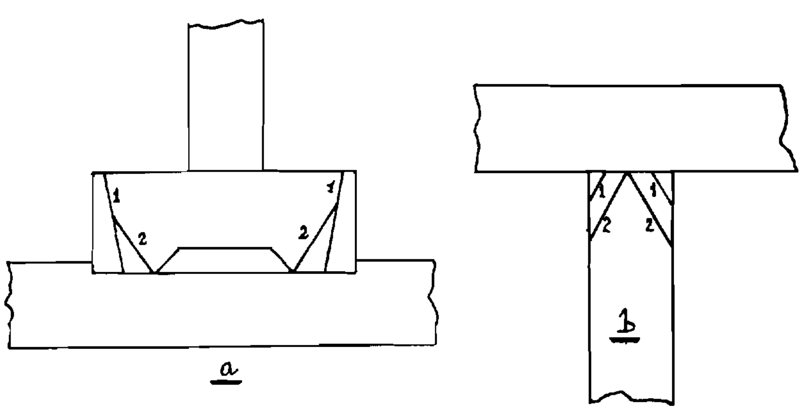



Fig. 60

"Saw cut" method
(a) base; (b) crown

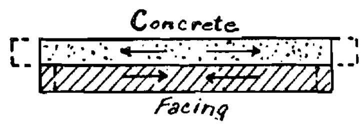



Fig. 61

Fig. 62

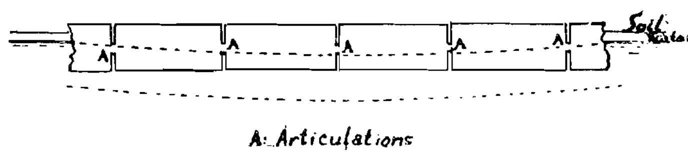



Fig. 63

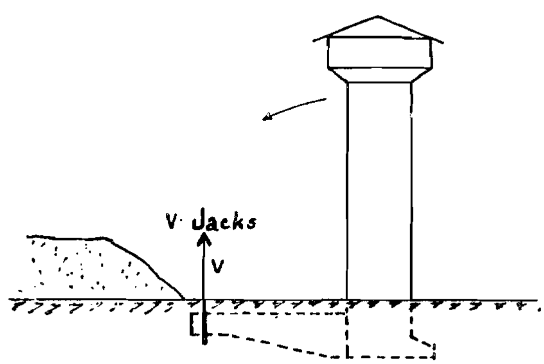



Fig. 64

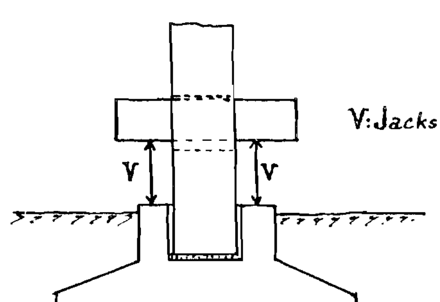



Fig. 65

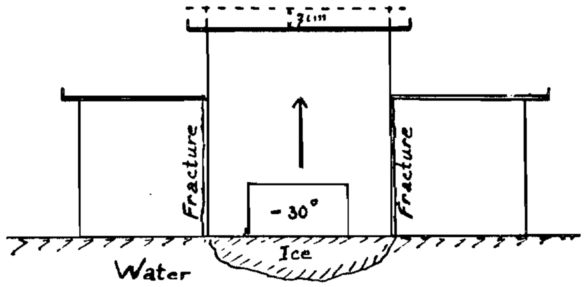



Fig. 66

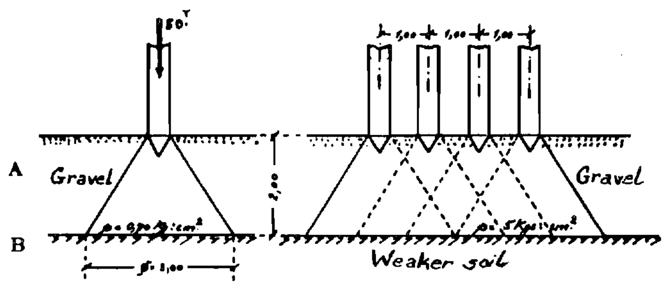
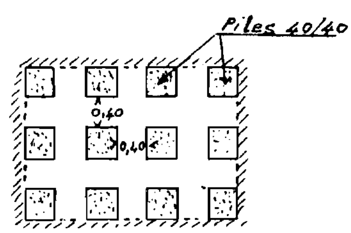



Fig. 67-68



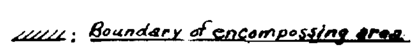




Fig. 69

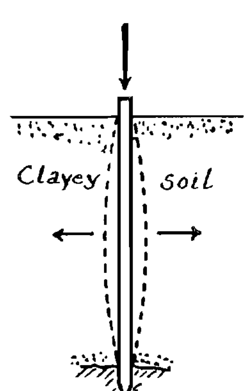



Fig. 70

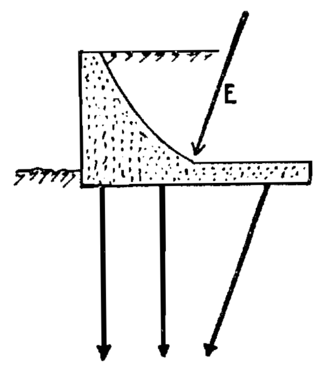



Fig. 71

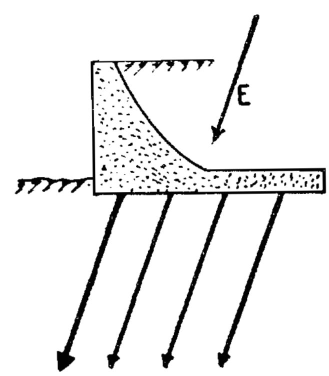



Fig. 72

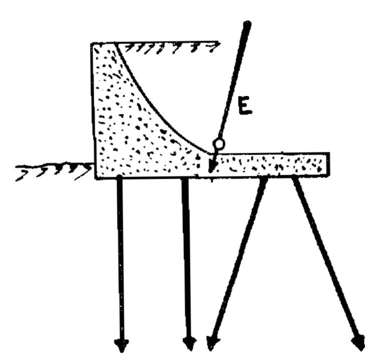



Fig. 73

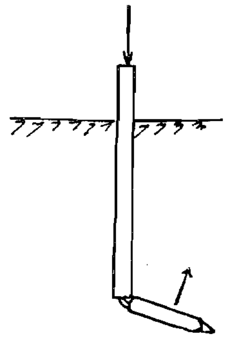


Fig. 74

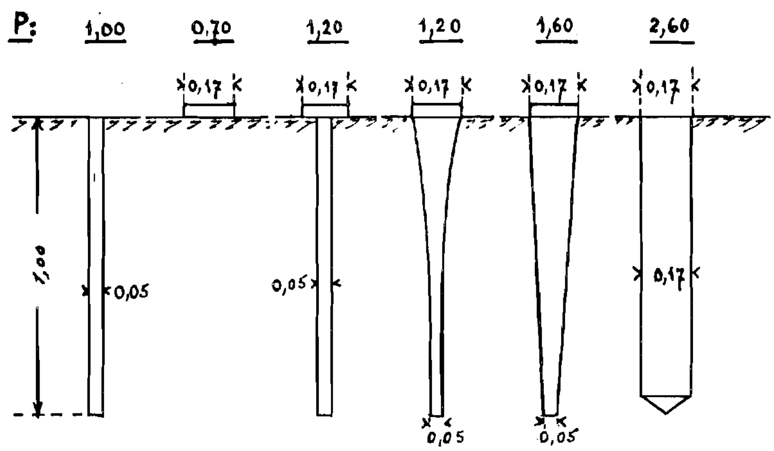



Fig. 75

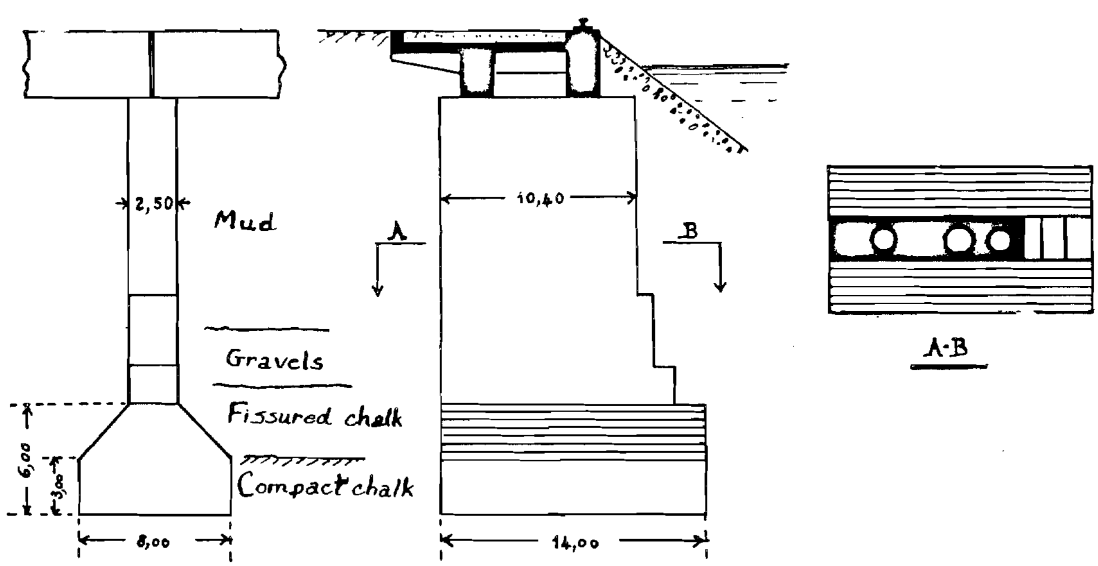



Fig. 76

Grand-Quévilly Pier

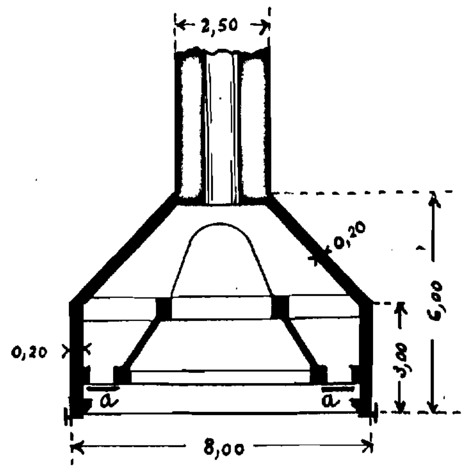



Fig. 77

Grand-Quevilly Pier.
Detail of caisson work compartment

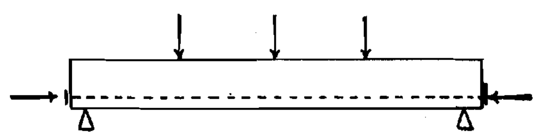


Fig. 78

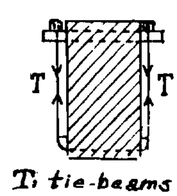



Fig. 79

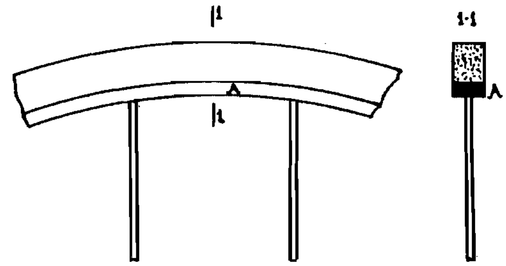



Fig. 80

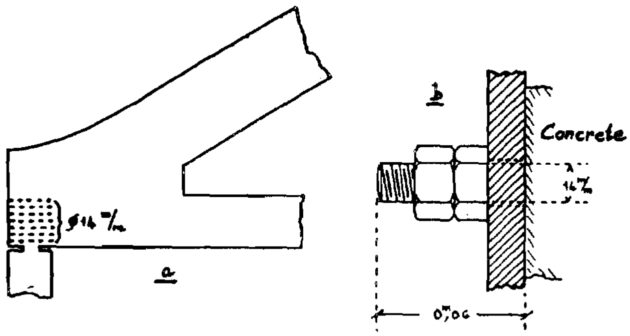



Fig. 81

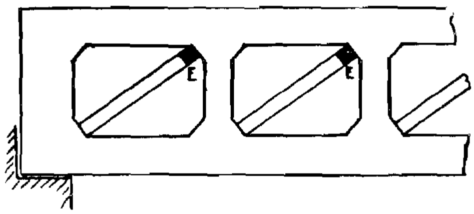



Fig. 82

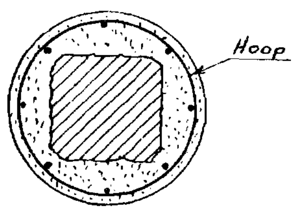
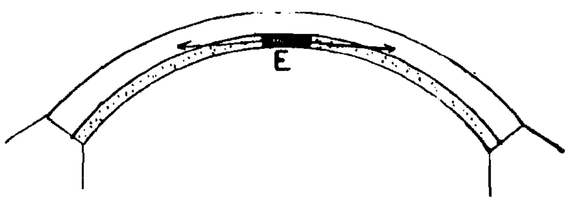



Fig. 83




E : Expansive keystone

Fig. 84

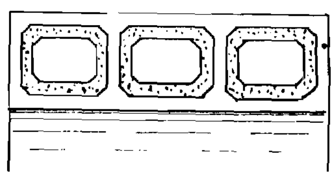


Fig. 85

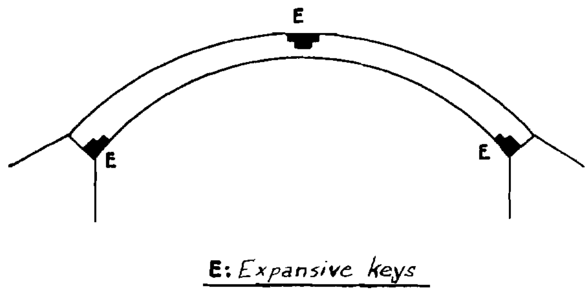



Fig. 86

Fig. 87 (a)

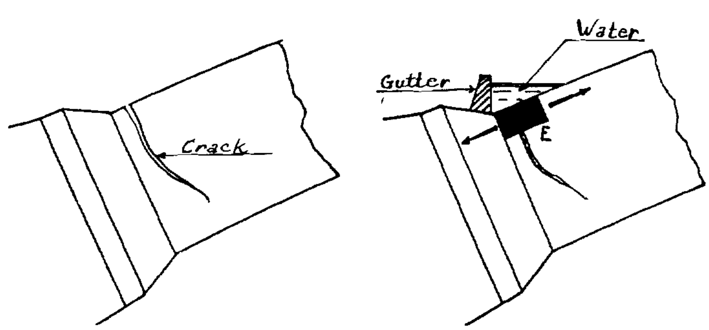



Fig. 87 (b)

Fig. 88

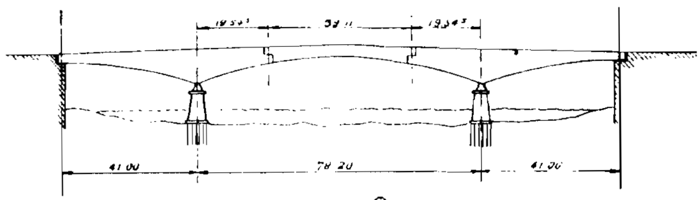
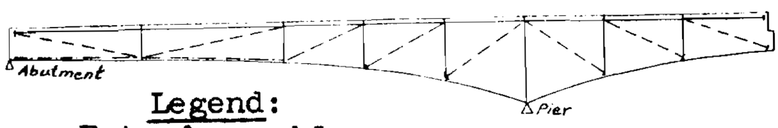



Fig. 89




Legend:
——Extrados cables
--- Inclined cables
—.— Intrados cables
←--- Anchorages

Fig. 90

Fig. 91

Fig. 92

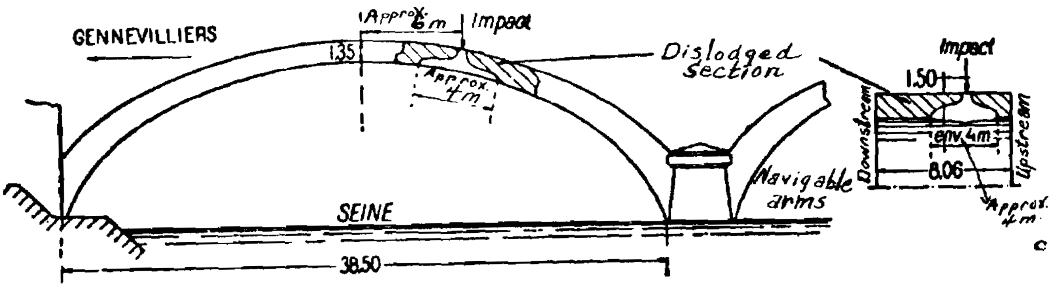



Fig. 93

Fig. 94

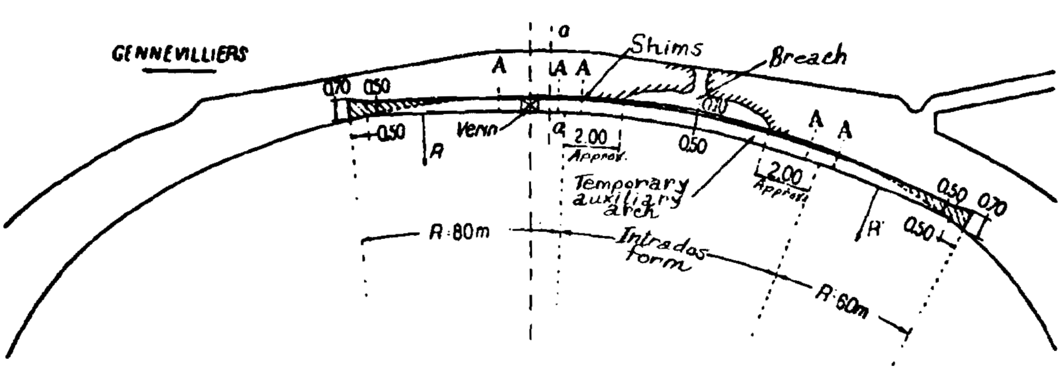



Fig. 95

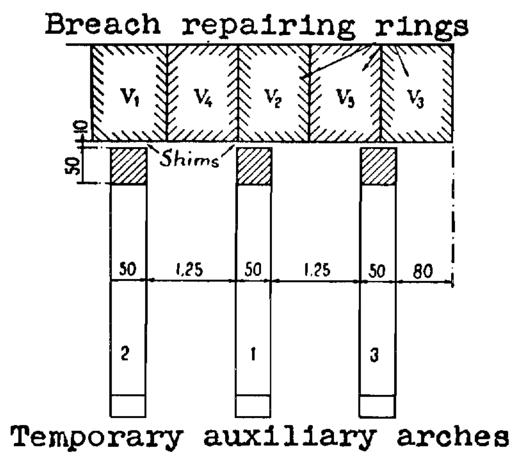

Breach repairing rings
V1
V4
V2
V5
V3
10
50
Shims
50
1.25
50
1.25
50
80
2
1
3
Temporary auxiliary arches

Fig. 96

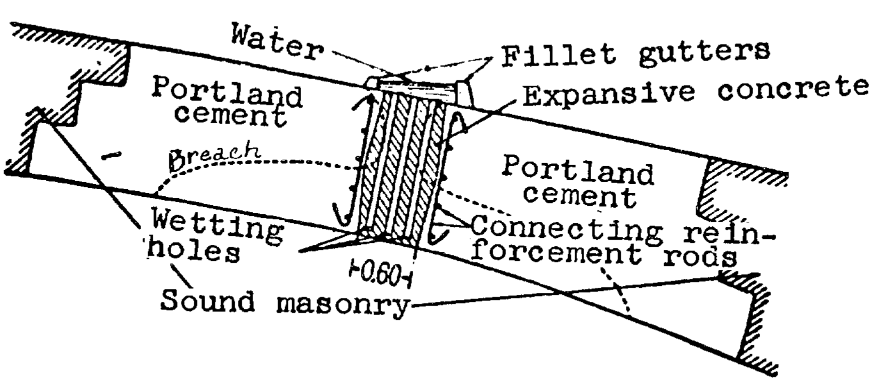

Water
Fillet gutters
Expansive concrete
Portland cement
Breach
Portland cement
Wetting holes
Connecting reinforcement rods
0.60
Sound masonry

Fig. 97

Fig. 98

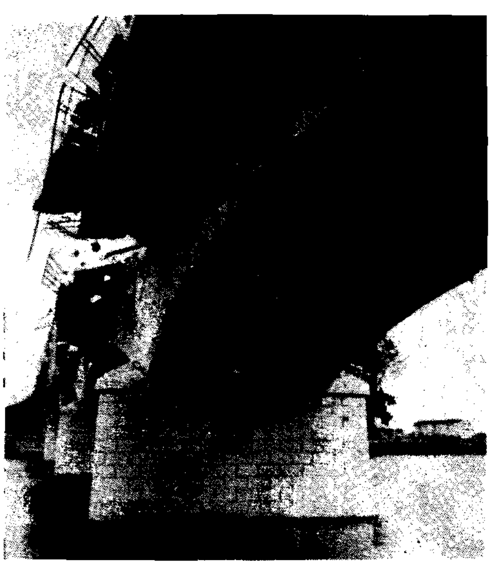


Fig. 99

Fig. 100

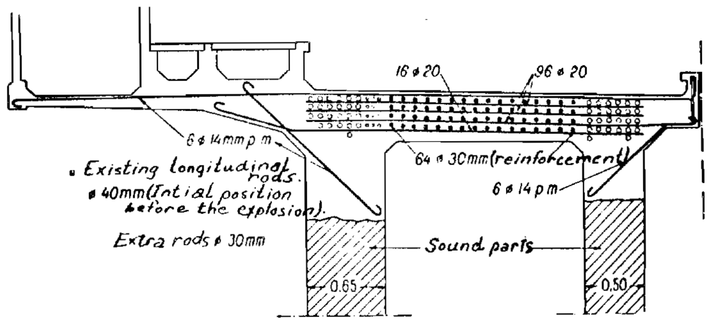



Fig. 101

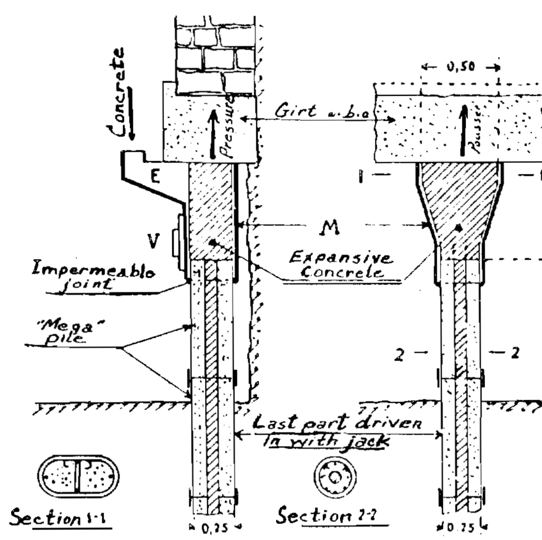



Fig. 102

Fig. 103

Fig. 104

Fig. 105

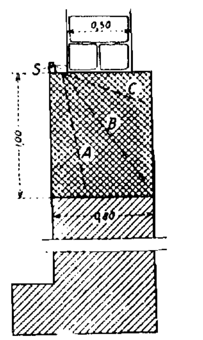



Fig. 106

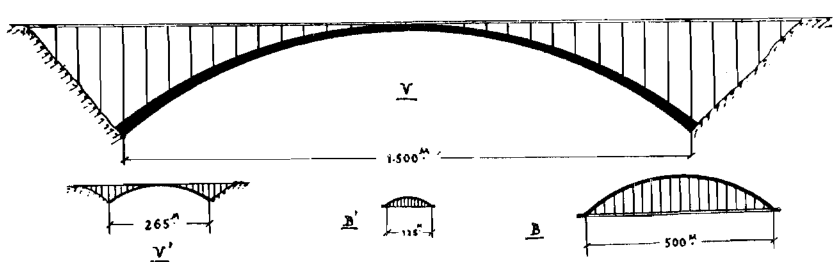



Fig. 107

V - Present-day possible arch span
V' - Actual record to date
B - Present-day possible bow-string span
B' - Actual record to date

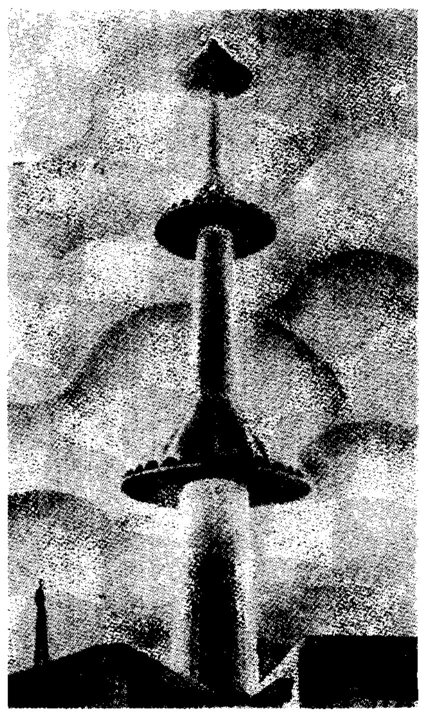


Fig. 108

Model of planned 2000 m high tower

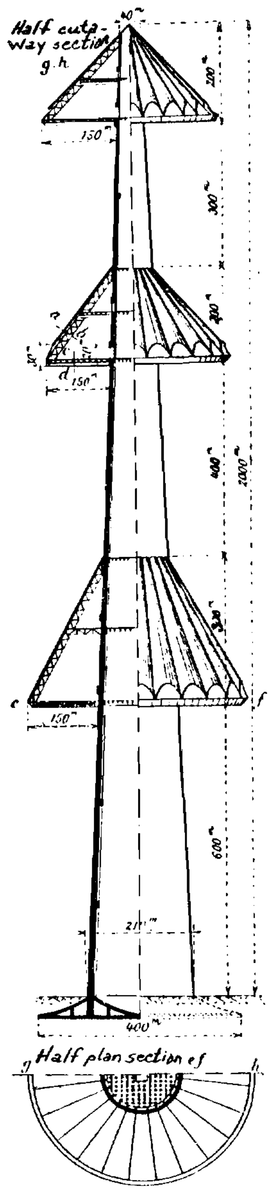



Fig. 109

Half cutaway section and elevation of tower