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VI

Concrete and reinforced concrete in hydraulic engineering
(Dams, pipe lines, pressure galleries etc.)

Beton und Eisenbeton im Wasserbau
(Staumauern, Rohrleitungen, Druckstollen usw.).

Application du béton et du béton armé aux travaux hydrauliques
(Barrages, conduites, galeries sous pression, etc.).

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VI

General Report.

Generalreferat

Rapport Général.

F. Campus,

Professeur à l'Université de Liége, Directeur du Laboratoire d'essais du Génie Civil.

The application of concrete and reinforced concrete to hydraulic engineering covers a field so large as to be incapable of complete or even cursory exploration within the limits of a general report. It appears to me, therefore, best not to attempt this, but to take advantage of the programme laid down for the Congress in confining myself to a brief summary of the various reports which have appeared in the Preliminary Publication. Incidentally I shall allow myself to express certain personal opinion in the form of contributions to the discussion, but it is a satisfaction to me to be able to record that generally speaking I find myself in sincere agreement with the eminent authors of the papers. I cannot but attribute this circumstance to the private connections which I have had the honour of maintaining with the majority of these gentlemen ever since our first acquaintance at earlier Congresses. A close reading of the six papers presented has, indeed, served to strengthen me in the opinion, formed at previous Congresses, that the engineering of different countries — at any rate so far as the continent of Europe is concerned — forms a unified whole. This, I like to think, is an outcome of the growth in number of international technical congresses — an outcome which in itself is enough to confound the criticisms sometimes voiced on the subject of these — and it goes to support the view that the meetings ought periodically to be repeated even should they become less arresting in character. I have felt in my perusal of the papers about to be summarised that any lack of the sensational is the result merely of the modesty of the authors concerned and is counterbalanced — to great advantage — by the masterly, fundamental almost perfect, depth of treatment and the interest that derives from these qualities.

Among the hydraulic engineering works to which reinforced concrete is applied it is natural that dams should occupy the most prominent place, having regard to their size, the difficulties attending their construction and the responsibilities involved. Professor *Ludin,* in his paper on the application of concrete to the construction of large dams in Germany, has given us a piece of work characterised by that merit which those who know his treatise „Die Wasserkräfte" — to say nothing of his other writings — have learned to take for granted. Since 1922 ten gravity dams in concrete have been built or begun, three of these exceeding 60 m in height. Only one multiple arch dam, of medium height, has been constructed in reinforced concrete. Several earth and stone dams facing have been provided with internal watertight screens of concrete, including the Sorpe dam which is the highest European structure of this type (62 m depth of retained water). The description of these gravity dams, the details of their execution, the observations made in the course of execution, and the remarks of the author of the paper on the subject of their development, all go to confirm the general tendencies of European practice. These may be summarised as follows:

a) The supersession of liquid concrete by wet concrete which is semi-fluid and very plastic, but not rammed.

b) The tendency in grading to increase the maximum size of stone aggregate and to use a smaller quantity of sand and in general a non-continuous grading of aggregates.

c) An increase in the cement content, the addition of hydraulic materials such as trass and ground furnace slag, or the use of special cements with a view to increased resistance to the aggressive action of water or of atmospheric agencies (on exposed surfaces); also a greater degree of compactness, a smaller development of heat and a smaller amount of shrinkage, etc.

d) The abandonment of stone facings or, indeed, of any kind of linings. But according to the author of this report, German practice has not yet reached a definite decision whether to use a homogeneous concrete or to form facings of richer and more carefully made concrete. The solution to this dilemma may, perhaps, be discerned in the general tendency, in most countries, to give up using excessively weak mixtures (see the paper by M. *Coyne*) but to pay special attention to the treatment of the facings (as by the adoption of vibration).

e) The provision of shrinkage joints, and of drainage both on the upstream face and in the foundation.

f) Mechanisation and modern organisation of plant and installation on site with a view to increasing the rate of progress, even to the extent of cooling thse concrete while poured as done already in America.

Attention may be drawn to the following points of detail:

a) The desirability of taking uplift into consideration, as long proved in Germany by measurements of pressures in the foundations of structures, measurements of percolation, etc.

b) The occurrence of shrinkage cracks in the lower portions of the dams at Agger, Bleiloch and Schluchsee, where the shrinkage joints although relatively close together did not reach down to the base of the work. These structures are slightly curved in plan, as is true of most German dams except that at Zillierbach. Here confirmation may be found for the opinion (which I have already maintained) that the slight curvature frequently conferred upon gravity dams does not, as regards their thicker portions, possess all those virtues which are apt to be claimed for it without sufficient reason, and that it offers no justification for relaxing the precautions required in this type of structure. On the other hand the fact that the crack in the Agger dam extended only half way up from the base[1] shows that the curvature may be effective in the thinner portions of the structure.

c) The use of stone dust which has the effect of weakening the cement is recognised as being undesirable, but on the other hand the addition of pozzolanic material such as trass, or hydraulic materials like ground slags, may be advantageous. An admixture of this material has been adopted in the most recent of the dams under construction, that at Hohenwarte in Thuringia. At the International Congress on the Testing of Materials in 1931, I took the occasion to point out the advantages of these admixtures, having myself made use of them in foundations exposed to aggressive waters; since then, however, I have abandoned this system in view of the production of metallurgical cement, which is in fact a ready made mixture of Portland and slag. In Germany, moreover, the use of special cements (such as trass and metallurgical cement) appears to be increasing, and there is a general tendency towards increased fineness in grinding which has the result of accelerating the rate of setting of these cements, and of increasing their workability and the watertightness of the concrete.

d) The mixtures employed in the most recent dams (such as that at Sorpe) are expressed by weight, and no longer, as has been the customary practice in Germany, by volume. The present writer favours specification by weight in order to ensure regularity in the quality of the concrete, and he presumes that the change in Germany has been made for the same reason.

M. *Coyne*, Ingénieur en Chef des Ponts et Chaussées, of Paris, is in charge of the work on the dam at Marèges in France, which is the largest European arch dam (90 m high and 247 m length of the crest); he modestly entitles his report "Observations on the use of concrete in solid dams". Actually his paper forms a general report such as the present writer would have liked to make, and the difficulty of summarising it will be obvious. It is a remarkably lucid exposition by an engineer whose mastery stands out against a background of great attainments. Confining himself to the question of concrete as a material for making dams, M. *Coyne* expounds its use as would a master sculptor his clay. He explains, discusses and formulates rules; and according to the preceding paper some of them appear to embody the same lessons as emerge from German experience. They include the adoption of a concrete which is plastic but not fluid; recognition of the importance of workability, to ensure the faultless placing of the concrete in the job; the use of mixtures sufficiently rich to ensure durability; the adoption of special cements with low evolution of heat and not susceptible to attack by aggressive waters; the employment of vibration — especially on the upstream face — as the finishing touch to the use of good plastic concrete; the use of correct grading of aggregates and in certain cases the adoption of artificial cooling these are the principal measures which M. *Coyne* recommends. Proportioning by weight, also, is favoured by him. In my own humble opinion I can only emphasise the danger inherent in discontinuities in the work, and the care necessary to ensure adequate bond of working joints.

[1] Preliminary Publication, Fig. 8, p. 1198.

At the International Congress on Testing Materials at Zurich in 1931 I have already supported his advise as to the exercise of control on the job by taking specimens from the work itself; likewise as to control over the density of fresh concrete and over the strength of mortar taken from the concrete (according to Bolomey).

Importance attaches also to his penetrating remarks, which are the fruit of well digested experience, on the danger of longitudinal cracks; on the mechanism of watertightness and stanching (especially that due to biological causes), on the deterioration of concrete; and on the subject of the considerable resistance offered against erosion when the concrete is dense and smooth. M. *Coyne* is a strong partisan of arch dams, and while not concerning himself in the present work with the question of design of dams he points out incidentally a defect of gravity dams: namely the deterioration of the concrete from climatic causes, which is the result of the low cement content in the mixtures usually adopted for plain concrete dams.

Prof. *Bažant*, of the Czechoslovak Polytechnic School of Prague, has contributed a masterly exposition under the title of "The Development of Design for Arch Dams". Considerations regarding the application of concrete, while not paramount in this particular study, are not absent therefrom, for when deciding upon the design of arch dams it is necessary to take account of the conditions of construction and of the mechanical and physical properties of the material such as shrinkage, low resistance to tension, the need for local reinforcement, contraction joints, etc. The author of this paper shows that modern arch dams originated in Europe, and justifies their points of superiority over gravity dams in a way which the author of the previous paper, M. *Coyne*, would be unable to contradict; he then proceeds to analyse the assumptions underlying the methods of design which have followed one another in the following sequence:

a) The arch is considered to be built up of independent arched elements each of which separately resists the hydrostatic pressure.

b) The foregoing action is supplemented by a resisting action due to the weight of the vertical elements of the dam, considered to be built in at the base.

c) The barrel arch is assumed to form a curved elastic shell.

This last concept, while theoretically the most accurate, has scarcely passed beyond the stage of infinitesimal equations, and in the opinion of the writer of the paper is not practicable. The two earlier ideas may take very varied forms from the simplest (in case *a*, the theory of a thin cylindrical shell) to the most complicated (in case *b*, the "trial load method" of the Americans). In theory, all these methods are inaccurate, or rather are to be described as approximate. The writer of the paper confines himself, very properly, to a clear and detailed explanation of the details of this development. Here M. *Coyne* will allow me to refer to his opinion as to these theories and, at the same time, as to a method worked out by him which is different again (though related to method *a* above) in which the resisting arch elements are not independent of another but are bound by isostatic surfaces, generally having the character of inclined arches, and offering resistance to the hydrostatic pressure both as arches and as buttresses.

This method, which I believe to be unpublished — and for that reason, I suppose, unknown to M. *Bažant* — has been applied as a check for the large dam at Marèges. I hope that my indiscretion in referring to it here will induce M. *Coyne* to report on this matter in due course.

It is to one of M. *Coyne*'s principal colleagues, M. *Mary*, Ingénieur des Ponts et Chaussées, of Paris, that we owe a paper of very great interest both in itself and for its documentary value, entitled "The Hoop Reinforcement of Pressure Conduits for the Hydro-Electric Plant at Marèges". In this he has described one of the most remarkable items of the work at Marèges, in which concrete has been applied with marvellous ingenuity. The essential requirement was that of constructing pressure conduits in reinforced concrete with an internal diameter of 4.40 m to withstand an internal hydraulic pressure equivalent to a head of 102.50 m of water (10.25 atmospheres), without the thickness exceeding 0.40 m, while guaranteeing complete safety and, of course, complete watertightness. The solution adopted was to surround the concrete pipes by cables forming a circumferential reinforcement which was placed in tension beforehand. The idea of pre-stressing was not in itself original, but its application to a pipe-line constructed underground was so. Its boldness called for preliminary experiments, and the effect of these was both to indicate the aptness of the solution and to suggest how it could best be put into practice in the actual job. The cables, spaced longitudinally at intervals of 0.50 m, were given a pre-imposed tension of 135 tonnes after the envelope had been concreted, this being done by increasing the diameter of the ring of cable through the agency of two opposed jacks which were affixed to the concrete wall of the pipe. The cables when deformed in this way were secured in a mortar, made by filling ciment fondu into gaps left for the purpose. Apart from this the pipes are provided only with some local reinforcement.[1] Measurements of internal stresses, carried out with the acoustic string measuring instruments of M. *Coyne*, have served to confirm the success of this work. All the various phases of its design and execution, together with the results obtained, are explained in the paper with that meticulous care which reflects the personality of the author and which entitles him to the reader's gratitude.

Leaving the subject of dams and works connected therewith, we now pass on to those other prodigies of engineering, rendered equally formidable to the engineer by difficulties of quite another order, due to construction having to be carried out under water at great depth. These are represented in the paper by Professor *G. Krall* and Mr. *H. Straub* on "New Dry Docks in the Ports of Genoa and Naples".

Italy is a country renowned not only for large dams, but as one where nature has forced the attainment of mastery in maritime engineering. The works described and explained by the authors of the paper are remarkable modern examples of this, reflecting credit on them as much for the science displayed, the accuracy of the investigation, and the assurance with which the work has been carried into effect. As regards width the two dry docks are identical; 40 m in width, 14 m depth of water below mean water level, 9 m thickness of side walls,

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[2] Preliminary Report, Fig. 10, p. 1218.

280 m length at Genoa and approximately 350 m length at Naples. But while the appearance is the same the design in the two cases is essentially different, in view of the rocky nature of the ground at Genoa and the soft ground at Naples. The problems to be solved in the two respective cases were not, therefore, questions of the use of concrete but questions arising from the nature of the ground — for here as in the case of the dams discussed by M. *Coyne* that is the governing condition. The use of concrete confers flexibility on the ideas of the engineer, and this, in combination with highly developed constructional technique, has allowed of a perfect solution being obtained.

The paper by Messrs. *Krall* and *Straub* provides a striking and up-to-date demonstration of the advantage — indeed necessity — of close correlation between design and construction. In the two dry docks under consideration the stresses were at least as high during the various stages of construction as in normal operation afterwards. It is the acceptance of this principle which constitutes one of the greatest advances ever made in constructional work, and the examples described by the authors are an admirable instance of its application. The principle is explained consisely but very clearly in the paper, but does not lend itself very well to summarising without going to excessive length. In the case of the Genoa dry dock, founded under compressed air on rocky ground, the main hyperstatic problem arising in this design was solved by an ingenious mode of construction. The side walls were built first with the aid of reinforced concrete caissons sunk by compressed air, and the end wall was constructed in the same way. The enclosure thus formed was closed at the open end by a caisson gate and was pumped dry to allow of concreting the bottom. During this phase of construction the enclosure had to withstand a water pressure equal to nearly 20 m a greater head than would be imposed on it later in actual service, and in order to provide for this without the use of heavy permanent reinforcement a temporary arrangement was adopted in which the side walls were caused to act as multiple arch dams between piers strutted against one another across the width of the dock. This gave rise to a statical problem analogous with that arising in arch dams, but with certain additional complications, which were solved in a neat and original way by Professor *Krall*.[3] It is to be noted that the deformations of the ground were taken into account the same way as the deformations of all structural parts. The procedure of the calculation is clearly explained. After the construction of the floor the temporary counterforts were demolished.

In the case of the dry dock at Naples, which was founded on soft ground, the problem was different and had reference to the structure in its final form. The walls and the floor were constructed separately by means of caissons, the latter being handled from travelling gantries of 68 m span running on two reinforced concrete bridges built parallel to the side walls of the dock. After the walls and the floor had undergone settlement independently, the joints between them were closed up with the aid of a diving bell. The static problem to be solved related to the stresses arising in the structure after the joint had been closed, taking due account of the deformability of the ground below: this problem is one of

[3] Preliminary Report, Figs. 5 and 6, pp. 1172—1173.

long standing which admits of solution by ordinary methods based on the strength of materials, as applied, for instance, in a recent investigation by one of my pupils (*F. Szeps:* Etude des constructions reposant sur un sol élastique. Revue Universelle des Mines, March 1936).

Professor *Krall* explains the principle of a very interesting method based on the use of ellipses of elasticity, due to *W. Ritter*. The confident application of this method to the dry dock at Naples was rendered possible through observations carried out on the dry dock at Venice under usual conditions of service, with a view to determining the "coefficient of soil elasticity" or of deformability of the ground. The instances in which an experiment has been carried out on so large a scale with so excellent a result must be rare.

The execution of the work was on a par with the design explained above. Points of interest include the use of a plastic concrete containing 300 kg of pozzolanic cement per cubic metre with a view to resisting the aggressive action of the sea water; the general adoption of concrete for the compressed air caissons reaching to depths of more than 23 m; the construction of what was in effect an auxiliary dry dock of reinforced concrete for the purpose of constructing the caissons at Genoa; and the reinforced concrete service bridges used ab Naples. In short, the paper by Messrs. *Krall* and *Straub* affords a striking proof of the utility of concrete and reinforced concrete for solving problems in hydraulic and maritime work; but emphasis should also be laid on the lesson it affords of the potency, to engineers, of a combination of the science of structural design with experiment, and with experience gained on actual works.

I have kept for last, but not least, the paper by Dr. *W. H. Glanville* and Mr. *G. Grime* dealing with the "Behaviour of Reinforced Concrete Piles in Driving", on account of its very special character which is less directly relevant to hydraulic work in the sense that, although reinforced concrete piles are very frequent and very valuable adjuncts to hydraulic construction, the question as treated by the authors is not directly related to works of this kind. The paper is none the less of very special interest both intrinsically and on account of the practical conclusions reached through the masterly scientific way in which the question is handled — a mastery which is characteristic of Dr. *Glanville,* well known for his researches on "creep strains" in concrete, and equally characteristic of the Building Research Station to whose higher staff he belongs.

The paper forms a very detailed summary of the investigations carried out for several years past at the Building Research Station dealing with the behaviour of reinforced concrete piles under the effects of driving. It differs only slightly from a more complete paper to which the authors refer, which appeared in the Journal of the Institution of Civil Engineers in December 1935, the only difference being the omission of a mathematical treatment of the question of the propagation of impact waves in the piles. Partial reports have already appeared in British journals and in the annual reports of the Building Research Station, and an official report on the investigation is to be published. The investigation was carried out theoretically on the basis of certain definite assumptions which agree well enough with practice, and later experimentally, first of all at the Research Station and then on actual works. Special mention should be made of the employment of a dynamic stress measuring instrument of extreme sensitivity which made use of piezo-electric quartz and oscillographic recording instrument in conjunction with a special form of accelerometer for controlling the maximum stresses at the head of the pile during the process of driving. Practical engineers are well aware how, in certain circumstances, the driving of reinforced concrete piles may give rise to difficulties, and I have myself given an account of such difficulties recently in Annales des Travaux Publics de Belgique, February 1935. The research undertaken by Dr. *Glanville* and Mr. *Grime* amounts to little less than a revelation as regards the magnitude of the mechanical phenomena produced in piles under the action of the impact of the pile driving hammer and as regards the resulting instantaneous stresses. The theoretical and related experimental steps disclose a whole range of details on which no exact information, it appears, has hitherto been available. Practically, an old rule is confirmed: heavy hammers are the best and it is advisable not to exceed a certain height of fall. Another rule, of no less importance, would appear to be new: if the maximum instantaneous stress in the pile is to be limited with a view to the preservation of the latter while driving, and it is desired to ensure maximum penetration, the head of the pile should be covered with a cushion as elastic as possible without reducing the efficiency of the energy transfer so as to be inconsistent with the conditions under which the pile driving is being done.

The essential practical outcome, therefore, is that which relates to the cushioning of the pile heads. The scientific question of how the dynamic stresses vary in their distribution may be regarded as solved; most often these stresses are at a maximum in the head and are independent of the nature of the ground, but sometimes, where the driving is very hard, they are at their maximum at the point. Rules are laid down for the composition of the concrete and the arrangement of reinforcement. These rules are in agreement with good practice, but it is certain that many contractors would benefit by considering the sound advice here given in reference to precautions in driving. In any case the strength of the concrete used in piles should not be less than 350 to 500 kg/cm^2 at the time of driving. Diagrams (dimensioned in British units) accompany the paper for the purpose of rapidly determining the optimum conditions for driving the piles in any given case, but in their present form these do not appear to be applicable to Continental conditions.

To conclude this general report it only remains for me to express the great pleasure which I have felt in reading the remarkable works which it has been my honour to analyse here, not only because of their high quality and great interest, but also because I have felt flattered to find in them a confirmation of the optimistic opinions I have expressed in earlier general reports on the subject of the unceasing progress which is occurring in the application of reinforced concrete; more particularly in that field of engineering construction which is acknowledged to be one of the most difficult of all. Nor has this progress by any means come to an end. In any case the future scope of reinforced concrete leaves no room for doubt. The extent of its applications continually gives rise to new and important problems; in reference to these the old dogmas appear relatively insignificant, and have no effect on development.

VI 1

Concrete in Hydraulic Works.

Beton im Wasserbau.

Le béton dans la construction hydraulique.

Hafenbaudirektor a. D. Dr. Ing. A. Agatz,
Professor an der Technischen Hochschule, Berlin.

a) *Introduction.*

The excellent paper on concrete in German dam construction by Professor *Ludin* offers an occasion to complete the treatment of this branch of work by giving an account of German experience in the use of concrete as applied to the engineering of waterways and in the construction of foundations.

The hydraulic engineer, unlike his colleagues engaged in other kinds of construction, is, unfortunately, unable to convey a proper idea of the magnitude of his works since as much as 75 % of this is surrounded by their natural enemies, earth and water. Their true extent can, therefore, be appreciated only from a statement of the quantities involved.

Since the great height of the bridge piers in some of the latest German bridges has several times been quoted, it may be useful as an example to mention here a reinforced concrete gateway of an ocean dock which has a total height of approximately 26 m. When, further, it is stated that plans are now in hand for reinforced concrete works of a similar nature to this, taking the form of a triple frame which is statically indeterminate to the eleventh degree and has a height of 32 m with a ground area of 56 by 65 m, it will at once be apparent not only that reinforced concrete is highly valued by engineers concerned with foundations and hydraulic works, but that if this method of construction were not available, such works would either be quite impossible or could be carried out only with difficulty — a fact which has been brought out, in the last — mentioned instance, by the author's comparative design for massive masonry construction. Moreover, we no longer live in those easy times when such jobs as this could be carried out at leisure, but are required to complete them in half or even one-third of the time previously considered proper.

The very fact, however, that engineers in this field of work are advocates of reinforced concrete for foundations and hydraulic construction, renders it necessary to observe that even yet no such improvement in the quality of concrete (and especially in its binding material) has been achieved as would justify regarding it as adequate in every possible case; and in this connection it must not be forgotten that not only reinforced concrete, but likewise steel, is subject to risks when exposed to aggressive subsoils and waters. The author, proud as he is of works carried out under his direction both in reinforced concrete and in steel yet feels forced, in the light of the stringent and ever recurring examinations which such structures are subjected, to acknowledge that concrete is a material which remains the plaything of human imperfection as well as of the attacks of earth and water.

b) *Experience and observations.*

The material normally adopted as aggregate is the shingle as found in the river beds, but in view of its lack of uniformity it is found desirable to test its granular composition and proportion of voids and to exercise continuous control over deliveries. Some engineers have sought to improve the quality of the aggregate by first sorting it into its components and then adding fine material or broken stone, but the author has not always deemed these measures to be justified, since in his experience the same strength is obtainable at equal cost (or even at a saving in cost and time) by the use of a richer admixture of cement.

The greatest importance is attached, in the first place, to compressive and tensile strength and to density, but on the other hand sufficient attention is not always paid to the life of the concrete and to its resistance against chemical and physical effects at the surface. In the author's opinion the quest of strength, which has been pursued in concrete works for the last fifteen years, is not of such decisive importance where massive hydraulic works and foundations are concerned.

There must always be a definite distinction between the more tenuous types of design appropriate to reinforced concrete structures above ground and the massive concrete work used below ground and in hydraulic structures. In the first case, adopt limits up to 65 and 1500 kg/cm^2 (for the concrete and steel respectively), and in the second case, for use in hydraulic and foundation works which may undergo later movement or which are exposed to chemical attack, however weak, the corresponding figures may be 30 and 1000 kg/cm^2.

Moreover, it is necessary to be clear that the strength at 28 days provides no definite criterion for the strength in the completed work, where the latter is of large size. For instance, in the construction of a lock the author found that a portion of the concrete made with 270 kg of blast furnace cement and 30 kg of trass per m^3 showed a strength in the job, when 28 days old, of only 80 kg/cm^2, which ought — according to his interpretation of the regulations — to have entailed its removal; and this applied "only" to 12000 m^3 of concrete. The same concrete, however, after 90 days reached 159 kg/cm^2, a figure only 9 % below that obtained for another part built with concrete which after 28 days had already reached a breaking strength of approximately 125 kg/cm^2. The time of year, weather conditions, height and thickness of the blocks concreted, treatment of the concrete within the shuttering, type of shuttering, all exercise an important effect on the 28-day strength.

In massive works below ground or below water the author's practice has been seldom to stress the concrete to more than 30 kg/cm^2, and in his opinion it is unimportant whether the concrete shows a strength of 150 or 180 kg/cm^2 at

90 days, but proportionately more important that it should possess endurance. The factor of safety will then always be at a minimum of 5, whereas for other aspects of foundation or hydraulic work, for instance as regards the stress in the steel used in cofferdams, and as regards the carrying capacity of piles, a maximum factor of safety of 2 is accepted.

The variations in strength obtained when concrete specimens of identical composition are tested go to show that concrete must continue to be looked upon as a very rough kind of material, so that an adequate margin of safety in its use is essential.

Further, there is a difference between the compressive strength of cement at 450 to 550 kg/cm^2 and that of granite at 800 to 2700 kg/cm^2 or of sandstone at 1500 kg/cm^2, which means that we have not yet attained to the possibility of equalising the strengths of the binder and the aggregate. Then there is the question of the chemical attacks to which the steel within the concrete is exposed in works below ground or below water; and the importance of the resistance offered by materials against chemical attack is appreciated by all who have had occasion to observe the damage suffered by steel or concrete works exposed to aggressive waters. Since this quality is determined not only by strength, but, more particulary, by density, which, in turn, depends upon the methods of concreting adopted — methods which are still comparatively rough and ready — it must henceforth be an object of endeavour on the part of cement manufacturers to improve the quality of the cement, and on the part of the contracting industry to improve the quality of the concrete.

In works constructed for purporses of water transport, the use of trass has been found advantageous as an addition to the aggregates and binders, and in the author's opinion, having regard to the available qualities of binders, this advantage will be retained in the future.

What is of great importance is that the addition of trass certainly does confer upon the concrete the density essential to it. Earlier misgivings as to the addition of trass to blast furnace cement have happily been allayed since such material had proved successful in large harbour works. The amount of trass added must, however, always be regulated according both to local conditions and to the purpose of the concrete, and in the author's opinion it would be a mistake to set up any definite standards in this matter. On the question of water content the author is in agreement with Professor *Ludin* in regarding any excess, producing too fluid a concrete, as harmful to density and strength. The middle way between stamped and poured concrete should always be taken, according to conditions within the shuttering. Whether the result be described as soft concrete or plastic concrete is really more a question of name than of limit to the water content. Where the reinforcing steel is very crowded, the concrete must necessarily be introduced in a somewhat softer condition than where only a small amount of reinforcement, or none at all, is present.

The author is unable to understand the occasional tendency to return to the use of stamped concrete. Sufficient should have been learnt from earlier experience in this respect, as well as from recent research, to show that "earth-damp" concrete has its use only where the vibration process is applied; or for very thin-walled constructions, but not with usual methods of working or in jobs of large dimensions. As regards the mixing and placing of concrete, all methods of placing, whether by the use of channels, chutes, conveyor belts, funnels, pipes or pumps, are to be regarded as of approximately equal merit. Whatever the process adopted, however, it is essential that concrete with the proper water content should arrive in the shuttering without having become unmixed. In many cases the choice of method will be governed by local conditions and by those peculiar to the job. It is true that the adoption, for instance, of either the pump or the conveyor-belt method places a definite upper limit on the water content, but so far as the properties of the resulting concrete are concerned, the methods of placing and mixing are less important than the proper treatment of the concrete within the shuttering. Here the vibration process, provided the dimensions of the work and the water content are suitable for its use, offers the likelihood of considerably increasing the strength and density of the concrete-qualities which, it must be remembered, are in the end dependent not on machines but on the human factor.

In the matter of division into blocks and keying, the mistake is still made of arranging horizontal and vertical working joints from considerations of design and statics alone, without due regard to the exigencies of construction, such as, for instance, adaptation to suit the sizes and numbers of the mixing and placing plants, and according to whether the concrete is to be deposited in one or more layers.

Since every working joint implies an interruption in the monolithic character of the concrete its statical repurcussions cannot always be left out of account- quite apart from the fact that chemical and physical attack usually has its origin in a working joint (more often horizontal than vertical). Endeavours should, therefore, be made to increase the height of the layers to a maximum by the use of "silo" or sliding shuttering, and to adopt vertical joints in preference to horizontal, with suitable precautions. When horizontal joints are unavoidable their roughening to provide a key should never be neglected. In structures which are to be watertight — as, for instance, dry docks — the joints must be carefully filled. The type of such filling adopted in the extension to the Kaiser Dock in Bremerhaven has proved itself entirely successful in six years service, and the author would be ready to use it again, especially since it possesses the advantage that the lead-wool caulking used can, at any time, be easily reconditioned if required; though in the work named above this has not yet been necessary.

Shuttering lined with steel — or iron shuts has the undoubted advantage that it can be stripped from the concrete without damaging the surface of the latter, and that a smooth surface is left. The author, therefore, attaches as great a value to this as to the placing of a framework of rolled steel sections within the concrete to allow of careful and easy arrangement of the reinforcing bars. The additional costs thus involved are relatively small since the rolled sections can be utilised in the statical design of the structure, apart from the fact that they facilitate rapid concreting even where the work is of great height.

The author's observations on finished works have not led him to attach fundamental importance to the question whether mass concrete is to be kept permanently damp. It is true that a great difference between external and internal temperatures may lead to cracking, but this cannot be avoided even if such structures could be permanently kept damp. But the risk can be minimised if the dimensions of the structure and of its component blocks are suitably chosen.

As regards the question of whether or not concrete should be faced with masonry, the author is in favour of *unfaced* concrete, because the presence of a facing necessitates thin layers of the blocks themselves and thus impairs the monolithic character of the concrete, throwing away the chief advantage of this material. If concrete is used in conjunction with steel reinforcement, the facing militates against the full utilisation of the cross section from a statical point of view. Of course, however, the size and slape of the structure will influence the decision to be taken in this respect.

Where the concrete is exposed to external attack the reinforcing bars should be placed further in than usual, 10 cm being regarded as the minimum depth of cover, but the amount depends on the shape of the work. In certain cases a thin wire netting should be embedded at 3 to 4 cm from the surface in order to avoid surface cracking. There would be no objection to a special facing concrete if it could be placed in a single run together with the main concrete and intimately connected to the latter.

Supervision of the execution of the work can never be too careful and conscientious. By this means alone, having regard to the comparative novelty of this material and shortness of experience in its use, is it possible to extend the use of concrete in large works connected with water transport.

In purely reinforced concrete structures it is, of course, necessary to attach much greater importance to the quality of aggregates and binders, the water content, the reinforcement and the preparation of the concrete, because the relative thinness of the structural members and the high stresses imposed upon the material in them entail very careful preparation of the concrete. In construction for water transport, and in foundation work, however, any excessive thinness of reinforced concrete structures are normally avoided for the reasons already given, because, by contrast with bridges and building structures, the statical and chemical demands made upon them are less easy to evaluate and less accurately known. This is not to say that a return should be made to the practice of using excessively massive structures, but only that the mistake should be avoided of replacing the predominantly block-like construction appropriate to water transport by a network of posts and beams. It must always be left to the discretion of the engineer to find a middle way, satisfying on the one hand the statical leanings of the designer and on the other hand taking due account of the susceptibility of purely reinforced concrete work to damage when used below ground or below water.

c) *Conclusion.*

If, in conclusion, the author may be allowed to compare his experience of works below ground and below water with structural engineering above ground he must once again affirm — despite the boldness of his colleagues in that field, which is a matter of continual wonder to him — that it is those engaged on foundation work and in hydraulic engineering who have to contend with greater difficulties. It is impossible for them to adopt finely articulated structures, because the magnitude and manner of incidence of the attack to be apprehended from their enemies, earth and water, are not known — and, moreover, despite the valued mathematical activity of soil mechanists, never can be fully known, because it is not a question of one definite material but rather of a conjunction of conditions, more or less complicated in each particular case.

It appears necessary, therefore, to emphasize the danger of overvaluing a purely theoretical and mathematical conception of the agencies earth and water when from time to time the theorists offer us, who have to design and execute, a basis for our calculations. Construction below ground and under water remains first and foremost a science of experience, though it imposes upon its practitioners the high demand to master theory also so as to be able to evaluate it correctly. A practical engineer "without" theory appears, to the writer, as dangerous as a theorist "without" extended practice.

Colleagues in the field of structural engineering should remember, when drawing up their regulations for concrete and reinforced concrete, that while the knowledge they possess is fully valid for their kind of work it cannot always have the same validity when applied to the subject matter of this paper.

Merely as an example, it may be mentioned that in the reinforced concrete and steel structures designed by the author the permissible stresses laid down for structural engineering above ground were not binding, because to the author the final criterion was the limit of elasticity, always assuming the possibility of assessing the magnitude and direction of the forces in the least favourable case. In other cases, where the structure undergoes movement the amount of which cannot be estimated, stresses must be kept within such limits which are far below those adopted as criteria in normal structural engineering.

The governing factor in the treatment of works below ground and below water is not the values of the stresses, but the correctness of the assumptions as regards incidence of load and as regards movement of the work and its component parts.

One more point may be made. In structural engineering above ground relatively small quantities of concrete and thin sizes are involved, whereas construction below ground and in water is a mass problem. To produce 300 000 m^3 of reinforced concrete in one year in a single job demands an altogether different scale of appraisal than, for instance, 10 000 m^3 of high grade reinforced concrete.

VI 2

The Bridge over the Lagoon at Venice.

Die Brücke über die Lagune in Venedig.

Le pont de la lagune de Venise.

G. Krall,
Professor der Universitäten Rom und Neapel, Rom.

This remarkable structure (Fig. 1) is a bridge 22 m wide and almost 4 km long, which was constructed in the relatively short period of eighteen months by the Società Anonima Italiana Ferro-Beton of Rome, and it is mentioned here as an example of a well-planned job.

Figs. 2, 3, 4 and 5 show the progress of construction. A beginning was made at a point practically midway between Venice and Marghera. Fig. 2 shows the

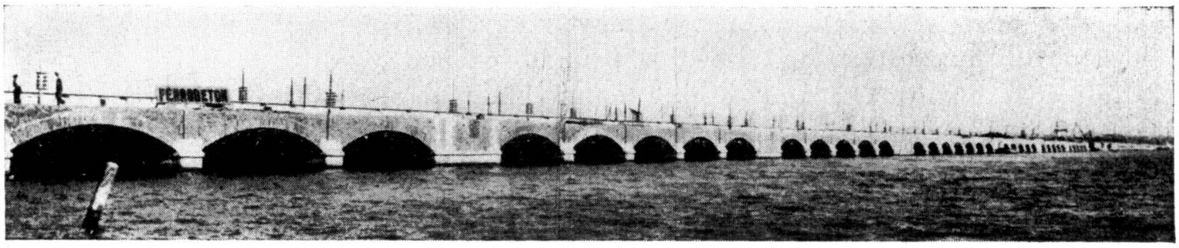

Fig. 1.

driving of the piles; Fig. 3 the construction of the piers; Figs. 4 and 5 that of the arches.

The mechanical plant comprised twelve travelling bridge cranes in two groups, the first group working towards the right, and the second towards the left, and the following details will be given in respect of one of these groups. The first of the bridge cranes was used for driving the steel-sheet-pile cofferdams; the second carried two grabs which were used for excavating a pit measuring 40 m $\times$ 2 m; the third crane carried two moveable electric pile drivers capable of a daily output of up to 1000 linear metres of Considère piles 30 cm $\times$ 30 cm in cross section. The fourth crane was used for constructing the piers, the fifth for placing the masonry blocks, and the sixth for withdrawing the sheet piles.

In this way, at a cost of 300000 man-days in the course of twelve months, a total length of 200 km of piles was driven, 20000 m^3 of concrete were poured, 10000 m^3 of masonry were placed and the arches were completed.

The following particulars of the piers may be given. The small parabolic openings in the piers were provided, not only with a view to saving concrete and stone, but in order to minimise obstruction to the current in the lagoon. In this connection the author may be allowed to mention a problem which arose at

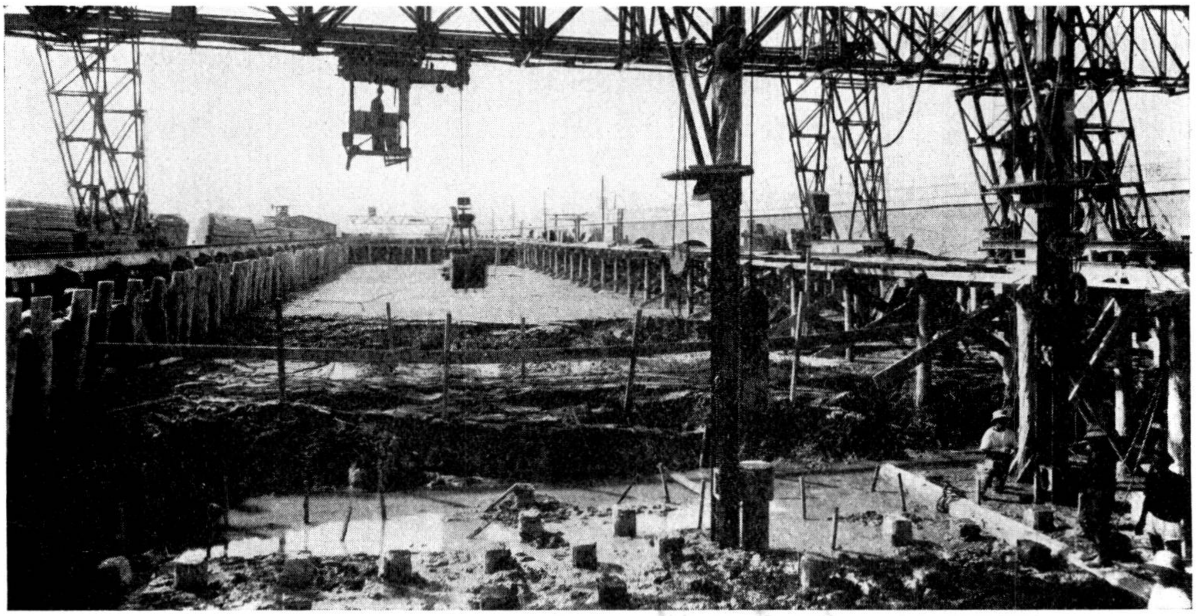

Fig. 2.

the time of the competition for this bridge: assume a uniform and practically endless current of water and assume that in this current there is a pier of cross sectional area A; the shape of this area is to be so arranged that the disturbance

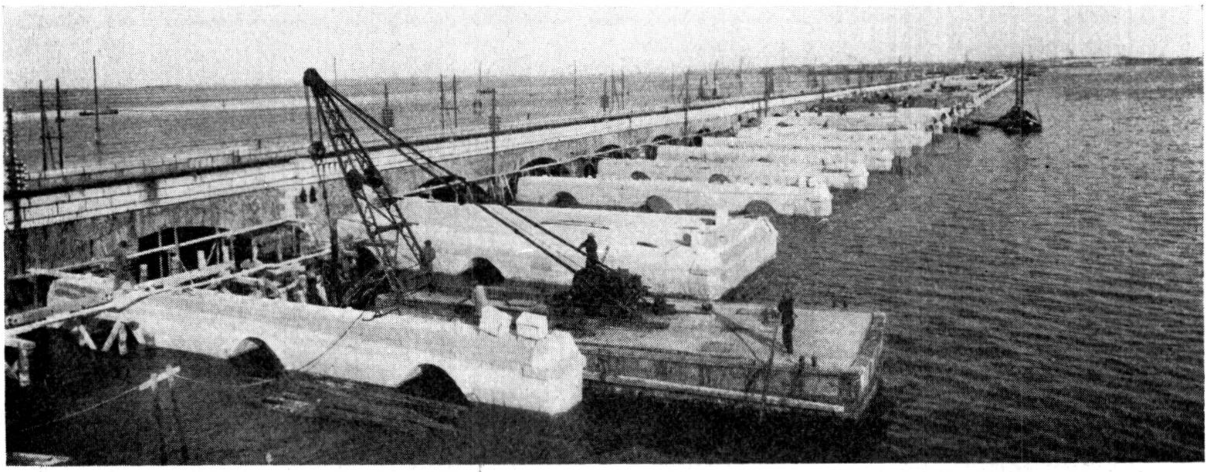

Fig. 3.

of the current is reduced to a minimum. If, now, it is remembered that the magnitude of this disturbance depends on the difference E in the kinetic energy T and T′ before and after the erection of the pier, it will be found that E is independent of the shape of the area A, and is not a function of the perimeter, but is a linear function of the area.

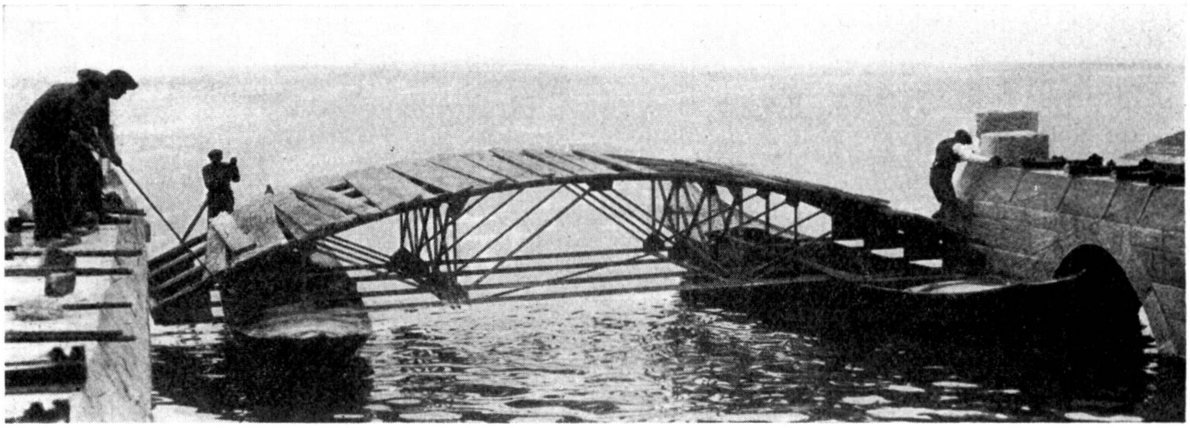

Fig. 4.

From the hypotheses based on a potential of current, it follows that the best solution is that in which the area A of the pier is minimised, and this was in fact the solution adopted.

Fig. 5.

It should also be noticed that the two face walls of the arch afford bracing to the piers, the stability of which would otherwise appear precarious in view of the heavy live load, and the fact that there are only vertical piles. These arguments were confirmed by a number of measurements carried out on the site to study the effect of horizontal loads on the vertically driven piles.

Special attention was paid in the construction to temperature and shrinkage stresses. Making the usual assumption that tensile stresses cannot be resisted and that as a result of the limited cross section or resisting moment which is statically effective the longitudinal force will fall outside the core, it was calculated that adequate safety was assured, and this was confirmed by observations on the finished work.[1, 2]

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[1] See *G. Krall:* Intorno al calcolo degli sforzi di temperatura nelle volte in calcestruzzo o muratura. Il Cemento Armato, 1936, N° 3.

[2] See *A. Signorini:* Sul profilo delle pile da ponte. Rendiconti Accademia da Lincei. Vol. XII, p. 579—581, 1930.

VI 3

Defective Concrete.

Mangelhafter Beton.

Béton défectueux.

Ministerialrat D. Arp,
Reichs- und Preußisches Verkehrsministerium, Berlin.

Great as the advances in concrete and reinforced concrete construction have been during the last few decades, we must not delude ourselves into thinking that perfection in the use of this material is anywhere near being reached. Reports of damage to concrete work from frost or the attack of acid waters or acid subsoils are often made, brought about by an insufficient quantity of the binding medium, incorrect granulation of the aggregates resulting in lack of density, or lack of uniformity in their distribution through carelessness in placing the mixture. Too often we see jobs in which the water has percolated through numerous horizontal working joints to cause hideous stains and efflorescence. Renderings, even if applied by means of compressed air, may show deterioration in places through flacking, and blistering, and so confer a tumble-down appearance on the structure as a whole. What large job in concrete or reinforced concrete is completely free from cracks? The observant visitor is apt to see only these and his impressions are formed accordingly; and it is a false consolation to suppose that most of the cracks do not go far into the concrete mass, for it is extraordinarily difficult to fix with certainty the depth of a crack. Most cracks go much deeper than is generally believed.

It is often said that the cracks in the tension zone of reinforced concrete structures do not impair the stability of the steel, provided the width of such cracks does not exceed a certain limit at the surface; the author, however, is of opinion that all cracks are an evil, and that their presence points to errors in design or in execution which must be avoided whatever the circumstances. Often, when inspecting handsome reinforced concrete bridges only a few years old, he has noticed with regret how the concrete has already begun to flake off here and there, because of rusting at stirrups which have not been held at a sufficient distance from the shuttering.

The defects thus outlined would be of rarer occurrence if the existing regulations were observed with more understanding and if every supervisor and worker on the job were impressed with the delicacy of the work and the meaning of carefulness in his duties. It might well indeed be desirable to fix somewhat stricter limits in these regulations — as, for instance, in regard to the minimum concrete cover for reinforcing bars, the screening curves for the aggregates, and the maximum and minimum limits of water content.

In considering the danger of rusting of reinforcing bars in delicate structures, it appears to the author a matter of regret that up to the present so little attention has been paid to the question of galvanising the steel, an expedient which provides excellent protection. It has long been established that galvanising does not reduce the bonding quality.

On the question of consistency we ought in no circumstances to allow ourselves to be led back, on the strength of laboratory results and theoretical considerations of plasticity, to the use of "earth-damp" concrete. The concrete must always be soft enough to make it fill the shuttering under its own weight as densely as possible, even if the supervisor and workers have occasionally been remiss in their care. In Germany the placing of concrete by means of pumping has rightly attained to a considerable development during the last few years; the great viscosity of mixture has proved an advantage in this method, and the mixture requires no more than 9 % of water in proportion to the dry mix. Unfortunately the concrete pump up to the present has required the exclusive use of stone aggregate measuring not more than 80 mm in any direction. In all the larger concrete and reinforced concrete jobs constructed on the Mittelland Canal during the last few years the whole of the concrete — whether conveyed by pump, conveyor belts, channels or in any other way — has been introduced into the shuttering by means of fixed funnels according to the "Kontraktor" system, which was actually devised for placing concrete under water. The characteristic of this system is that the funnels in question are raised with the concrete in such a way as to keep the lower edge of the funnel always at a certain depth within the fresh concrete. The number of funnels is determined by the size and shape of the plan of the construction. In all these jobs without exception it has been found that concrete poured in this way, so as to cause it to pile up uniformly around the pipe in the dry, is remarkably uniform and dense, and no laitance appears either inside or on the surface. In these jobs horizontal working joints have been as far as possible avoided altogether, so that the whole mass is in fact made monolithic. In the doublelift lock at Allerbüttel, for instance, two walls each 14.2 m high and 9.3 m wide at the bottom were built in this way, in blocks about 15 m long, each block being completed in one non-stop working operation from the bottom to the top without the introduction of any horizontal joint. All fittings in these walls, such as the foot plates, horizontal and vertical edge protectors, ladders and bollards were erected within the shuttering beforehand and so cast into the concrete; waterproof joints and guide rails for the lock gates, roller-sluices, etc. in the lock-heads were installed in the same way.

Despite the successful results obtained in these works a few parts of them exhibit the defects indicated above, namely cracks; some of these being very fine surface cracks, others deeper and even extending right through the mass. At any rate the appearance of such cracks in the blocks constructed during the colder seasons of the year afford an indication of the important part played by temperature effects. In large masses of concrete cracks are apt to be produced at places wherever the stresses from live load, shrinkage and temperature combine to make up a maximum which exceeds the tensile strength of the concrete. In massive concrete works such as lock walls, dams and the like, the shrinkage stresses which result from drying are relatively unimportant (except as regards a thin surface layer) because there is always sufficient moisture present in the interior of the concrete mass to mitigate this effect. It is chiefly the temperature stresses that are responsible for the cracks occurring in the cross-walls. These temperature influences give rise to considerable secondary stresses which have not, as a rule been taken into account when calculating the stability of the work, and they endanger the concrete at a time when its strength has not yet reached an appreciable value.

The cracks most frequently observed are vertical transverse cracks in the walls. These are the least dangerous kind, since, when all is said, they merely represent an addition to the number of joints purposely provided, and their disadvantage lies mainly in impairing the water tightness. Horizontal cracks also occur, and these are more serious. The worst of all are longitudinal cracks which destroy the assumed integrity of the cross section. In the finished work they can be detected only if there are internal passages such as for instance the circulating channel in a lock or the inspection shaft and galleries in dams. *Vogt*,[1] who has made an investigation of practically all dams existing in the world built up to 1930, gives many examples of vertical, horizontal and longitudinal cracks in such structures, and very few dams are mentioned by him as being completely free from any kind of cracks. Similar experience has been recorded in concrete dams since constructed in Germany, regarding which Professor *Ludin* has given an account before the Congress.

The fact that stresses in the concrete may attain considerable values on account of temperature changes alone is brought home when one considers the magnitude of the heat changes to which large concrete masses are exposed, and the movements to which they are compelled through the effect of such changes. Concrete which has been placed at a mixing temperature of $+ 25^0$ C will thereupon harden in accordance with the dimensions of the shuttering, but in the course of time its volume must diminish to correspond with the mean annual temperature, which may be a matter of $+ 10^0$ C. Assuming a coefficient of thermal expansion of 0.000012 a wall block 15 m long constructed in this way must suffer a contraction of 2.7 mm in cooling to the mean annual temperature, if its movement is not restrained.

The most far reaching effects are those which result from the increase in temperature which the concrete in large blocks undergoes in the course of setting. With the usual conditions of mixing, the increase in temperature of the concrete in usually medium sized walls of ship canal locks may reach 35 to 40^0 C in the interior of the concrete. In large gravity dams the difference between the internal and external temperature may be even greater, since the escape of heat from one block is hindered by the construction of its neighbour immediately or soon afterwards. The magnitude of the stresses imposed by the gradual cooling before the final temperature is reached may be gathered from the consideration that the volume of a block of approximately 1000 m^3 content, which has been formed at

[1] Prof. Dr. *Fredrik Vogt:* Shrinkage and cracks in concrete of dams. D.K.N.V.S. Skrifters, Trondheim, 1930, N° 4.

a mixing temperature of $+ 25^0$ C and has its temperature increased to 40^0 C in setting, must shrink approximately 2 m^3 smaller than its maximum size in the process of reaching a temperature of $+ 10^0$ C.

The firm rock foundation below a dam participates in the expansion and contraction of the concrete only to a small extent, and in the same way the lower blocks of a large gravity dam, which have already partly cooled off, hinder the movements of the fresh blocks formed above them. It is clear that in a concrete dam many movements to and fro must take place, giving rise to considerable stresses before any load from water pressure has been imposed. The result is that the true conditions of stress in such a dam are exceptionally difficult to determine, especially so in the first few years after filling with water.

What, then, are the measures which can and should be taken to minimise the formation of cracks in concrete? Of course an eminently suitable cement should be chosen, which besides having as high a tensile strength and elongation as possible, shows very low shrinkage and, above all, produces a minimum amount of heat. In Germany it has been sought to promote these qualities in nearly all the newer dam works by making additions to the cement, particularly the additions of trass, or alternatively in a few cases certain other lime-binding materials such as "Thurament" which is a basic blast furnace slag cement which has been merely ground but not otherwise specially treated. In the Saaletal dam near Hohenwarte, now under construction, a mixture of three components — Portland cement, trass and Thurament — is being used in the proportions of 36 parts by weight of the first mentioned, 40 of the second and 24 of the third. It must always be borne in mind, however, that whatever care may be exercised in the choice and composition of the binding material the secondary stresses, which may give rise to the formation of cracks, are diminished only to a small extent.

Further, the water cement ratio should be kept as low as possible, in order that no reduction in strength may occur. Should one, from fear of cracking, return to the use of "earth-damp" concrete placed in thin layers and rammed or vibrated, in order that the setting heat may to a great extent escape into the air? In the author's opinion concrete so made is the worst kind of all and, moreover, in a dam containing several hundred thousand cubic metres of concrete there is not time to delay the sequence of layers to such an extent that the accumulation of setting heat may be avoided.

The richer the mix the greater the shrinkage and hence the greater the setting heat. Let us, then — it is sometimes suggested — adopt the leanest possible concrete; but a lean concrete cannot be expected to be watertight or to resist chemical and atmospheric effects. In view of these considerations a few recent dams have been built with a thick core of lean concrete and the outer layer, both on the water and on the dry sides of the dam, of rich concrete, the expectation being that in this way the dam as a whole will undergo smaller amounts of movement from shrinkage and temperature changes. Where is the guarantee, however, that the stresses in the contact zone between the rich and the lean concrete will not exceed the permissible amount, with a consequent risk of crack formation in course of time — cracks which may run along the length of the dam and endanger its stability? Again, how will it be possible to prevent water from percolating in course of time to the lean core, where it may bring about chemical changes? The author would not advise such a method of construction. He is entirely of the opinion that nowhere in hydraulic construction should a lean concrete be used in a position where the water may penetrate in any way. The expressions "lean concrete" and "economy concrete" should be struck right out of the technical vocabulary.

He looks upon these devices for avoiding cracks as being of little or no use, and sees only one possibility of serving the desired purpose, which is by cooling the concrete.

The components of the concrete may be cooled before or during the mixing process, or the concrete may be cooled after its introduction into the shuttering, or these methods may be combined. The cooling of the ingredients of the mix

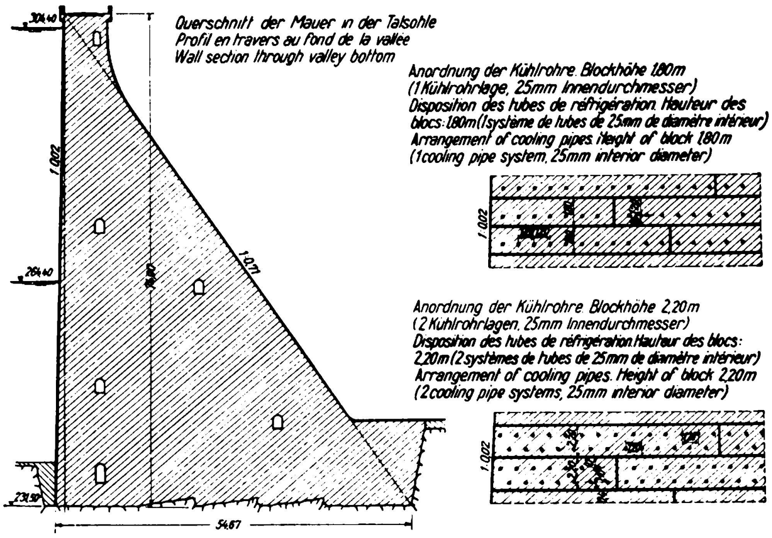


Fig. 1.

Cross section of upper Saale river barrage near Hohenwarte, showing arrangement of internal cooling tubes.

is very effective during the hot part of the year and enables the heat curve to be considerably lowered, but in this way no direct influence can be exerted over the development of the harmful setting heat. That can be done only by cooling the concrete actually placed.

By spraying all concrete which is fresh and has not yet fully hardened with water for a long time, with the object of rendering the drying process more uniform and of providing an adequate supply of water to the surface layers for the continuation of the setting process, a considerable reduction in the excess heat from the concrete mass may, of course, be secured, but this effect cannot penetrate right into the interior of large blocks.

In the case of the Grimsel Dam in Switzerland, and also in other dams, large slots have been allowed to remain open for a long period between the individual large constructional blocks and also at certain of the vertical working joints, before being filled up with concrete, the idea being that in the meantime the air might serve as a means of escape for the heat within the concrete, which would thus be more rapidly drawn away. The author has no exact information as to the success of these measures, but it is clear that no extensive and uniform effects in preventing the formation of temperature cracks can be expected from them.

The only rational method appears to him to lie in the internal cooling of the concrete wall by a system of cooling pipes uniformly distributed through the cross section at not too great intervals. Such cooling is being applied with success by the American engineers in the Boulder Dam in Colorado now approaching completion, though in that instance the reason for the adoption of cooling was not the question of temperature cracks, but the desire to use this means of bringing various sectors of the arc dam to their final dimensions as quickly as possible so that the gaps separating the elements of the arch might be filled with cement grout under pressure, in time to enable the dam to stand the water

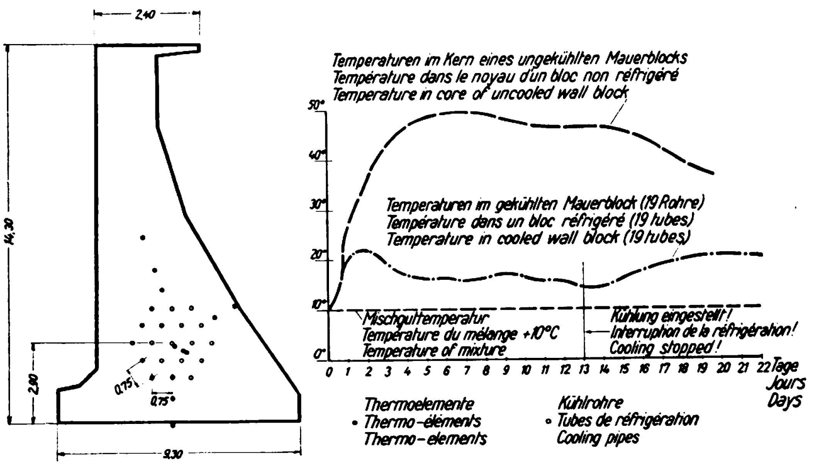


Fig. 2.

Artificial cooling of the concrete wall of the Allerbüttel double barrage (German Midlandcanal).

pressure. But the un-sought for absence of cracks in the concrete was an additional advantage of the cooling system not originally intended.

This example of the Boulder Dam serves to bring out the advantage of the internal cooling of the concrete applied to the German dam, already mentioned, in the upper Saale near Hohenwarte, in which the concreting was begun in the late autumn of 1936. Here it is a question not of an arch but of a gravity dam, so that the grouting of the gaps under pressure is of secondary importance and the only purpose of the cooling is to carry away the heat of setting at such a rate that no harmful stresses will arise and entail the risk of crack formation. It may be seen from Fig. 1 how the cooling pipes are distributed over the whole cross section within any of the working blocks. The bore of the iron pipes is 25 mm and the horizontal interval 1.2 m, the vertical interval depending on the thickness of the layers in the blocks, which is not yet fixed. If this is taken as 1.8 m then that dimension will also correspond to the vertical spacing of the cooling pipes; where layers more than 2.20 m high are adopted two rows of cooling pipes will be used having an interval equal to half the height of the block, that is to say, 1.10 m. The pipes will carry a slow circulation of water which has been cooled down to between + 6 and + 9° C, until the working block in question has reached the mean annual temperature, and then, as was done at the Boulder Dam, the pipes will be filled with cement grout under pressure. The Hohenwarte dam is approximately 400 m long at the crown and is 75 m high at the deepest part, and the total volume of concrete will be approximately 450,000 m^3; altogether some 200,000 m of cooling pipe will be necessary apart from the supply and return connections. The additional cost due to the provision of cooling will be about 3.5 % of the total cost of the work not including the power station, or approximately 6 % of the cost of the dam by itself.

In addition to the internal cooling of the concrete blocks, provision has here been made for cooling the mixture in the mixing plant during the summer months.

At Hohenwarte the cooling through the water pipes is not, as in the case of the Boulder Dam, to be delayed for some weeks after a block has been concreted, but is to be begun immediately after the concreting, with a view to cutting off the peak of the heat curve in advance. Criticisms to the effect that the setting heat is being removed are not valid, since it is a question of removing the heat which has already been set free by the chemical process, and the slowing down of the hardening process of the concrete by the application of cooling to reduce the temperature can be regarded only as an advantage.

The fact that the proposed cooling process is attended by no disadvantages is shown by experiments carried out on large concrete blocks in the Institute for Frost Investigation at Magdeburg—Glindenberg, and also in actual practice in the double lift lock at Allerbüttel to which reference has already been made. In the latter a number of the blocks the lock walls were provided with cooling water pipes arranged in different ways, one such arrangement being that shown in Fig. 2 which also serves to indicate the remaining measurements of the wall. The temperature of the cooling water in this experiment was 7^0 to 9^0 C. In other experiments it was higher, as the water from the neighbouring canal was taken during the warmer part of the year. The variation in temperature in these, as well as in a few blocks which were not cooled and in the ground below, was measured by means of electrical thermometers and automatically recorded. The curves of temperature reproduced in the figures indicate that the cooling was effective in carrying away a considerable amount of the heat of setting, the height of the cooling curve being reduced by some two-thirds. In other wall blocks at Allerbüttel, in which the spacing and diameter of the pipes was varied, the results obtained corresponded to these variations. In none of the cooled blocks did any cracks arise, though the pipes were provided only in the core.

These experiments at the Allerbüttel lock serve at the same time to indicate the possibility of applying the concrete cooling process even in works of smaller dimensions, making use of relatively simple means, and in this way to protect the work from the cracking which might otherwise occur. The small expense of cooling is out of all proportion to the great value thereby obtained.

VI 4

Temperature Rise in Concrete Dams.

Temperaturerhöhung in Betonstaumauern.

L'échauffement dans les barrages en béton.

N. Davey,
Ph. D., B. Sc., Assoc. M. Inst. C. E., Garston.

In large masses of concrete, such as dams, the central portion loses heat very slowly, and the concrete is cured under almost adiabatic conditions; as a result a very high temperature may be reached. The rise in temperature will depend on the type of cement used, as indicated by the figures in Table 1, the mix proportions, the size of mass of the concrete, the rate of placing, the insulation afforded by the shuttering and the external conditions.

Table 1.
Heat Evolved by Different Types of Cement.

Type of cement	Number of consignments tested	Heat evolved in g — calories per g at the end of		
		1 day	2 days	3 days
Normal Portland cement	13	23—42	42—65	47—75
Rapid-hardening Portland cement	13	35—71	45—89	51—94
Portland blast-furnace cement	6	18—28	30—51	33—67
High-alumina cement	3	77—93	78—94	78—95

Some attention has been given at the Building Research Station to the effect of heat evolution on the strength and other properties of concrete.

It has been found that the strength is developed more rapidly at the centre of a mass of concrete, where the temperature is higher, than at the edges. In one mass measuring 3 ft. $\times$ 4 ft. $\times$ 2 ft. 6 in., in which concrete composed of one part of rapid-hardening Portland cement, two parts of river sand and four parts of gravel, and having a water-cement ratio of 0.6 by weight was used, the strength after 3 days of the concrete at the centre was found to be over 50 per cent. greater than that at the corners where the loss of heat through the shuttering was greater.

Since both shrinkage and creep vary with strength it is reasonable to suppose that they also will vary throughout the mass of the concrete. In addition, concrete in large masses hardens at a time when the temperature increase due to beat evolution is considerable and, consequently, the return of the concrete to normal temperature, which in some cases may take many months or even years, must be accompanied by a heat contraction which is additional to any shrinkage effects.

It is particularly desirable that these temperature effects in mass concrete should be reduced to a minimum by the use of selected cement and by the careful design of the concrete mix, and that it should be possible to predict from the results of laboratory tests of the cement the temperature that may be attained in a mass of concrete made with it. The study of the problem at the Building Research Station[1] has therefore been extended with the object of obtaining records of the temperatures attained in the concrete of three large dams, and to compare these records with the time-temperature curves given by laboratory tests of the cements used for these works. Although further tests are needed before an exact correlation can be established, approximate relations can already be given which may be used for the tentative prediction of the temperatures that will be reached in a large mass of concrete made with a given cement.

The observations may be conveniently grouped into two series: one series made on concrete deposited in the Tongland and Clatteringshaws Dams of the Galloway Water Power Works, and the other series on concrete deposited in the Laggan Dam of the Lochaber Water Power Works.

The Tongland Dam, across the River Dee, near Kirkcudbright, is about 850 ft. in length and includes an arch dam in reinforced concrete and a gravity section. The Clatteringshaws Dam is of the gravity type and has a total length of 1450 ft. across the Blackwater of Dee. The Laggan Dam, near Fort William, is approximately 700 ft. long and 138 ft. high, and is of the gravity type. In the Tongland and Clatteringshaws Dams the concrete was placed in lifts varying in depth from 4 ft. 6 in. to 6 ft. 0 in. and in the Laggan Dam from approximately 3 ft. 3 in. to 3 ft. 9 in.

It was possible to observe the temperature rise in masses of concrete placed in the Tongland and Clatteringshaws Dams. The observations were made by inserting a maximum thermometer in a pipe embedded in the mass.

Each mass constituted a lift poured in one operation, the depth of the lift varying from 4 ft. 6 in. to 6 ft. 0 in. Class "0" concrete (3 cwt. cement, 12 cu ft. Gatehouse sand and 20 cu ft. of Porphyrite aggregate) was used in all these masses, except at the Clatteringshaws Dam in which 12 per cent. of displacers were added. Samples of the cement and aggregates were forwarded to the Building Research Station and the temperature rise was measured on completely insulated samples of concrete using the same mix as used in the actual masses.

In the Laggan Dam the temperatures were observed in the central portion of the dam by means of a series of Cambridge resistance thermometers. The concrete was placed in lifts of approximately 3 ft. 6 in. and the mix contained 370 lb. of cement per cu yd. The water added to each batch varied considerably and depended on the amount of moisture present in the sand and aggregate, but the mix was of a stiff consistence. The granite displacers averaged approximately 5 per cent. of the total mass of concrete deposited. The shuttering was

[1] *Davey, N.:* "Correlation between Laboratory Tests and Observed Temperature in Large Dams". Building Research Technical Paper N° 18, 1935.

of tongued and grooved timber 2 in. thick with necessary bracing. The freshly deposited concrete was covered with heavy coconut matting immediately after placing. This matting was kept wet until it was necessary to lift it for placing the succeeding lift.

An examination of the rise in temperature at the interior of lifts of concrete showed that in the majority of instances, particularly in the Laggan tests, two peaks were reached. The first rise in temperature is very rapid and in actual practice may exceed in rapidity the rise recorded on samples of similar concrete placed at the same temperature and cured in the laboratory under adiabatic conditions; the reason being that a certain amount of heat is received from the preceding lift. The first rise is followed by a less rapid fall in temperature, but

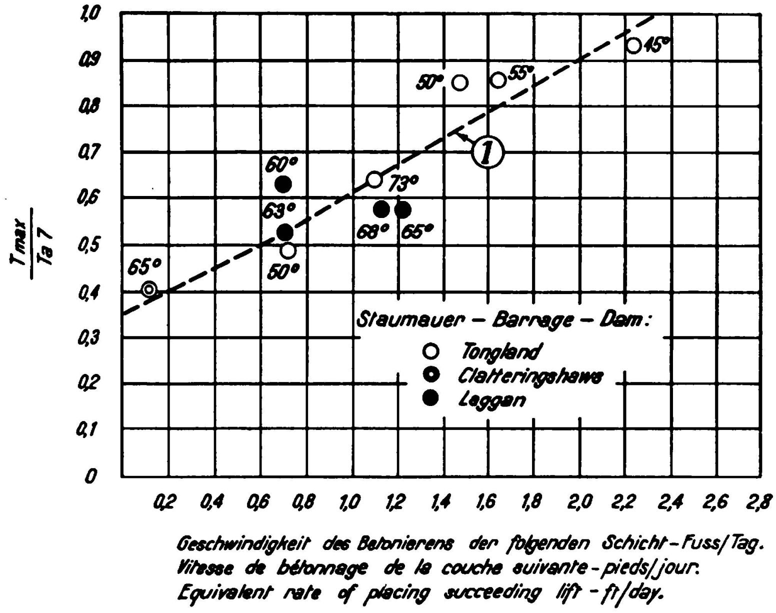


Fig. 1.

Temperature rise in mass concrete.
(The placing temperatures are indicated.)

this fall is arrested by the heat received from the succeeding lift of concrete and a second peak temperature is experienced.

As a general conclusion it may be stated that in placing concrete in lifts of equal thickness and of the same proportions of mix the first peak temperature T_{max} in any particular lift is dependent upon the age of the preceding lift, and the second peak temperature T'_{max} is dependent upon the time interval before the succeeding lift is placed.

Figure 1 gives the relation between the ratio $\frac{T_{max}}{T_{a7}}$ and the equivalent rate of placing the preceding lift, T_{a7} being the temperature rise in a completely insulated sample after 7 days; in Figure 2 is given the relation between the ratio $\frac{T'_{max} - T_{max}}{T_{max}}$ and the rate of placing the succeeding lift. If, then, the value T_{a7} is known it is possible to determine with very fair accuracy the temperature rise likely to occur in a mass of concrete of similar thickness and placed under similar temperature conditions as those recorded here.

The existence of high temperatures in the heart of the dam with a much lower temperature at the surface must result in the development of high stresses near the surface. It is therefore essential that some idea of the maximum temperature rise that can be allowed without the formation of cracking, due to temperature, should be obtained. Observations made on the Laggan Dam have

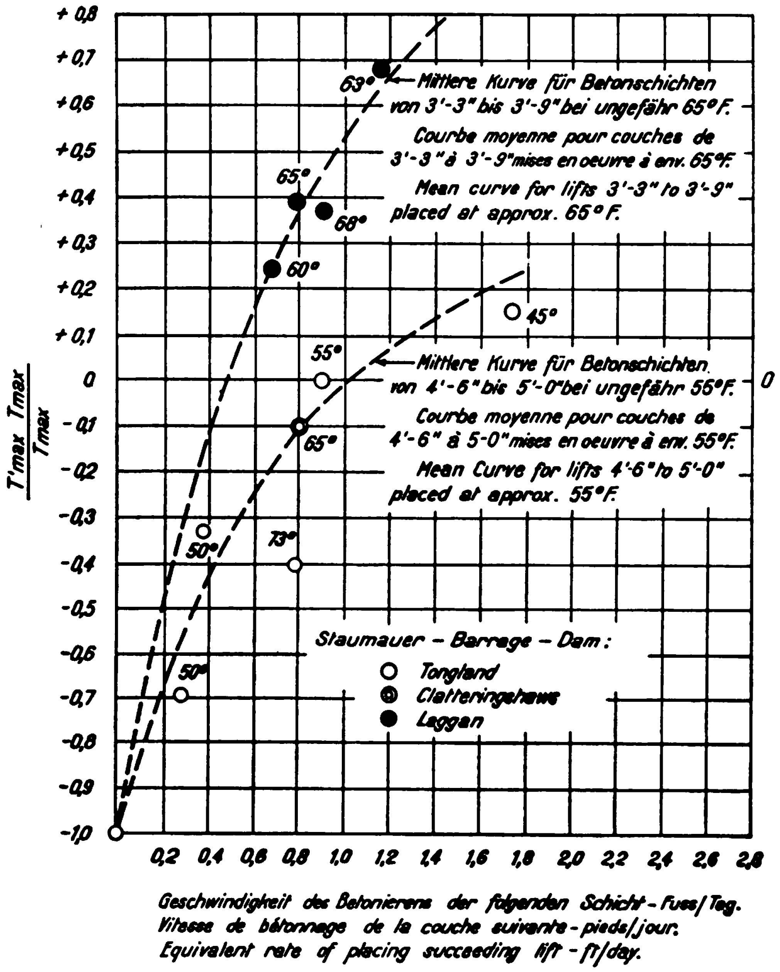


Fig. 2.

Temperature rise in mass concrete.

(The placing temperatures are indicated.)

been helpful on this point. The time at which the maximum difference in temperature occurred between the concrete in the heart of the dam and at the surface has been recorded. This corresponded with the date at which the concrete in the heart attained its second maximum temperature (T'_{max}). In the case of block IV south of the dam this date was 4 to 5 weeks after depositing the concrete in the heart of the block.

An examination of the temperatures recorded at points Nos. 5, 4 and 3 — 3 ft., 23 ft. and 43 ft. respectively, from the upstream face of the dam during the

10 days following 27th July, 1933 — is interesting in that it shows how quickly the gradient changed near the surface of the dam.

date	Thermometer No. 3	Thermometer No. 4	Thermometer No. 5	Air Temperature
27. 7. 33	109° F	106.5° F	108° F	62° F
29. 7. 33	109	107	97	54
31. 7. 33	108.5	107	90	60
2. 8. 33	108	107	88	54
4. 8. 33	108.5	108	84	55
6. 8. 33	108.5	108	83	55

The gradient observed on 6th August, 1933 is shown in Figure 3. From a point about 10 ft. in the mass to the exposed surface the gradient is seen to be about 50° F. The difference in temperature between the heart of the dam and the air at the exposed surface approached 55° F. This amount is not entirely

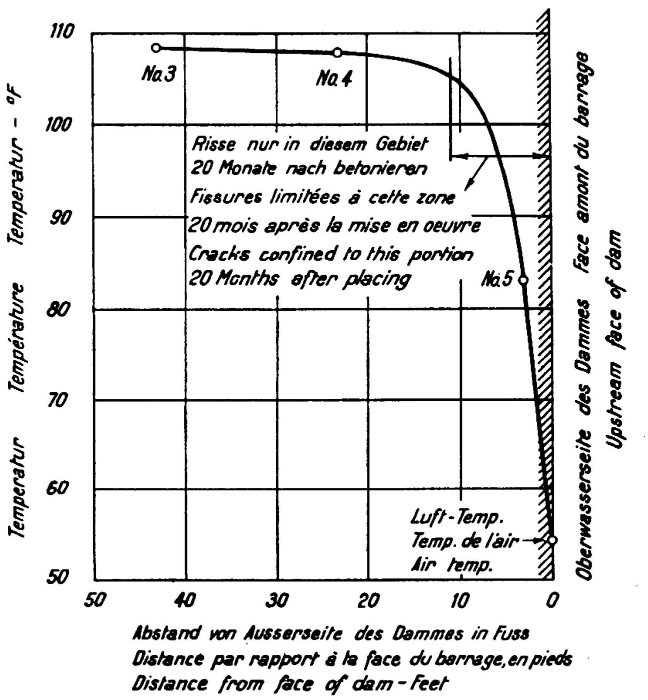


Fig. 3.

Temperature gradient in Laggan Dam, 6th Aug. 1933.

due to the rise in temperature of the concrete due to the heat of hydration (approximately 45° F) but also to a fall in the average air temperature of about 10° F. Cracks became visible in the concrete by the end of the month, but the cracks had not penetrated as far as the inspection gallery, situated 8 to 11 ft. from the upstream face of the dam, at the end of 20 months. The cracks are therefore confined to the surface and seem to be definitely accounted for by the development of the sharp temperature gradient in this region.

Taking a value for the effective modulus of elasticity of 1×10^6 lb. per sq. in. for the concrete which had an average compressive strength of approximately 2,800 lb. per sq. in. at 28 days, and a coefficient of thermal expansion of $\cdot 6 \times 10^{-6}$ per 1^0 F, the tensile stress set up due to the temperature gradient alone would probably exceed 300 lb. per sq. in. This value is excessive and would result in cracking. If the risk of cracking is to be reduced to within reasonable limits the tensile stress in the concrete (due to temperature) should not be allowed to exceed at the most 150 lb. per sq. in. This in turn means that the temperature difference between the centre and the surface of a mass should not exceed about 25^0 F. To achieve this the rate of placing of a concrete similar to and placed at the same temperature as that used in the Laggan Dam, with a cement which generates 65 calories per gram at 7 days, would have to be restricted to 0.5 ft. per day i. e. lifts of 3 ft. 6 in. placed at intervals of not less than 7 days. This figure assumes that the average air temperature from the time of depositing the concrete to the time when the maximum gradient occurs does not change appreciably. Seasonal temperature fluctuations will have the effect of increasing or decreasing the gradient through the dam. With a rising air temperature the gradient would tend to be less steep and the reverse effect would be anticipated if the air temperature was steadily falling. If cement which generated only 55 calories per gram after 7 days were used, the rate of placing could be increased by about ·20 per cent. without incurring additional risk of cracking.

It cannot be claimed that finality has been reached in this investigation, which must be extended to cover conditions of placing other than those of the Laggan and Galloway Dams.

VI 5

Elastically Built-in Arch Dams.

Elastisch eingespanntes Talsperrengewölbe.

L'arc de barrage élastiquement encastré.

Dr. sc. techn. K. Hofacker,
Zürich.

The expression "arch dam" will be used to indicate an arch having its axis bent to a circular curve and having constant thickness which may be large in proportion to the span. By contrast with the "bridge arch", which may be calculated accurately enough on the basis of *Navier*'s theory of bending, the arch dam requires to be analysed by reference to the mathematical theory of elasticity if an accurate picture of the real conditions of stress is to be obtained.

If the water pressure acting on an arch dam is devided up in the customary way between a combined system of horizontal arches and vertical cantilevers, then any desired loading diagram may be drawn for each of the separate elements. The method of calculating the stresses in the vertical slab-shaped elements of a beam in accordance with the actual conditions of stress and strain has been known for a long time, even experimental investigations in this direction have frequently been carried ont. The calculation of stresses in the horizontal curved elements of an arch has hitherto been worked out only for the special case of a rigidly fixed arch; neither does the author know of any exact measurements of the stresses or strains effected in the laboratory on models of such arch dams. It was, therefore, deemed to be of special interest to carry out a theoretical and experimental investigation into the general question of an elastically built-in arch loaded with any desired water pressure.[1]

We will assume a slice of annular shape subjected to the uniplanar conditions of stress shown in Fig. 1.

Any given loading diagram may be represented with the aid of a Fourier mathematical series:

$$\sigma'_r = A'_o + \sum_{n=1}^{\infty} A'_n \cdot \cos n\varphi + \sum_{n=1}^{\infty} B'_n \cdot \sin n\varphi \tag{1}$$

In Fig. 2 we see the stresses operating on an element dF at the point 0 and may write down the condition of equilibrium. In view of the relation between stresses and elongations, that is to say the differences in displacements u and v

[1] *K. Hofacker:* Das Talsperrengewölbe, 1936. Gebr. Leemann & Co., Zürich.

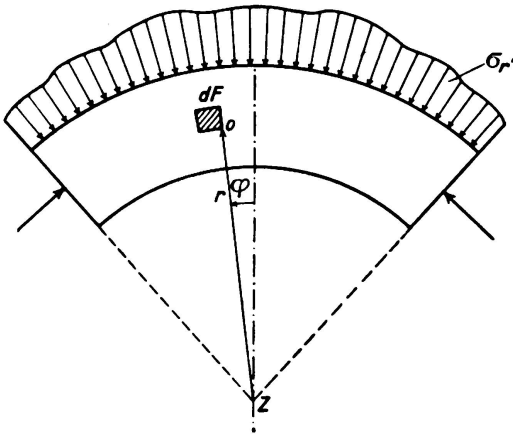


Fig. 1.

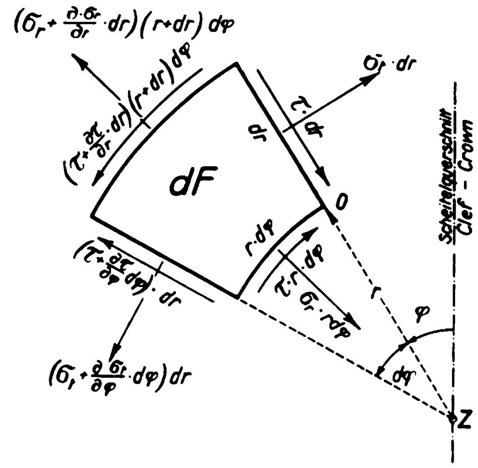


Fig. 2.

measured in a radial and a tangential direction respectively, we obtain the two differential equations:

$$\begin{aligned}&\frac{1}{r^2}\frac{\partial^2 u}{\partial \varphi^2}+\frac{2m}{m-1}\left(\frac{\partial^2 u}{\partial r^2}+\frac{1}{r}\frac{\partial u}{\partial r}-\frac{u}{r^2}\right)\\&+\frac{1}{r}\frac{\partial^2 v}{\partial r\cdot\partial\varphi}-\frac{1}{r^2}\frac{\partial v}{\partial\varphi}+\frac{2m}{m-1}\left(\frac{1}{mr}\frac{\partial^2 v}{\partial r\cdot\partial\varphi}-\frac{1}{r^2}\frac{\partial v}{\partial\varphi}\right)=0\end{aligned}\tag{2}$$

$$\begin{aligned}&\frac{\partial^2 v}{\partial r^2}+\frac{1}{r}\frac{\partial v}{\partial r}-\frac{v}{r^2}+\frac{2m}{m-1}\frac{1}{r^2}\frac{\partial^2 v}{\partial\varphi^2}\\&+\frac{\partial^2 u}{\partial r\cdot\partial\varphi}\cdot\frac{1}{r}\frac{m+1}{m-1}+\frac{\partial u}{\partial\varphi}\cdot\frac{1}{r^2}\cdot\frac{3m-1}{m-1}=0\end{aligned}\tag{3}$$

The general solutions for u and v read as follows:

Radial displacements

$$\begin{aligned}E\cdot u=&-\frac{m+1}{m}\cdot\frac{a_0}{r}+\left[\frac{2(m-1)}{m}\cdot b_0-\frac{m+1}{m}c_0\right]\cdot r+\frac{2(m-1)}{m}\cdot c_0\cdot r\lg r\\&+\left(\frac{m-1}{2m}\cdot a_1+2\beta_1\right)\varphi\cdot\sin\varphi-\left(\frac{m-1}{2m}\cdot c_1+2\delta_1\right)\varphi\cdot\cos\varphi\\&+\left[\left(a_1+\frac{m-1}{m}\cdot\beta_1\right)\lg r+\frac{m-3}{m}\cdot b_1 r^2+\frac{m+1}{m}\frac{\alpha_1}{r^2}\right]\cos\varphi\\&+\left[\left(c_1+\frac{m-1}{m}\cdot\delta_1\right)\lg r+\frac{m-3}{m}\cdot d_1 r^2+\frac{m+1}{m}\cdot\frac{\gamma_1}{r^2}\right]\sin\varphi\\&+\sum_{n=2}^{\infty}\left[-\frac{m+1}{m}\cdot n\cdot a_n\cdot r^{n-1}-\left(\frac{2n}{m}+(n-2)\frac{m-1}{m}\right)b_n\cdot r^{n+1}\right.\\&\qquad\left.+\frac{m+1}{m}\cdot n\cdot\alpha_n\cdot r^{-n-1}+\left(\frac{2n}{m}+(n+2)\frac{m-1}{m}\right)\cdot\beta_n\cdot r^{-n+1}\right]\cos n\varphi\\&+\sum_{n=2}^{\infty}\left[-\frac{m+1}{m}\cdot n\cdot c_n\cdot r^{n-1}-\left(\frac{2n}{m}+(n-2)\frac{m-1}{m}\right)\cdot d_n\cdot r^{n+1}\right.\\&\qquad\left.+\frac{m+1}{m}\cdot n\cdot\gamma_n\cdot r^{-n-1}+\left(\frac{2n}{m}+(n+2)\frac{m-1}{m}\right)\cdot\delta_n\cdot r^{-n+1}\right]\sin n\varphi\end{aligned}\tag{4}$$

Tangential displacements

$$E \cdot v = -\frac{m+1}{m} \cdot \frac{\alpha_0}{r} + 4 c_0 r \cdot \varphi + \left(\frac{m-1}{2m} \cdot a_1 + 2\beta_1\right) \varphi \cdot \cos\varphi$$
$$+ \left(\frac{m-1}{2m} \cdot c_1 + 2\delta_1\right) \varphi \cdot \sin\varphi$$
$$+ \left[-\left(a_1 + \frac{m-1}{m} \cdot \beta_1\right) \lg r - \frac{m+1}{2m} \cdot a_1 + \frac{5m+1}{m} b_1 r^2 + \frac{m+1}{m} \cdot \frac{\alpha_1}{r^2} - \frac{m+1}{m} \cdot \beta_1\right] \sin\varphi$$
$$+ \left[\left(c_1 + \frac{m-1}{m} \cdot \delta_1\right) \lg r + \frac{m+1}{2m} \cdot c_1 - \frac{5m+1}{m} \cdot d_1 r^2 - \frac{m+1}{m} \cdot \frac{\gamma_1}{r^2} + \frac{m+1}{m} \cdot \delta_1\right] \cos\varphi \tag{5}$$
$$+ \sum_{n=2}^{\infty} \left[\frac{m+1}{m} \cdot n \cdot a_n \cdot r^{n-1} + \left(n \frac{m+1}{m} + 4\right) b_n \cdot r^{n+1} + \frac{m+1}{m} \cdot n \cdot \alpha_n \cdot r^{-n-1} + \left(n \frac{m+1}{m} - 4\right) \cdot \beta_n \cdot r^{-n+1}\right] \sin n\varphi$$
$$+ \sum_{n=2}^{\infty} \left[-\frac{m+1}{m} \cdot n \cdot c_n \cdot r^{n-1} - \left(n \frac{m+1}{m} + 4\right) \cdot d_n \cdot r^{n+1} - \frac{m+1}{m} \cdot n \cdot \gamma_n \cdot r^{-n-1} - \left(n \frac{m+1}{m} - 4\right) \cdot \delta_n \cdot r^{-n+1}\right] \cos n\varphi$$

From these displacements the stresses may be calculated as follows:

Radial stress

$$\sigma_r = \frac{a_0}{r^2} + 2 b_0 + c_0 (2 \lg r + 1) + \left(\frac{a_1 + \beta_1}{r} + 2 b_1 r - \frac{2\alpha_1}{r^3}\right) \cos\varphi + \left(\frac{c_1 + \delta_1}{r} + 2 d_1 r - \frac{2\gamma_1}{r^3}\right) \sin\varphi$$
$$+ \sum_{n=2}^{\infty} [n(1-n) \cdot a_n \cdot r^{n-2} + (n - n^2 + 2) b_n r^n - n(n+1) \cdot \alpha_n \cdot r^{-n-2} - (n^2 + n - 2) \beta_n \cdot r^{-n}] \cos n\varphi \tag{6}$$
$$+ \sum_{n=2}^{\infty} [n(1-n) \cdot c_n \cdot r^{n-2} + (n - n^2 + 2) \cdot d_n r^n - n(n+1) \cdot \gamma_n \cdot r^{-n-2} - (n^2 + n - 2) \cdot \delta_n \cdot r^{-n}] \sin n\varphi$$

Tangential stress

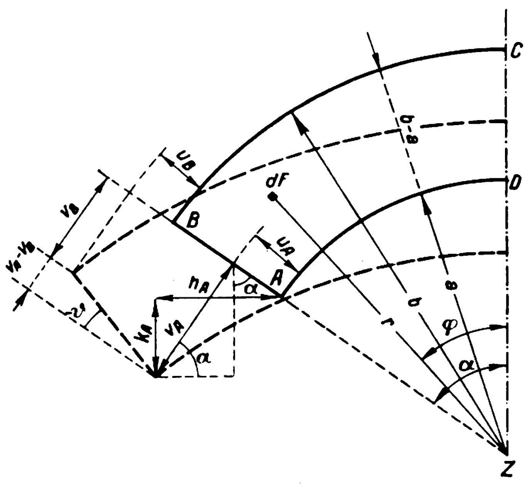
$$\sigma_t = -\frac{a_0}{r^2} + 2 b_0 + c_0 (2 \lg r + 3) + \left(6 b_1 r + \frac{2\alpha_1}{r^3} + \frac{\beta_1}{r}\right) \cos\varphi + \left(6 d_1 r + \frac{2\gamma_1}{r^3} + \frac{\delta_1}{r}\right) \sin\varphi$$
$$+ \sum_{n=2}^{\infty} [n(n-1) \cdot a_n \cdot r^{n-2} + (n+1)(n+2) \cdot b_n \cdot r^n + n(n+1) \cdot \alpha_n \cdot r^{-n-2} + (n-2)(n-1) \cdot \beta_n \cdot r^{-n}] \cos n\varphi \tag{7}$$
$$+ \sum_{n=2}^{\infty} [n(n-1) \cdot c_n \cdot r^{n-2} + (n+1)(n+2) \cdot d_n \cdot r^n + n(n+1) \cdot \gamma_n \cdot r^{-n-2} + (n-2)(n-1) \cdot \delta_n \cdot r^{-n}] \sin n\varphi$$

Shear stress

$$
\begin{aligned}
\tau = \frac{\alpha_0}{r^2} &+ \left(2 b_1 r - \frac{2\alpha_1}{r^3} + \frac{\beta_1}{r}\right) \sin\varphi \\
&- \left(2 d_1 r - \frac{2\gamma_1}{r^3} + \frac{\delta_1}{r}\right) \cos\varphi \\
&+ \sum_{n=2}^{\infty} \left[n(n-1)\cdot a_n \cdot r^{n-2} + n(n+1)\cdot b_n \cdot r^n \right. \\
&\qquad \left. - n(n+1)\cdot \alpha_n \cdot r^{-n-2} - n(n-1)\cdot \beta_n \cdot r^{-n}\right] \sin n\varphi \\
&- \sum_{n=2}^{\infty} \left[n(n-1)\cdot c_n \cdot r^{n-2} + n(n+1)\, d_n \cdot r^n \right. \\
&\qquad \left. - n(n+1)\cdot \gamma_n \cdot r^{-n-2} - n(n-1)\cdot \delta_n \cdot r^{-n}\right] \cos n\varphi
\end{aligned}
\tag{8}
$$

Knowing the general laws governing stresses and displacements, the question becomes one of determining the constants with the aid of the marginal conditions, by equating the corresponding values of σ_r from Equation (6) and σ'_r from Equation (1), that is to say, by identifying the trigonometrical terms which correspond to these coefficients.

As regards the radial edges, only conditions governing the displacement of the extreme fibres can be laid down. When the arch is rigidly fixed the condition obtains that the extreme fibres undergo no displacement. With elastic fixation the displacements of the extreme fibres of the arch must have the same values as the corresponding points in the abutment, which is loaded by normal and shear stresses in the cross section where the arch is built-in. This question has been considered in greater detail in the publication by the writer already cited. With a view to simplifying the method of calculation the radial and tangential



$$h_A = u_A \cdot \sin\alpha + v_A \cdot \cos\alpha$$

$$k_A = v_A \cdot \sin\alpha - u_A \cdot \cos\alpha$$

$$\vartheta = \frac{v_B - v_A}{b - a}$$

Fig. 3.

displacements for the corner points A and B of the arch, the calculated increase in length h_A of the inner chord of the arch, and the rotation ϑ suffered by the cross section at the springing, are all shown in Fig. 3.

The theoretical studies were checked by measurements on celluloid models.

Fig. 4 shows the appearance of an elastically built-in arch dam which is loaded by radial pressures on the outside face.

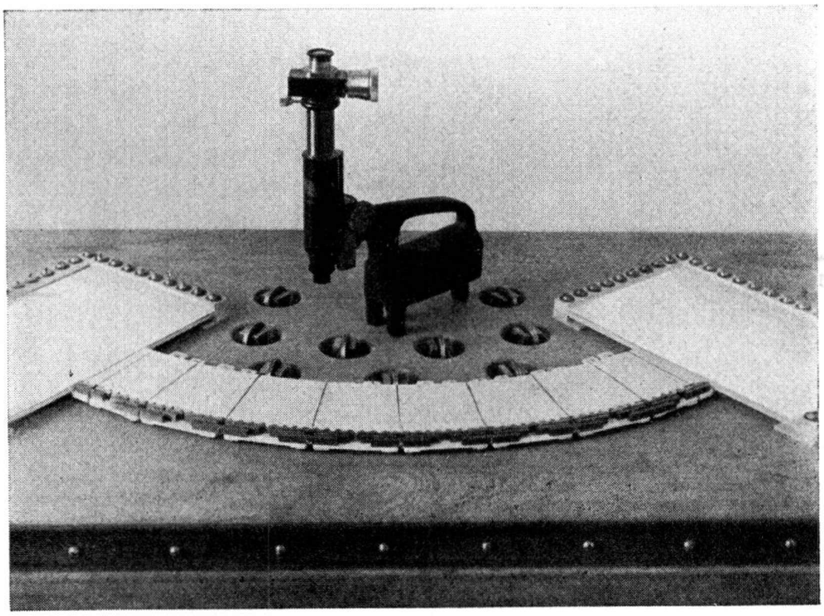


Fig. 4.

In Fig. 5 are shown the displacements of the periphery of the circular arc as measured with the aid of a microscope, and also the amounts of the dis-

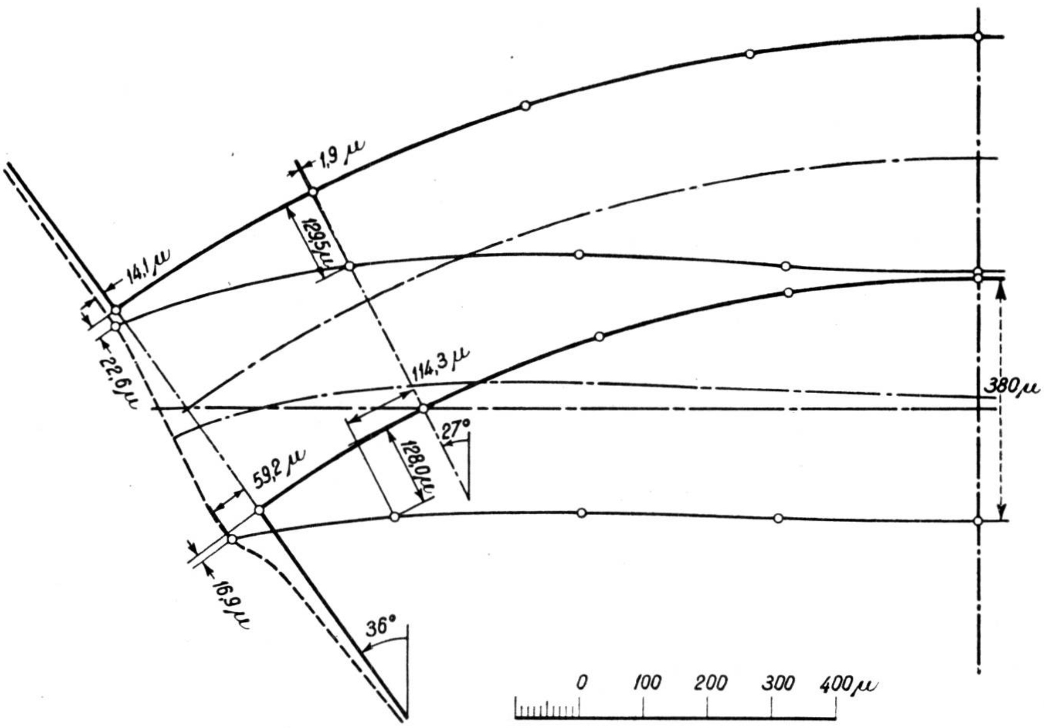


Fig. 5.

placements measured in the two sections $\varphi = 36^0$ and $\varphi = 27^0$. If the drop in the crown at the inner edge be calculated, as for instance by regarding the displacements of the extreme fibre of the section $\varphi = 27^0$ as abutment displacements in respect of the elastically stressed portion of the arch at this

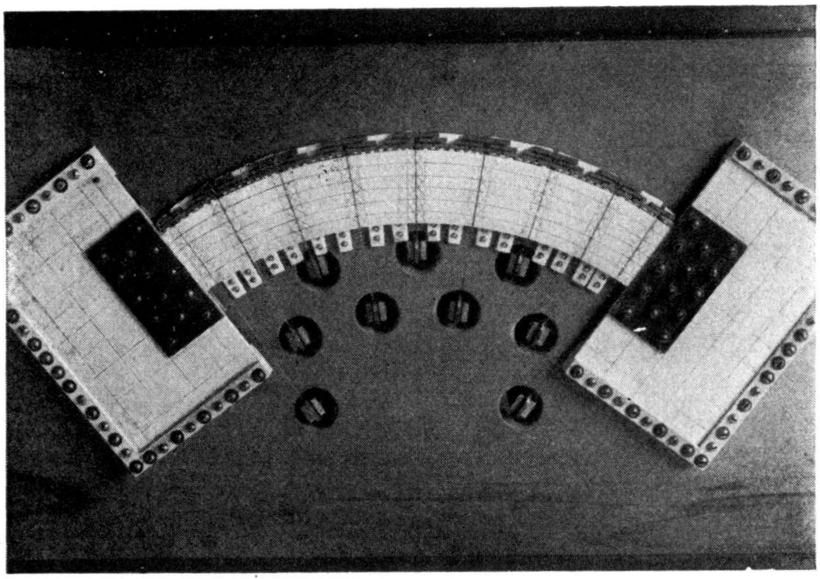


Fig. 6.

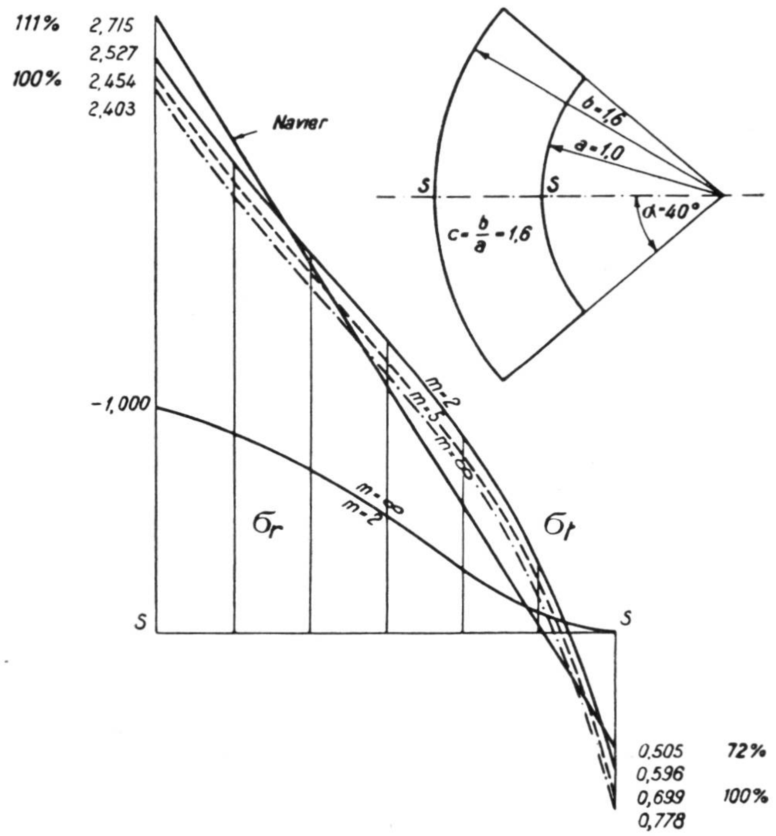


Fig. 7.

radial section; it will be seen that the measured values are only about one-third of one per cent. greater than those calculated; the agreement, therefore, is close enough to allow of recognising the theoretical bases of the problem.

If the drop at the crown on the inner edge is calculated by reference to the measured amount of displacements at the corner points of the abutment section, the result so obtained differs by approximately 4 % from the measured amount. Fig. 7 shows the model of the rigidly fixed arch. If it is assumed in this case that the law of stress and strain holds good as far as the portion which is built in, then the drop at the crown, as calculated, works out some 15 % lower than as measured. The greater amounts of deformation which in fact arise in the neighbourhood of the built-in cross section are the result of the concentration of stress which exists on the side exposed to the air. The investigations[2] in this direction which have hitherto been made are based on the assumption of rigid restraints at the ends.

By reference to an example of an arch dam subject to water pressure, the stress diagram according to the accurate theory will be compared with that obtained by means of the *Navier* approximation, which has hitherto been the only method in use for examining elastically built-in aches. In Fig. 7 there may also be recognised the influence of the *Poisson* number m for the transverse contraction of the stress values. The approximate solution gives values which are approximately 28 % too low for the tensile stresses at the crown assuming a *Poisson* ratio of $m = 5$ in concrete.

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[2] *M. Caquot:* Annales des Ponts et Chaussées, 1926, IV, July-August, p. 21. *R. Chambaud:* Génie Civil, 1926 (vols. 99 and 100).

VI 6

Thick-Walled Reinforced Concrete Pipes: Proposals for Increasing their Statical Efficiency.

Dickwandige Eisenbetonleitungen. Vorschläge zur Verbesserung ihres statischen Wirkungsgrades.

Tuyaux de béton armé à parois épaisses. Propositions en vue d'améliorer leur rendement statique.

Dr. Ing. Dr. techn. W. Olszak,
Zivilingenieur, Katowice (Pologne).

It is a characteristic of thick walled structures loaded as in Fig. 1 by normal forces uniformally distributed over the circumference that (whether reinforcement is provided or not) only very ineffective use is made of the material. This fact is made clearly apparent by the well known formulae of Lamé in which the conditions of stress arising in thick walled structures of this kind are defined on the basis of their elastic, isotropic behaviour. If the cross section of a thick pipe line, or of a tunnel shaft or other mining structure analogous thereto is designed by keeping the maximum stresses below certain values which are looked upon as "permissible" — the method which continues to serve the basis of most of the official rules for statical calculations — then it is straightaway obvious that long before the remainder of the cross sectional area is stressed up to its permissible limit the concentration of stress at the inner face will bring the tangential stress σ'_t up to its limit σ_{zul} (see Fig. 2).[1]

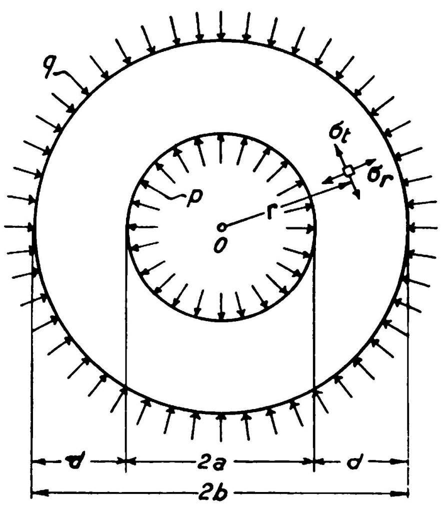

Fig. 1.
General arrangement.

If, on the other hand, the design is based on the observance not of permissible stresses but on permissible strain — a procedure which is preferable to the first mentioned on statical grounds — then the conditions which arise in the case here contemplated may in some circumstances be still more unfavourable. The magnitude of the strain σ'_{red} (see Fig. 2) in structures subject to an internal hydrostatic pressure p follows an even more steeply rising curve than does the distribution of peripheral stresses σ'_t, as is at

[1] Here, and frequently in what follows below, the radial stresses σ'_r are ignored as being of smaller importance.

once apparent on plotting the radial stresses σ'_r. The heavy concentration of strain $\sigma'_{red\,max}$ which thus occurs on the inner face is not only very undesirable as regards the statical stability of the structure but also (especially in the case of pressure pipe lines) very dangerous if faultless operation is to be ensured, since it is precisely at this point that cracks and other defects are apt to originate in the event of casual overloading produced by water-shocks.

These conditions obtain even in unreinforced pipes and in similar thick walled concrete structures which may properly be treated as isotropic, displaying the same elastic behaviour in all directions so that the Lamé formulae, already

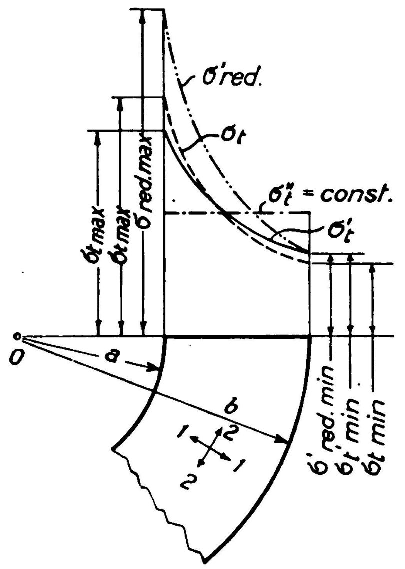

Fig. 2.

Distribution of tangential (circumferential) stresses.

——— σ'_t in material of isotropic texture ($E_1 = E_2$).

- - - - σ_t in material of (polar or cylindrical) orthotropic texture ($E_2 > E_1$).

-·-·- σ''_t constant, with improved construction (non-homogeneous orthotropic texture of material). (E_2 variable.)

-··-··- σ'_{red} strain in material of isotropic texture.

mentioned, can be directly applied. When the cross section is strengthened by the provision of steel reinforcement the result becomes even more objectionable; such reinforcement is usually provided, to make sure of coping with any tensile stresses liable to arise in pipe lines, and it becomes indispensable when either the internal pressure p or the external pressure q attains any considerable value, for the thickness of the wall cannot on purely practical grounds be increased beyond certain limits, where this would make the structural elements clumsy and difficult to handle. Moreover, the strength effect does not increase in direct proportion to the thickness of the wall; the rate of increase of strength (in relation to the diameter of the pipe) becomes slower as the thickness becomes greater. The necessary increase in resistance against internal or external normal pressures would be proportional to the thickness of the pipe only if the distribution of the stresses were such that the average circumferential stresses σ''_t assumed to exist over the cross sections were the same for any thickness ($\alpha = \frac{a}{b}$ = any chosen value, using the notation shown in Fig. 1); or, assuming a constant value of $\sigma'_{t\,max}$, roughly $\sigma'_{t\,max} = \sigma_{zul}$.

We know, however, that in fact σ''_t rapidly decreases as the thickness of the wall increases, and in order to determine this numerically it is sufficient to relate the circumferential stress assumed te be evenly distributed over the cross section to the maximum stress arising therein.

$$\sigma''_t = \eta' \sigma'_{t\,max}. \tag{1}$$

For isotropic thick walled pipes

$$\eta' = \frac{\sigma''_t}{\sigma'_{t\,max}} = \left(\frac{ap}{b-a}\right) : \left(\frac{a^2+b^2}{b^2-a^2}\,p\right) = \alpha\,\frac{1+\alpha}{1+\alpha^2}, \qquad (2)$$

when for simplicity the internal normal pressure p is taken as the only loading, so that $q = 0$.

The "coefficient of utilisation" η' — or "statical efficiency" of the structure as this characteristic figure will be called in what follows below — may be found from Table 1 in relation to different ratios of $\alpha = \frac{a}{b}$:

Table I.

α	0.0	0.2	0.4	0.5	0.6	0.8	1.0
η'	0.00	0.23	0.48	0.60	0.71	0.88	1.00

This table shows that as the thickness of the wall increases the material is less and less effectively utilised, and consequently the structure becomes less and less economical. Even with a quite moderate thickness of wall the material is very wastefully employed.

It should also be noticed that as soon as $p > \sigma_{zul}$ $\left(\text{or } q > \frac{\sigma_{zul}}{2}\right)$ an unreinforced form of construction becomes altogether impossible, because even if the cross section were infinitely thick $(\alpha = \frac{a}{b} = 0)$ the maximum stress would never fall below the value $\sigma'_{t\,max} = p$ (or $\sigma'_{t\,max} = -2q$).

In practice, therefore, recourse is had to the proved expedient of steel reinforcement, divided between several (usually two) concentric rings. In this way the tensile or compressive stress in the concrete is reduced approximately in the proportion $\frac{100}{100 + nF_2}$ wherein F_2 represents the percentage of circumferential reinforcement and $n = \frac{E_s}{E_c}$ the ratio between Young's elastic moduli for steel and concrete respectively. In this way, however, the elasticity of the structural element around the periphery is reduced; that is to say it becomes unisotropic, and behaves in a way directly opposite to that previously explained, so that the lack of uniformity in distribution of stress through the thickness of the wall which is so undesirable an occurrence becomes further emphasised, or in other words the σ_t line (see Fig. 2) becomes still steeper, and the dangerous concentration of stress on the inner face of the pipe partly counteracts the favourable effect of the reinforcement provided.

There is no particular difficulty in representing this effect mathematically. The unisotropic character of the compound structure due to the reinforcing steel is expressible by saying that in thick walled pipes and in similar reinforced concrete structures the elastic behaviour differs along each of three principal directions which are respectively given by (1) the direction of the radius vector r (percentage of reinforcement F_1, elastic constants E_1 and m_1) (2); the tangent t to the concentric circles drawn with the axis of the pipe as origin F_2, E_2, m_2; and (3) the longitudinal deformation z of the cylindrical structure (F_3, E_3, m_3). Hence, by contrast with the rectilinear orthogonal anisotropy (or "orthotropy" as it is sometimes called), we now have a condition of curvilinear anisotropy for which the author proposes the designation *"polar orthotropy"* in the case of thin slices or *"cylindrical orthotropy"* in that of pipes or cylinders of finite or infinite length.[2]

Under certain conditions the moduli of elasticity in these three directions (E_1, E_2, E_3) may differ greatly from one another, and the same is true of the Poisson's ratios (or, more properly, the coefficents of transverse deformation m_1, m_2, m_3). These constants are, however, not independent of one another, for the following simple relationship exists between them:

$$m_1 E_1 = m_2 E_2 = m_3 E_3 \tag{3}$$

The value of this constant product will be denoted henceforth by M. The shear moduli G are without significance for the present argument.

The different values of the elastic constants along the three principal directions are influenced by the method of manufacturing the pipe (as, for instance, by the centrifugal process), but they are primarily due to the differing percentages of steel reinforcement in each direction.

If the percentage of reinforcement in one of the directions i ($i = 1, 2, 3$) is denoted by F_i, then the modulus of elasticity of the compound body along this direction may be made to agree with the formula of Professor *M. T. Huber*[3]:

$$E_i = E_b \lambda_i \quad \text{where } \lambda_i = 1 + (n - 1)\frac{F_i}{100} \tag{4}$$

using the ratio $n = E_s/E_c$ as already defined. It is true that this assumes the substitution of the unstable unisotropic compound body by an ideal stable orthotropic model — a simplification which is necessary for the clear understanding and numerical expression of what follows below, and is the more justifiable the greater the closeness of the structural element (steel reinforcement) in proportion to the remaining dimensions. Later, when applying the results obtained by this method in practice account must be taken of this fact.[4]

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[2] It was originally intended to discuss the whole complex of problems of polar-orthotropic slices and cylindrical-orthotropic pipes in the present paper, but the limitations later imposed upon the length of individual contributions led the author to relegate the more exhaustive treatment of these to a publication of his own, „Beiträge zur Statik von polarorthotropen Scheiben und zylinderorthotropen Rohren", in Der Bauingenieur, 1936, N° 31/32. In the present paper frequent reference will be made to this using the abbreviation WO 17.

[3] *M. T. Huber:* Probleme der Statik technisch wichtiger orthotroper Platten. (Statical problems of orthotropical shells of technical importance.) Academy of Technical Sciences, Warsaw, 1929 (in German), p. 13.

[4] An accurate account of this discontinuity, in terms of elastic theory, would be possible only by abandoning the simpler case of a homogeneous anisotropic model and considering the problem spatially (with axial symmetry) as that of a non-homogeneous complex of two isotropic components, somewhat after the manner that Dr. Ing. *A. Freudenthal* has dealt with hooped columns („Verbundstützen für hohe Lasten", Columns of compound nature and heavy loads, Berlin, 1933). To anyone sufficiently interested who is not afraid of the time and trouble involved this offers the certainty of a profitable and illuminating task.

The determination of the transverse extension coefficient m_i ($i = 1, 2, 3$) is more difficult because only a very limited amount of experimental data are at present available on this matter. The determination of these values (which, as will presently appear, is necessary in order to arrive at the conditions of stress and strain arising in the anisotropic compound construction) will be explained by the writer elsewhere (WO 17).

In a cylindrical structure subject to uniform internal or external pressure the conditions of strain and stress do not depend upon the co-ordinate z, in other words it is a case of the "plane problem" in elastic theory. As a result of the circular symmetry here assumed, and of the geometrical form of the cross sections and external loads, the conditions of strain and stress are also independent of the central angle φ, so that the whole field of stress is influenced only by a single variable, namely the radius vector r. Instead of partial differentiations, total differential quotients will arise.

Equal emphasis should be laid on the remarkable fact that in an unisotropic structure of material the uniplanar condition of *strain* (deformation) and the uniplanar condition of stress are not interchangeable, this being directly contrary to what is true of isotropic materials. In the latter case, as is well known, it is frequently of no importance whether a piece be cut out of the inside of an (indefinitely) long cylinder so that the restraint of the portions adjoining its front and back surfaces prevent it from deforming except internally, or whether a similar slice be taken from the free end of a cylinder so that it possesses complete freedom of deformation. In these two cases the conditions of stress in the coordinated planes of an isotropic slice are identical (apart from considerations into which the constants of the material enter). In isotropic media these two cases can always be handled in the same way.[5]

This is not true for an anisotropic structure. Here not only is the distinction between a uniplanar condition of strain and a uniplanar condition of stress essential in principle, but the fields of stress deviate *effectively* from one another. This difference, numerically considered, can however be ignored in practical engineering, because in the group of equations for radial and tangential stresses σ_r and σ_t the difference is given by the "structural figures" s and t which differ only slightly from one another (as will be shown below). The plane anisotropic problem must further be divided into the study of an annular disc of an infinitely long pipe and that of infinitely long but developed pipe. The numerical solution to these problems must also be separately approached.

There is insufficient space here to work out this procedure of calculation for all the possible variants and cases that can arise, and reference must be made to the work "WO 17" cited in footnote,[2] but the line of thought to be followed in dealing with the simplest case, namely a circular annular slice of limited thickness (or depth) may be indicated here briefly to serve as a guide towards the final result.

[5] See, for instance, the author's paper „Der ebene Formänderungs- und Spannungszustand der Elastizitätstheorie" (The plane stress and strain conditions of the elastic theory) in Zeitschrift des Österr. Ingenieur- und Architektenvereines, 1936, N° 15/16, where it is shown that in an isotropic structure both the limiting cases mentioned above will *always* admit of a formal treatment in common, even where there is a complex of interrelated zones, or where displacements (instead of stresses) are laid down as the limiting conditions.

The starting point is found in the equation of equilibrium for an annular element bounded by two neighbouring radial sections and by two neighbouring concentric circles within the annulus:

$$\sigma_t = \frac{d}{dr}[r\sigma_r] \tag{5}$$

Since, in any case likely to arise in practice, the annular slice will be reinforced with steel mainly in the direction of the periphery, the modulus of elasticity E_2 in that direction will always be greater than the modulus of elasticity E_1 measured in the direction of the radius vector r, that is $E_2 > E_1$, use being made of Equation (4) for calculating E_2. (If, however, the conditions differ from these, as for instance in the case of a fly-wheel, where radial reinforcement may have to be taken in account, the line of thought will still be the same in principle though the quantitative results will be different.) The influence of special methods of making and placing concrete (such as the use of vibration or centrifugal action) on the ratio $\frac{E_2}{E_1}$ must for the present be left out of account in the absence of any experimental data on the subject.

Taking account of the difference in the elastic properties of the material in the two opposed curvilinear principal directions at right angles to one another, "1" and "2", the fundamental relationship between the deformation and stress components to be considered here, which result in the statical indeterminacy of the elastic problem for any continuous medium, may be written as follows,

$$\left.\begin{aligned} \varepsilon_r &= \frac{du}{dr} = +\frac{1}{E_1}\sigma_r - \frac{1}{m_2}\frac{1}{E_2}\sigma_t, \\ \varepsilon_t &= \frac{u}{r} = -\frac{1}{m_1}\frac{1}{E_1}\sigma_r + \frac{1}{E_2}\sigma_t, \end{aligned}\right\} \tag{6}$$

wherein u represents the increase in radius.

It will be noticed that here the stress component σ_z is completely eliminated ($\sigma_z = 0$) this being characteristic of such a case of the uniplanar stress condition as has just been dealt with (and also of what many authors[6] call the quasi-planar stress condition) when the plate (or slice) is of small thickness (depth). Actually the thinner the disc in proportion to its other dimensions the more nearly will this condition be approached. It should also be noticed that all the stress values (σ, τ) and also the strain deformation components (ε) are averages taken through the thickness of the disc.

The solution to the group of equations (6) for the stress component leads to the following stress-strain equations:

$$\left.\begin{aligned} \sigma_r &= \frac{M}{m_1 m_2 - 1}\left[\frac{u}{r} + m_2\frac{du}{dr}\right], \\ \sigma_t &= \frac{M}{m_1 m_2 - 1}\left[m_1\frac{u}{r} + \frac{du}{dr}\right]. \end{aligned}\right\} \tag{7}$$

[6] See, for instance, *H. Reißner* and *F. Strauch:* Ringplatte und Augenstab (Annular slab and eye-bars). Ing.-Archiv, 1933, p. 483.

There remains only one further step, namely that of substituting the values obtained from (7) in the equations of equilibrium (5), giving rise to the differential equation

$$\frac{d^2u}{dr^2}+\frac{1}{r}\frac{du}{dr}-\frac{m_1}{m_2}\frac{u}{r^2}=0 \tag{8}$$

from which the radial displacement u is obtainable. The integral of this

$$u=Ar^s+Br^{-s}, \tag{9}$$

wherein

$$s=\sqrt{\frac{m_1}{m_2}}=\sqrt{\frac{E_2}{E_1}} \tag{10}$$

must be made to comply with the marginal conditions prescribed:

$$\sigma_r=\begin{cases}-p\\-q\end{cases}\quad \text{for } r=\begin{cases}a\\b\end{cases} \tag{11}$$

After a somewhat lengthy process of calculation and the introduction of the unnamed (dimensionless) "reduced" radial co-ordinates $\rho=\frac{r}{b}$ the desired stress values are finally obtained in the form[7]:

$$\left.\begin{aligned}\sigma_r&=\frac{1}{1-\alpha^{2s}}\left\{\rho^{s-1}[p\alpha^{s+1}-q]-\left(\frac{\alpha}{\rho}\right)^{s+1}[p-q\alpha^{s-1}]\right\},\\ \sigma_t&=\frac{s}{1-\alpha^{2s}}\left\{\rho^{s-1}[p\alpha^{s+1}-q]+\left(\frac{\alpha}{\rho}\right)^{s+1}[p-q\alpha^{s-1}]\right\},\end{aligned}\right\} \tag{12}$$

wherein α denotes the ratio of internal to external diameter (as already used), $\alpha=\frac{a}{b}$, which definitely fixes the form of cross section.

It may easily be shown that by putting $E_1=E_2$ (so that $s=1$) the transition to the Lamé formulae which hold good of isotropic bodies is obtained, this being a special case of the argument given here.

The case, which is of special interest here, of a uniformly distributed internal pressure p (simultaneously with $q=0$) imposed alone, leads to the expressions

$$\left.\begin{aligned}\sigma_r&=\frac{1}{1-\alpha^{2s}}\left(\frac{\alpha}{\rho}\right)^{s+1}[\rho^{2s}-1],\\ \sigma_t&=\frac{s}{1-\alpha^{2s}}\left(\frac{\alpha}{\rho}\right)^{s+1}[\rho^{2s}+1],\end{aligned}\right\} \tag{13}$$

which, when $s=1$ (and therefore $E_1=E_2$), also of course lead to the generally well known formulae for thick-walled isotropic pipes.

The extreme fibre stresses of the circumferential stresses may be taken as especially characteristic values of the stress components derived above. For the

[7] The same results may be confirmed in a rather different way by introducing what is called the "stress function", out of which the stress components are obtained by derivation.

inner face where $r = a$ we obtain with $\rho = \alpha$ the maximum stress that can arise anywhere in the cross section, amounting to

$$\sigma_{t,r=a} = s\frac{1+\alpha^{2s}}{1-\alpha^{2s}}\,p, \qquad \left(\sigma'_{t,r=a} = \frac{1+\alpha^2}{1-\alpha^2}\,p\right); \tag{14}$$

which is always positive, and therefore tensile. For the outermost fibre $r = b$, and with $\rho = 1$ we obtain

$$\sigma_{t,r=b} = 2s\frac{\alpha^{s+1}}{1-\alpha^{2s}}\,p, \qquad \left(\sigma'_{t,r=b} = 2\frac{\alpha^2}{1-\alpha^2}\,p\right). \tag{15}$$

If the results obtained in this way are compared with those for the extreme fibre stresses existing in an isotropic form of body[8] (as given here in brackets on the corresponding lines in order to facilitate comparison) it will be seen that in the reinforced (and, therefore, orthotropic) pipe the outer fibres and the material close to them are relieved of load relatively to the unreinforced form of construction ($\sigma_{t,r=b} < \sigma'_{t,r=b}$). The result is that the concentration of stress existing at the inner face — already dangerous with the kind of anisotropy here considered ($E_2 > E_1$; $s > 1$), which is the very one that most often arises in practice — is thereby considerably worsened ($\sigma_{t,r=a} > \sigma'_{t,r=a}$).

This is true of the annular slice of limited thickness (depth) which we have just examined, that is to say for the case of uniplanar *stress* conditions. When the case of a pipe of finite but great length ($E_z = k = \text{const.} \neq 0$) or infinite ($E_z = k = 0$) length is considered, the conditions become even more unfavourable. Without entering on the mathematical consideration of the conditions of stress and strain arising in these latter cases (which for lack of space must be relegated to sections III and IV of the work "WO 17" cited in the footnote[2]) the results for these two cases may be indicated here. By a process of reasoning similar to that indicated above, the remarkable fact is established that the stress components σ_r and σ_t are exactly the same in form as those determined by the group of equations (12), with the sole difference that the term s which occurs as a factor and exponent, instead of being evaluated from (10) now has the value

$$t = \sqrt{\frac{m_1 m_3 - 1}{m_2 m_3 - 1}} \tag{16}$$

Instead of the characteristic $\sigma_z = 0$, which arose above, we now have the function

$$\sigma_r = \frac{1}{m_3}(\sigma_r + \sigma_t + kM) \tag{17}$$

which determines the stresses acting in the direction of the length of the cylinder; stresses which now cause purely plane deformations in the individual lamellae.

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[8] The un-dashed values of functions relate to orthotropic structure while those with one dash refer to isotropic structure, in order to distinguish the results. The double dash used already in Equations (1) and (2), and often in what follows below, relates to the ideal case G where the circumferential stresses are uniformly distributed over each radial section, and to the characteristic values of this.

In engineering practice the conditions indicated by

$$t \geqq s \geqq 1 \tag{18}$$

always obtain, and it will at once be apparent that in the case of cylindrical elements of structure (of whatever length) — that is to say in the case where plane *deformations* occur — the absence of uniformity in the distribution of stress becomes even more marked (even if not appreciable in practice) than in the case of a thin annular slice — namely in the case of a plane *stress* condition.[9]

Tables II and III contain a numerical summary of the results indicated above. Table II gives the determining values s and t for polar and cylindrical orthotropic texture wherein it is assumed that $n = \frac{E_s}{E_c} = 10$. (As regards the numerical

Table II.

F_2 en %	λ_2	s	t
0	1.00	1.000	1.000
1	1.09	1.043	1.044
2	1.18	1.086	1.088
3	1.27	1.127	1.130
4	1.36	1.166	1.170
5	1.45	1.204	1.210
7	1.63	1.277	1.286
10	1.90	1.378	1.393

Table III.

s, t		1.00				1.20				1.50				G	
α	ρ	S′	D%	A%	U%	S	D%	A%	U%	S	D%	A%	U%	S″	U%
0.00	α	1.00	±0	+ ∞	∞	1.20	+ 20	+ ∞	∞	1.50	+ 50	+ ∞	∞	0.00	±0
	1	0.00		± 0		0.00	± 0	± 0		0.00	± 0	± 0			
0.25	α	1.13	±0	+ 239	300	1.29	+ 14	+ 287	351	1.55	+ 37	+ 365	435	0.33	±0
	1	0.13		− 61		0.12	− 8	− 64		0.10	− 23	− 70			
0.50	α	1.67	±0	+ 67	100	1.76	+ 5	+ 76	112	1.93	+ 16	+ 93	133	1.00	±0
	1	0.67		− 33		0.64	− 4	− 36		0.60	− 10	− 40			
0.75	α	3.57	±0	+ 19	33	3.62	+ 1	+ 21	36	3.69	+ 3	+ 23	39	3.00	±0
	1	2.57		− 14		2.56	− 1	− 15		2.53	− 2	− 16			
0.90	α	9.54	±0	+ 6	11	9.56	+ 0.2	+ 6	11	9.58	+ 0.4	+ 7	12	9.00	±0
	1	8.54		− 5		8.54	~ 0.0	− 5		8.51	− 0.4	− 5			
1.00	α	∞	±0	±0	0	∞	±0	±0	0	∞	±0	±0	0	∞	±0
	1														

[9] The special cases where, exceptionally, the signs of equality hold good in (18) are further discussed in WO 17. It is apparent in any case that the values s or t (which would be unity in a material of isotropic structure) may be regarded as a measure of the extent of deviation from isotropy; they are, as it were, characteristic structural figures.

values of m_1, m_2, m_3 comparison should be made with "WO 17"). It is evident that the difference between s and t is inappreciable *in practice* (though, as already explained, it is essential to distinguish between the two values *in principle*).

The functional relationships indicated in Table III are more important, and this summary enables the stress values in isotropic texture ($S' = \sigma_t/_p$) to be compared with those in the orthotropic texture ($S = \sigma_t/_p$) contemplated here. The last two columns enable consideration of the case to which we are now leading up, characterised by an entirely uniform distribution of the circumferential stresses

$$S'' = \sigma''_t/p = \frac{a}{b - a} = \frac{\alpha}{1 - \alpha}$$

over each radial section. It is assumed that only an internal hydrostatic pressure p occurs, so that $q = 0$.

The upper figures in Table III give the stress values at the inner face ($\rho = \alpha$) and the lower figures those at the outermost fibre ($\rho = 1$), for differently proportioned cross sections ($\alpha = \frac{a}{b} = 0, \, ^1/_4, \, ^1/_2, \, ^3/_4, \, ^9/_{10}, \, 1$). The columns D give the percentage differences from the values for isotropic material as calculated by means of Lamé's equations, and the columns A give the positive (+) or negative (—) percentage deviations from the ideal case of a completely uniform distribution of stress S″. The algebraic sum of these deviations A serves as a measure of the lack of uniformity, U, which can be read off in the appropriate columns and which affords a good indication of how unfavourable the stress is distributed in all thick walled structures constructed in the way hitherto customary. This undesirable effect becomes particularly bad when the thickness of wall is increased (that is to say when α decreases) or when heavy reinforcement is used (that is to say when s or t become large).

As the thickness of the wall and the heaviness of the reinforcement increases, the flow of stress becomes more and more concentrated within an ever narrower inner zone, the internal stresses being forced more and more towards the inside face and the outer portions of the cross section being correspondingly withdrawn from participation in the stress.

In the limiting case where the reinforcement is very strong and inelastic $\left(\frac{E_2}{E_1} \longrightarrow \infty\right)$ only the innermost fibre would be left to perform the whole of the statical function by offering resistance under an infinite amount of stress.

It will be seen from the columns D how great is the error involved in calculating orthotropic compound structures by the simple Lamé formulae and in designing them accordingly, as has hitherto always been the practice. In structures intended for special purposes, as for instance the reinforced concrete pipe described by the present author for hydraulic erosion mining[10] (using an operating pressure equal to 20 or more atmospheres) in which the thickness of the wall has necessarily to be great and the reinforcement heavy, the error on this

[10] *W. Olszak:* Eisenbetonrohre für Spülversatzzwecke (R.C. pipes for erosion mining). Zement, 1935, Nos 14, 15, 16.

account may be as much as 15, 20, 30 or more per cent. In such cases simplified assumptions no longer hold good and it is not permissible to make use of them in the calculations, especially if this is done (as here) at the expense of safety against breakage and cracking.

The statical efficiencies η of reinforced pipelines and of similar reinforced concrete structures are notably smaller than that of unreinforced pipes. The values of η are always smaller than the values of η' indicated in Table I.

The foregoing considerations suggested the idea of trying to construct thick walled pipes (and other cylindrical structures similarly loaded), in which the uneven distribution of stress with its attendant evils (poor utilisation of the material, increased risk of cracking and fracture starting from the inner face, unavoidable wastage of material, etc.) might be eliminated.

It is, in fact, possible by a simple method to impose beforehand on the compound construction an elastic behaviour whereby the desired purpose is completely attained in practice, and in this way to substitute for the accepted method of building thick walled structures a new method which is superior in every respect.

At first sight the proper thing to do might seem to be to make the reinforcement heavier close to the inside face where the flow of forces is concentrated, but this would be altogether wrong. It is true, indeed, that the increase in the maximum stress at the inner face implies a corresponding relief of stress in the outer portions of the thick walled pipe, and that the increased steepness of the line of distribution of the circumferential stresses has the effect of displacing the centre of gravity of the stress diagram inwards; moreover, as pointed out above, this phenomenon becomes particularly marked when the reinforcement is especially heavy and the wall especially thick.

But to attempt to combat this undesirable disposition of the stress by altering the uniform arrangement of the reinforcement (making it heavier towards the inside of the annulus), so that the centre of gravity of the steel shall approximately coincide with that of the stress diagram, is merely to encourage still further the concentration of stress at the *inner* face of the pipe, and to provoke a further increase in a maximum stress which is already harmful: — for it is a characteristic of any statically indeterminate system to act in such a way that its "stronger" (less yielding) portions tend to participate more fully in the internal flow of stress.

This circumstance suggests a directly contrary procedure to the above: namely that of increasing the heaviness of the steel reinforcement *outward* and of thereby gradually increasing the modulus of elasticity E_2 towards the outer portion of the cross section, so as to cause the latter to participate more fully in carrying the stress, and, in the ideal case, ensure that every fibre of the pipe shall be uniformly strained (and in the case of an internal hydrostatic pressure p shall carry a uniform tensile stress).

The usual treatment of the problem is, therefore, completely reversed. Hitherto we have sought to define the statical values of stress and strain in accordance with the known elastic properties of compound bodies; now we lay down *a priori* a certain condition of stress and seek to define elastic properties, not yet known, which shall correspond with the desired distribution of stress.

In reinforced concrete structures it is relatively easy to vary the elastic properties of the compound body at will, for one is dealing with two structural components — concrete and steel — which differ greatly from one another in this respect, and by suitable adjustment of the effective cross sections of these two components it is possible in a simple way to attain the desired objective, namely the improved construction of the compounded material.

Once again it is impossible to go into details of the calculation — more especially as there is more than one way of reaching the goal, for the solution depends less on the absolute elastic values than on their mutual relationship, that is to say on varying ratio $\frac{E_2}{E_1}$. Here either the value E_2 may be altered in relation to the fixed value E_1 or the desired relationship may be obtained by varying E_1 while E_2 is kept constant, or again by varying both values in opposite directions. The case previously considered of a (homogeneous) polar or cylindrical orthotropy will now be replaced by that of a curvilinear but movable and therefore non-homogeneous condition of orthotropy.

Further details will be given in a paper now ready and shortly to be published, which will be cited below by the abbreviation "WO 20".[11] Here reference will be made only to the results obtained in the simplest case of all which can be carried out in practice without any difficulty; the circumferential reinforcement is suitable increased outward while the value of E_1 is kept constant at E_b, in order to obtain the desired effect.

Starting from the fundamental requirement that the circumferential stresses are to be entirely uniform through-out every radial section, we may write

$$\sigma''_t = \frac{d}{dr}[r \cdot \sigma''_r] = \text{const.} = C = \frac{ap - bq}{b - a}. \tag{19}$$

From this, by simple integration, we obtain the law governing the radial stresses:

$$\sigma''_r = C + \frac{D}{r}, \tag{20}$$

wherein the constant of integration D has to be chosen to suit the marginal conditions (11). By putting

$$D = ab\,\frac{q - p}{b - a} \tag{21}$$

we satisfy (11) at either face.

We will now consider the relationship between the conditions of stress and those of strain. If, for the sake of simplicity, we take the case of an annular slice of limited thickness or depth (that is to say the case of uniplanar *stress* condition) we may apply the group of equations (6), provided that we now take E_2 not as a constant but as a function of the radius vector, $E_2 = E_2\,(r)$.

[11] Meanwhile published in the Polish journal Czasopismo Techniczne, 1937, N^os 1, 2, 3, 4, 5, 6.

If, further, we utilise the first of the relationships (3) (an assumption which *is strictly* fulfilled for the case of polar or cylindrical orthotropy as hitherto taken, and the validity of which still would remain to be checked in the present case, but which may provisionally be accepted as adequate) then nothing more is wanted for arriving at the desired function E_2 which remains unknown.

The result is simply

$$E_2 = E_b \cdot \lambda''_P, \tag{22}$$

wherein

$$\lambda''_P = \frac{\rho}{A_P + \rho + \frac{1}{b}\frac{D}{C}\ln\rho}. \tag{23}$$

If, on the other hand, an (infinitely) long pipe is to be considered, so that the condition of plane *strain* is used as a basis for the calculation, then the solution takes the form

$$E_2 = E_b \cdot \lambda''_R, \tag{24}$$

where

$$\lambda''_R = \frac{\rho}{A_R + \rho + \frac{1}{b}\frac{D}{C}\left(1 - \frac{1}{m_1 m_3}\right)\ln\rho}. \tag{25}$$

In order to avoid having to use an endless double series expression for determining A_P and A_R these may be arrived at from the following approximate expression which gives very good results (see WO 20):

$$A_P = -\frac{(n-1)F''_2}{100+(n-1)F''_2}\frac{1+\alpha}{2} - \frac{1}{b}\frac{D}{C}\ln\frac{1+\alpha}{2}, \tag{26}$$

$$A_R = -\frac{(n-1)F''_2}{100+(n-1)F''_2}\frac{1+\alpha}{2} - \frac{1}{b}\frac{D}{C}\left(1 - \frac{1}{m_1 m_3}\right)\ln\frac{1+\alpha}{2}. \tag{27}$$

The varying circular reinforcement f''_2 must be

$$f''_2 = \frac{100}{n-1}(\lambda'' - 1) \tag{28}$$

wherein λ'' has the value (23) or (25) according to the conditions of the problem. Of course the relationship

$$\frac{1}{b-a}\int_a^b f''_2 \cdot dr = F''_2. \tag{29}$$

must always hold good.

Naturally the function $E_2 = E_e \cdot \lambda''$ cannot actually be made smaller than E_c (that is to say λ'' cannot be less than unity), for we are only able to increase the modulus of elasticity of the compound body by suitably *increasing* the amount of the steel by comparison with E_c. We can, in other words, render the compound body "denser", but we cannot make "voids" in it. There is thus a certain practical limitation to the method, which as a guide may be put at

$\alpha = \frac{a}{b} \geqq (0.6$ to $0.8)$ according to the amount of reinforcement F''_2. But forms of construction thicker than those corresponding to the inequality just stated — i. e. with thicknesses greater than (0.4 to 0.2) b — will rarely occur in practice, and when they do occur recourse may be had to the more difficult method, that of altering E_1. In the upshot, therefore, it may be said that in all cases likely to arise in practice we can obtain the favourable uniform distribution of stresses.

As an example let us take a pressure pipe of the proportions $\alpha = \frac{a}{b} = 0.6$ capable of being influenced in the desired direction by the simpler method of suitably arranging the circumferential reinforcement.

We have

$$E_2 \cong \frac{\rho}{0{,}510 + \rho - \ln \rho},$$

$$f''_2 \cong \frac{100}{n-1}\left(\frac{\rho}{0{,}510 + \rho - \ln \rho} - 1\right).$$

As will be seen from Fig. 3, the increase in E_2 is almost rectilinear, its value at the inner face being $E_{2,\,r=a} \cong E_b$ and its maximum value, at the outer face, $E_{2,\,r=b} = 2.04\, E_b$. The line f''_2 also is almost straight, giving the corresponding values $f_{2,\,r=a} = 0$ and $f''_{2,\,r=b} = 7.4\,\%$ (for $n = 15$). The thickness of the pipe is divided into practically equal parts by the centre of gravity of the diagram, and the centre of gravity of the reinforcement comes at a distance of $^2/_3\, d = {}^2/_3\, (b - a)$ from the internal face of the pipe.

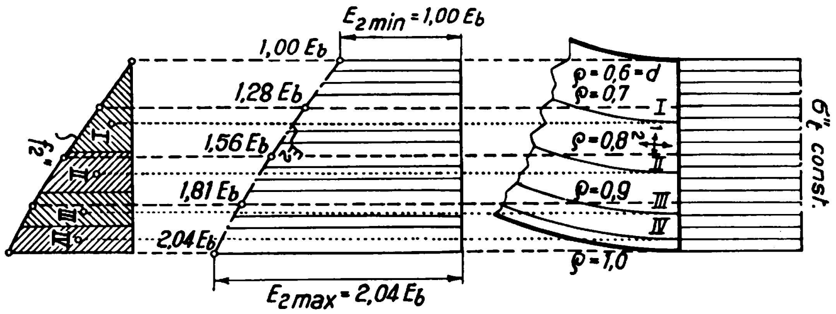

Fig. 3.

The uniformly distributed tangential stresses $\sigma''_t =$ const. are ensured by suitably adjusting the modulus E_2. This also gives the reinforcement diagram f''_2 whence the position of the reinforcing rings I, II, III, IV is determined.

The distribution of the steel reinforcement may advantageously be carried out by using a similar method to that commonly adopted in arranging the shear reinforcement in reinforced concrete beams, by reference to the shear diagram. It will be best to use as many and as thin reinforcing bars as possible in order to obtain as uniform a compounding effect as possible, and the reinforcing rings should then be placed at the centres of gravity of the corresponding portions of the areas f''_2, as illustrated in Fig. 3 for four bars of equal diameter. Further details as to the most favourable arrangement of the reinforcement *as a whole* F''_2 (29) may be found in WO 20.

It is possible now to go one step further. As is well known, the permissible stress in the compound body depends among other factors on the percentage of reinforcement, a quantity which increases outward if the improved solution here suggested is adopted. Matters may be so arranged that the permissible stress increases outward at a rate corresponding to the percentage of reinforcement, the case being analogous to that already considered, and the fundamental equation (19) beeing replaced by

$$\sigma_t''' = \sigma_{b\,zul}\left[1 + n\frac{f_2'''}{100}\right] = \sigma_{b\,zul}\,\frac{1}{n-1}\left[n\frac{E_2}{E_1} - 1\right], \tag{30}$$

wherein

$$E_2 = E_b \cdot \lambda''', \qquad f_2''' = \frac{100}{n-1}(\lambda''' - 1).$$

We will go no further into this special problem. By contrast with the procedure hitherto in question (wherein certain permissible stresses are not exceeded) it is more important to ensure that the amount of strain in the structure shall be the same at every point; that is to say to ensure that the degree of safety against breaking or cracking shall remain the same at every point in the thick walled structure. This cannot however be obtained either through the requirement (19) or through the more rigorous requirement (30), but must be derived from a special hypothesis of strain.

The considerations which are relevant here are either the *Guest-Mohr* hypothesis of shear stress or the *Huber* hypothesis of strain energy. Instead of (19) or (30) the requirement to be imposed is that the "reduced" stresses shall remain unchanged:

$$\sigma_{red}'''' = \sigma_t'''' - \sigma_r'''' = \text{const}, \tag{31}$$

or

$$\sigma_{red}'''' = \sqrt{(\sigma_r'''')^2 + (\sigma_t'''')^2 + (\sigma_z'''')^2 - \sigma_r''''\,\sigma_t'''' - \sigma_t''''\,\sigma_z'''' - \sigma_z''''\,\sigma_r''''} = \text{const}. \tag{32}$$

This, however, ensures an ideal solution from every point of view — statically (as regards safety from fracture), operationally (as regards freedom from cracking), and economically (as regards saving in material).[12] How the law $\lambda'''' = \frac{E_2}{E_1}$ reads in this case, and how the reinforcement is to be distributed (function f''''_2), cannot, for lack of space, be further discussed here and must be left for a separate later publication.

The statical efficiency η'' obtained by way of the improved constructions as here suggested is always $\eta'' = 1 = 100\,\%$ (by contrast with the rather meagre values of η and η'), quite independently of the wall thickness, and the notable saving in material indicated in Footnote 11 is straightaway realised.

[12] For comparison, the amount of material required in pipes made by the old method hitherto in use is 20, 50, 100 or more per-cent greater (according to the dimensional ratio α and the amount of reinforcement F_2) than by the improved method. See WO 19.

VI 7

The Application of Pre-Stressing in Dams.

Anwendung der Vorspannungen auf Staumauern.

Utilisation des précontraintes dans les barrages.

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In Volume I of the Publications of the I.A.B.S.E., 1932, reference was made to certain possibilities of design in the strengthening of old dams, and an account was given of work planned in Algeria based on our ideas in application to the Cheurfas dam. At that time only preliminary studies had been carried out, but since then the work has been completed with entire success and is worthy of mention at the Congress.

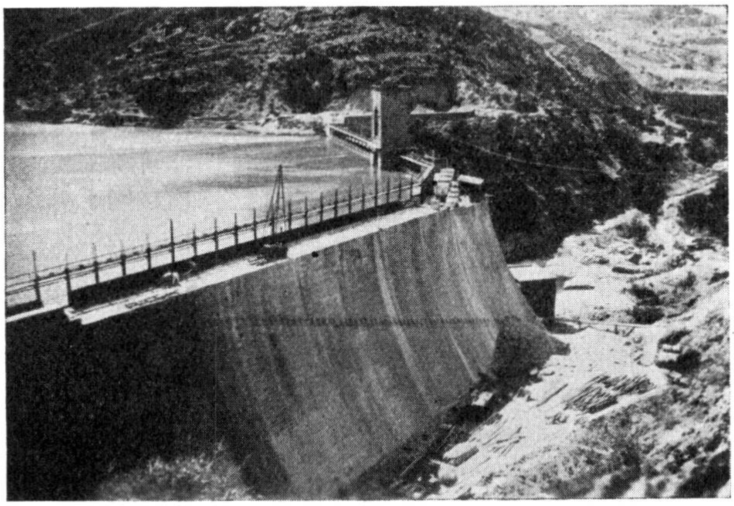


Fig. 1.
Cheurfas dam.

The Grands Cheurfas dam which is situated at Oued Mékerra 20 km above Saint-Denis du Sig in the Department of Oran, Algeria, was constructed between 1880 and 1882 with a view to the irrigation of the fertile plain of Sig, this being the name given to the Mekerra in the lower portion of its course. The structure is a gravity dam made of random masonry and has a maximum height of 30 m (Fig. 1).

It belongs to a series of French gravity dams constructed during the past century wherein the margin of safety is sometimes very low. A number of these dams have failed, notably that at Bouzey, and (to mention cases nearer to the present example) the Oued Fergoua and the Hebra dams.

It was in consequence of this last mentioned accident that the Government-General of Algeria decided to that the Cheurfas dam should be strengthened.

The procedure adopted for this purpose consists in attaching the structure to the ground below by means of tie bars, which in a sense play the part of very

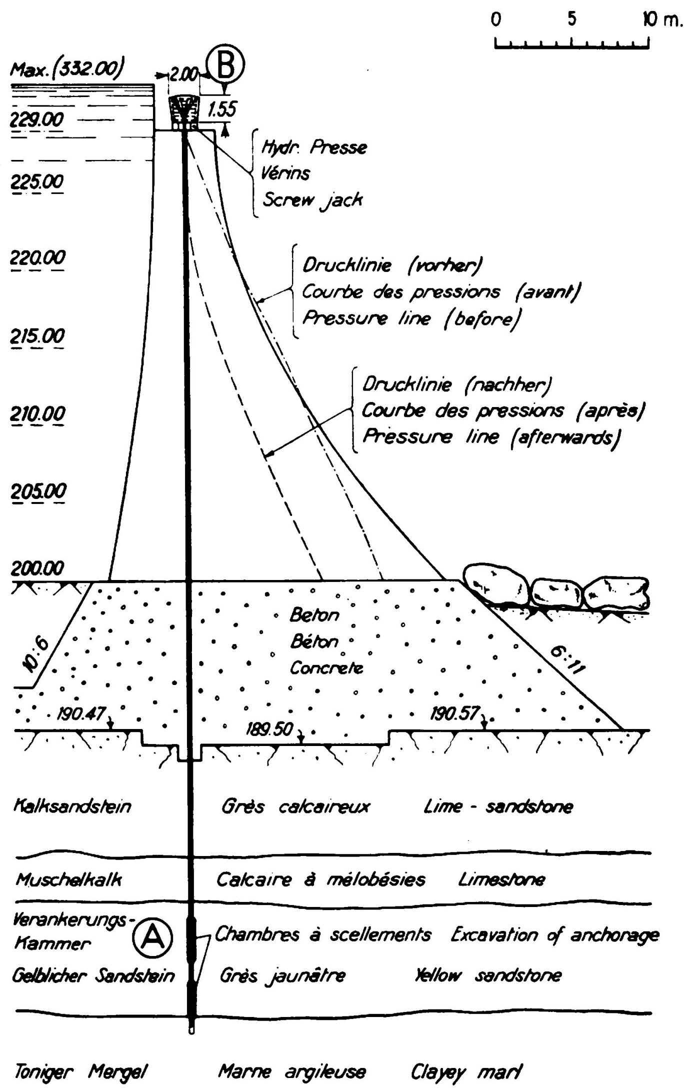


Fig. 2.

Effect of anchoring tie in changing the line of pressure.

large turnbuckles. If, for instance, such a tie be assumed to be stretched between the point A in the foundation and the point B on the crest of the dam (Fig. 2), it will be seen that by adjusting the load and the number of tie bars the line of pressure can be modified at will, and the structure may be given any desired factor of safety, or its height may be increased. (The increase provided for at Cheurfas was 3 m.) So far nothing more is involved than a new application of a very simple idea which is constantly being used in current practice; but the novelty lies in the scale of the forces involved, for in fact each of the ties exerts a force of 1000 tonnes at the crest of the dam and as there are altogether 37 of them in the work the result is to form an artificial load equivalent in the aggregate to one-third of the natural weight of the structure and half of the pressure due to the water. The line of pressure can thus be controlled at will, and is deflected powerfully in the direction of safety.

It need hardly be said that the introduction of forces of this order was a matter which called for long and careful preparation, particularly since the foundation

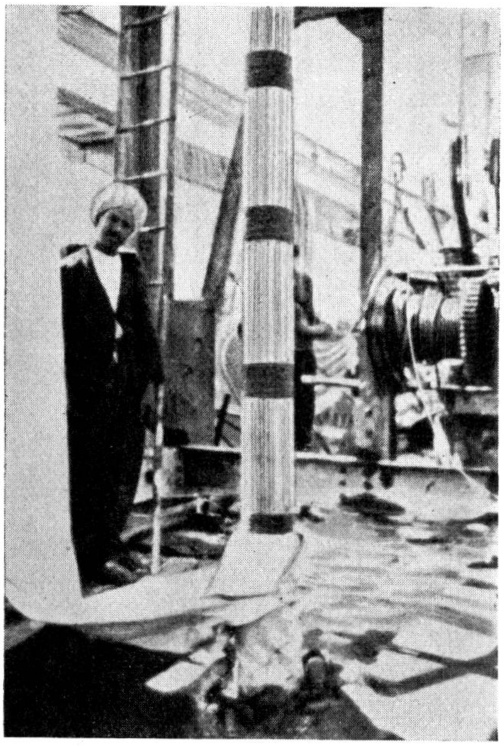


Fig. 3.

Insertion of the 1000-ton anchoring tie.

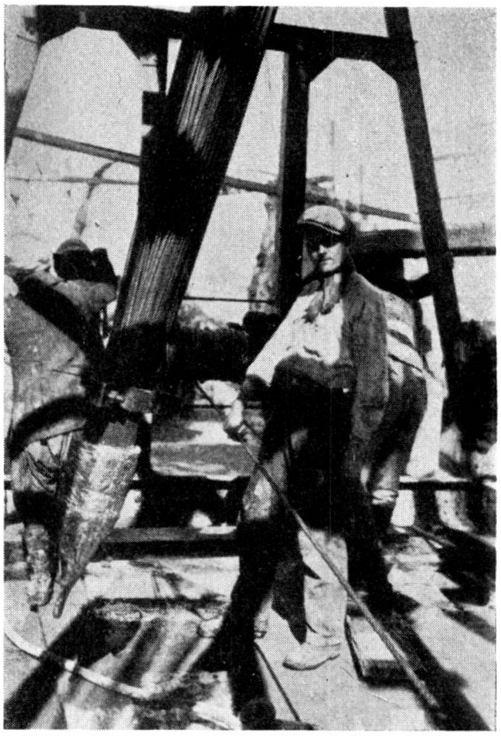


Fig. 4.

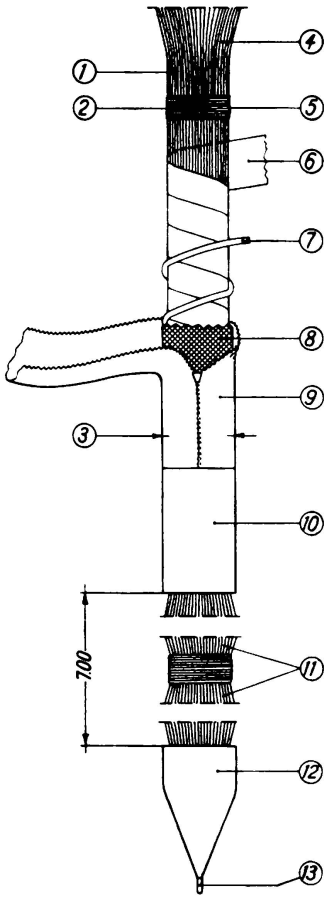
Bottom end of 1000-ton anchoring tie.

underneath the dam is of very inferior quality, consisting of soft sandstone containing pockets of chalk, marl and even quicksand.

Vertical shafts 25 cm in diameter and 50 m deep were cut through the dam into the foundation below, and at the bottom these shafts were widened out to form two anchoring chambers one above the other each 3 m long and 38 cm in diameter, by means of an enlarging tool. Within each shaft there was introduced a cable consisting of 630 parallel wires of hard steel, 5 mm in diameter (Fig. 3), and these wires were bound over the whole length except for some metres at the bottom (Fig. 4); where they enter the anchorage chambers they spread out under the weight of the cable itself. Cement was then injected at the bottom of the shaft by means of a pipe lowered together with the steel cable, but to prevent the whole length of the cable from being sealed and grouted to the shaft it was enclosed, above the level of the anchorage chambers, in a special sleeve of bituminous material between two fabric covers (Fig. 5). Except where actually sealed, the cable is, therefore, completely independent of the surrounding masonry shaft.

Above the crest of the dam the wires are spread on to a reinforced concrete crosshead, jacks being inserted between this and the crest to exert the necessary stretching force (Fig. 6 and 7). The unit stress in the steel is of the order of 80 kg/mm², or between six and seven times the permissible stress in reinforced concrete.

All the anchorages carried out in this way proved completely successful at the first attempt, despite the bad quality of the ground, and in the course of time



(1) 630 Nos of galvanised steel wires, each 5 mm in diam.
(2) 15 cm diam. of bound cable,
(3) 20 cm average diam. of completed cable.
(4) Flintcote coating.
(5) Bindings every 50 cm.
(6) Canvas soaked in Flintcote.
(7) Aloë-rope.
(8) Plastic layer (compound of fat and bitumen).
(9) Canvas cover with Zip-fastener.
(10) Mortar cover over wires and canvas.
(11) Wires laid bare for concreting-in.
(12) Conical white-metal shoe.
(13) Injection pipe.

Fig. 5.
Construction of anchoring cable.

they have lost at the most a few hundredths of their initial tension. The tension is easily controlled, and can at any time be restored by replacing the jacks under the crosshead of the cable.

The adoption of this system made it possible to realise an economy of ten million francs (Fig. 8). The credit for it is due to M. *Vergniaud,* Ingénieur en Chef des Ponts et Chaussées, and M. *Drouhin,* Ingénieur des Ponts et Chaussées, in conjunction with Messrs. *Rodio* as contractors.

The procedure can be applied in a number of ways. It has for instance been used for strengthening the lighthouse in the sea near Jument d'Ouessant (Fig. 9), rendering that structure invulnerable to even the heaviest storms, despite the great violence of their impact against its sides. This was accomplished by fixing the lighthouse to the ground by half a dozen tie bars, each stressed by 1500 tonnes.

It is, however, chiefly in new work that the process has been found effective and economical. The thrust of arch bridges or the tension in the cables of suspension

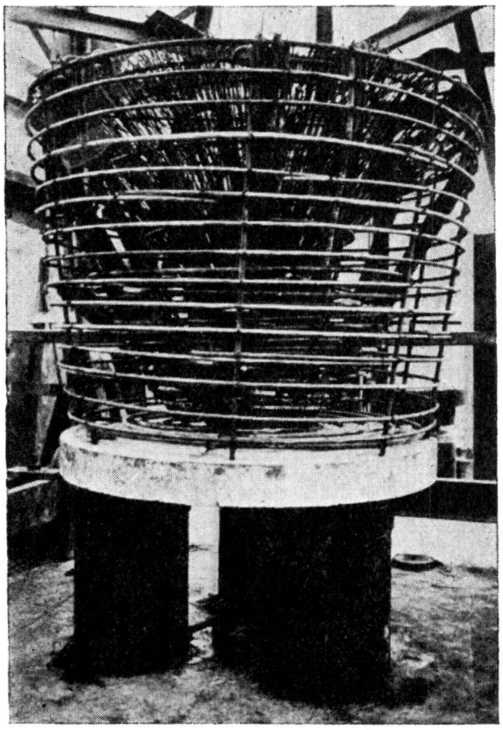


Fig. 6.
Reinforcement of head of anchoring tie.

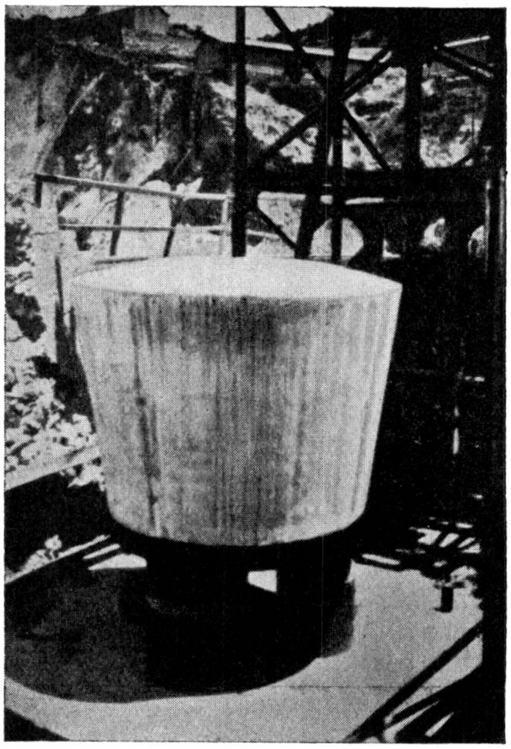


Fig. 7.
Concrete head of anchoring tie on three hydraulic jacks.

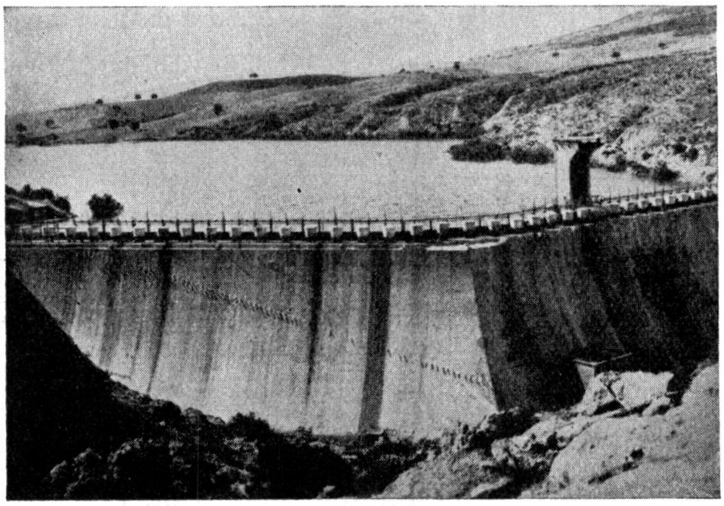


Fig. 8.
General view of Cheurfas dam after its reinforcement by 37 1000-ton anchoring ties, the heads of which may be seen.

bridges can be taken up in the same way, without the need to use huge masses of masonry which are costly and bulky.

It is, indeed, often a question of space which makes this expedient necessary. This is true in regard to certain abutments of arch bridges, or river training banks built on bad ground, which in this way can be fixed down with complete safety. Such a case arises on the right bank of the Marèges gravity dam which is founded on a thin ledge of granite. As a rule the quality of the ground will be sufficiently good to allow of dispensing with the precautions which were neces-

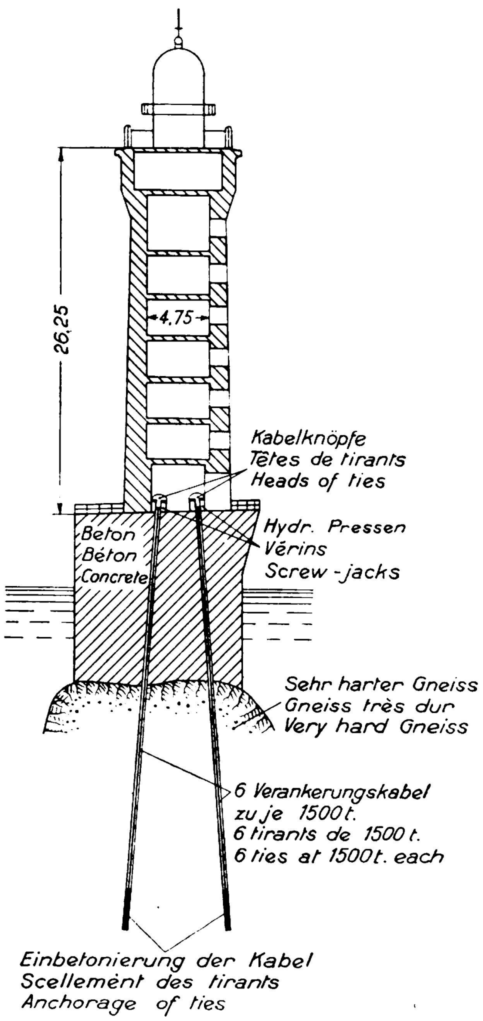


Fig. 9.

Section through lighthouse at Jument d'Ouessant with anchoring ties.

sary at Cheurfas, where the application was made the first time in difficult ground — for instance it should be unnecessary to enlarge the bottoms of the shafts and to provide the thies with a plastic protective cover; the tie could simply be introduced without any special preparation into a hole *not enlarged*, at the bottom of which an accurately pre-determined amount of cement grout would be introduced as in ordinary sealing work. Finally, after the tension had been imposed, the hole would be filled with concrete from top to bottom. Moreover the use of cables prefabricated in the workshop would greatly simplify maintenance on the job and would considerably reduce the cost (Figs. 10 to 15).

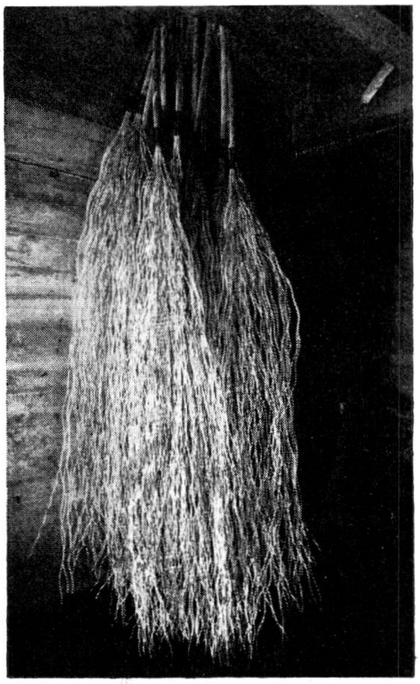


Fig. 10.
Marèges dam: anchorage of gravity abutment, showing the bottom end of the anchoring tie unravelled.

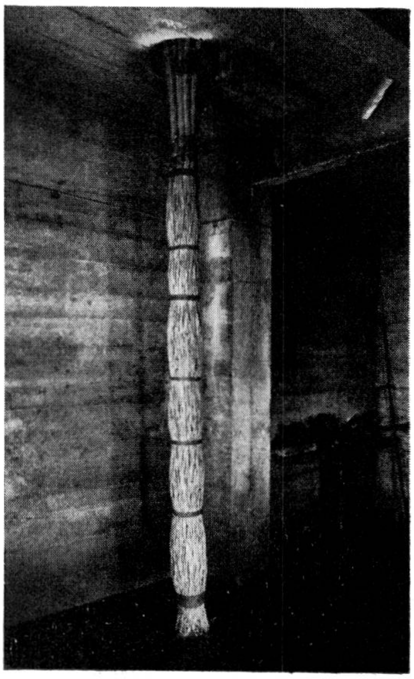


Fig. 11.
Marèges dam: anchorage of gravity abutment. The bottom end is shown bound to allow of insertion in the bore hole, but only the lowest of the bonds is to be retained.

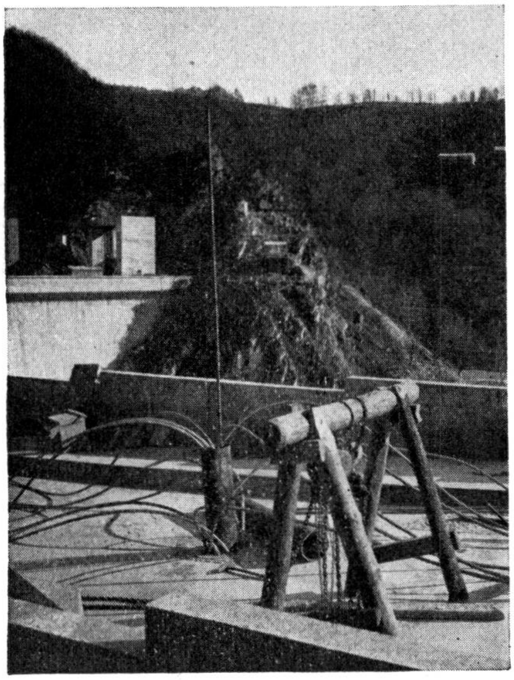


Fig. 12.
Marèges dam: anchorage of gravity abutment. Formation of the anchoring tie in progress on the crest of the dam, showing the pipe to be used for injecting cement grout around the bottom end.

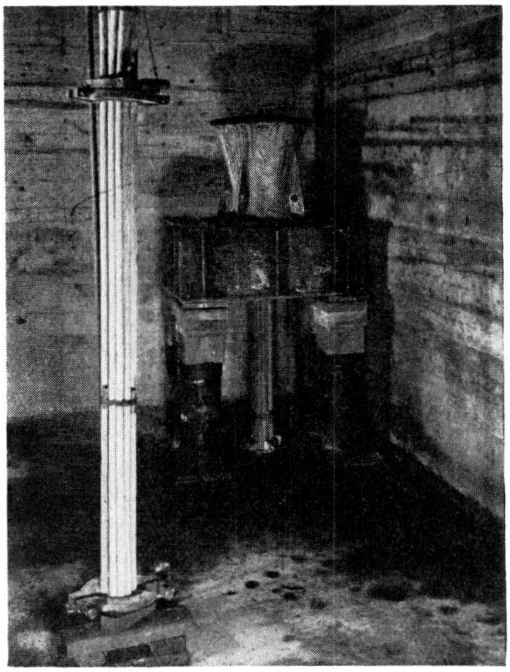


Fig. 13.
Marèges dam: anchorage of gravity abutment. In the foreground is part of an anchoring tie composed of 15 cables, and in the background the head of such a tie already tensioned and still supported on six hydraulic jacks.

The process thus made available is an accurate, powerful and economical means of creating certain artificial elastic conditions in mass structures, or even of greatly modifying their statical equilibrium. One may visualise in this way a fundamental change in most of the accepted forms of design for structures intended to resist lateral pressures.

Fig. 14.

Marèges dam; anchorage of gravity abutment. Anchoring head ready for casting the sleeve.

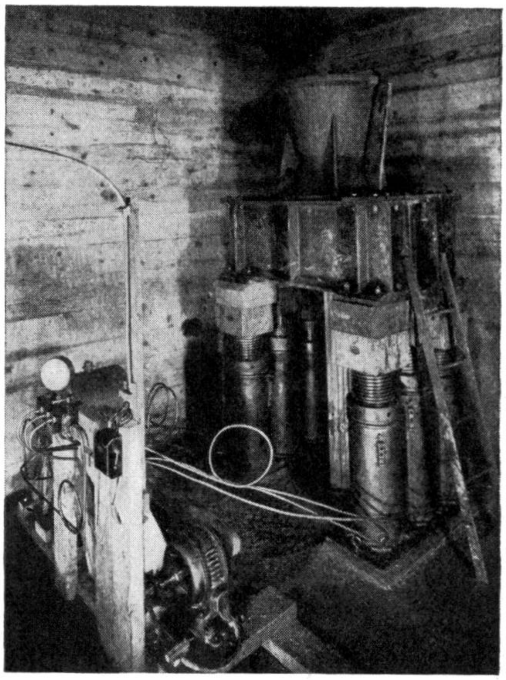


Fig. 15.

Marèges dam: anchorage of gravity abutment. Tensioning of tie by means of six hydraulic jacks.

VI 8

Concrete in hydraulic constructions.

Beton im Wasserbau.

Le béton dans les constructions hydrauliques.

Dipl. Ing. J. Killer,
Baden (Schweiz).

The design of concrete structures for hydraulic work should follow radically different lines from those customary in work above ground. In the latter the structures are for the most part protected from external influences, so that their cross sections can be determined from statical considerations alone, but below ground or below water the external influences play as important a part as the statical design of the structure. In view of the effects of aggressive water care must be taken to keep the concrete free from any tendency to cracking, and on this account the permissible stresses laid down in the regulations cannot be fully utilised. Above all, in work below ground or below water it is important to build in mass concrete, by contrast with work above ground where the highest of permissible stresses and the newest principles of construction may be exploited. At low levels, however, the principle of mass should predominate. Since every kind of water (but especially water which is poor in lime) attacks concrete, it follows that a greater thickness of construction will resist aggressive water longer than a light concrete member with correspondingly heavier reinforcement; it is of advantage, therefore, to use a larger quantity of concrete and a smaller amount of steel. In hydraulic works the concrete is to a great extent soaked in water, and in course of time, if the workmanship has not been good, this may lead to deterioration. Hence the concrete must be made as dense as possible. Another especially dangerous effect on the concrete is that of frost, and this applies especially at those points which are within the range of fluctuation of the water level.

Experience gained in the construction of dams and weirs in Switzerland has shown that a great deal of importance must be attached to the problem of frost. In the case of the Barberine and Wäggital dams, both constructed between the years 1922 and 1924, concrete made with 300 kg of cement per m^3 proved frost resistant, whereas the concrete containing 190 kg per m^3 used in the dam itself suffered great damage from the effects of frost, with the result that within a few years a lining of natural stone had to be added. In view of this unsuccessful experience in the use of a lean mix of concrete, the more recent dam constructions in Switzerland, those at Dixence and Etzel, are being concreted outside with 250 to 300 kg of cement per m^3, and even so are being lined with natural stone. The core of these dams was made with a mixture of 200 kg of cement per m^3. In the case of the Etzel dam the masonry lining on the water side was carried up to low water level. The practice of concreting from towers has also been completely given up. Whereas at Dixence the practice was maintained of carrying the concrete over a service bridge through a small system of channels to its point of application, the Etzel dam, now in course of construction, is being concreted entirely by means of cranes and buckets with a view to the avoidance of all risk of de-mixing. This dam is situated in the subalpine region and every precaution known to modern engineering is being adopted in order to make the work frost proof. It is an open question, however, whether in the future it will be necessary to go so far as to provide special outer concrete as well as masonry lining; in the interest of the economy of hydraulic power installations the author is of opinion that endeavours should be directed to the production of a concrete that can be counted upon to remain frost proof even at high altitudes — the more so since the stone facing, a mere palliative, prevents any control from being exercised over the concrete within, and since at high altitudes the effects of frost have been detected as far as 2 m into the mass. Stone facings are as a rule not thicker than 70 to 80 cm, so that frost is able to pass through them and enter the mass of concrete. It is possible to ensure frost proof concrete through exceptionally careful workmanship combined with low water content and a high proportion of cement; above all, care is necessary to guarantee that no unmixing takes place on the way from the mixing plant to the point of use. Experience teaches that concrete which has had to travel a considerable distance through channels is not as frost-resistant as concrete in the same job which has had to travel only a short distance through such channels.

It was formerly the standard practice, in Switzerland, to line the piers of weirs with natural stone masonry over the whole of their height, and good results were obtained by so doing. In more recent hydro-electric work the practice has changed to that of carrying the lining up only as far as boulders are liable to be encountered, and to leave the upper portion rough as if comes out of the shuttering. It is mainly within the range of fluctuations of water level, however, that serious frost damage has occurred, and the question has again arisen whether the lining ought not to extend over the whole height liable to be under water. Such a lining, however, is very costly, because the frost proof stone as a rule has to be brought from a great distance, and here again attempts must be made to obtain a concrete which will offer adequate resistance against external effects.

In one instance at a hydro-electric station, frost damage could be observed on the weirs proper while portions of the power house itself exposed to the same influence remained quite free from damage: the aggregates used and the mix adopted had been the same in either case. The heavier reinforcement in the part belonging to the power house entailed more careful concreting. This example clearly shows that provided the workmanship of the concrete is sufficiently careful a great improvement in quality can be ensured. Above all, in the construction of hydro-electric works where various portions are particularly liable to infiltration of water and are exposed to frost, the concreting should no longer be done by using chutes but only be means of cranes and buckets or belt conveyors. The latter method affords the best possibility of ensuring that the concrete will not become unmixed in transport.

Special care is needed in the concreting of pressure tunnels where the in-rush of water through fissures in the rock may give rise to great difficulties. Since the cross section now most commonly adopted for such tunnels is circular, the placing of the concrete becomes a difficult matter, and a very sloppy mixture containing an increased proportion of cement is necessary. Moreover, the concrete easily becomes de-mixed on its long journey through the tunnel, and it is essential to re-mix it immediately before it is placed.

When it is remembered that repair work to a pressure tunnel necessitates the whole of the power plant being put out of action, the importance of perfect treatment and working of the concrete used for such linings will be appreciated.

Good evidence of the fact that heavy frost damage is attributable mainly to the use of liquid concrete may be found in the unimportance of such damage in works built of earth-damp rammed concrete before 1920; before liqued concrete began to be used. Despite the fact that in these early jobs the sand and stone aggregates was, in most cases, taken as it was found near site, the concrete, old as it is, now gives a favourable impression; no doubt the reason for this is that the production of rammed concrete necessitated each layer being consolidated in turn and being free from any excess of water. This is a clear indication that in the future every attention will have to be paid to the working of the concrete — an object not attained merely by the winning of new knowledge through research, but requiring also effort on the part of colleges and institutions to ensure that in course of time engineers and specialist workers are turned out who have apprehended such knowledge and are capable of producing high quality frost-proof concrete on the job.

In view of the heavy vibration to which they are exposed through water shocks and the presence of the turbines, weirs and power houses should not be too lightly and elastically designed. In such works a relatively larger mass and correspondingly less reinforcement is appropriate. Side embankment walls should be provided with some system of drainage to carry water away from the concrete, and the crests of these walls should carry natural stone slabs to protect them against the effects of spalling by frost.

Concrete has been found a suitable material to use in hydraulic work to form those special shapes which modern practice demands — such as inlet spirals, suction pipes, spillways, etc. — at minimum cost. Since, however, these constructions are permanently immersed in water the concrete is very much exposed to frost which may in time damage the work, and the production of concrete for hydraulic jobs makes great demands on the constructing organisation. Concrete mixed with only just enough water to render it workable and containing at least 250 to 300 kg of cement per m^3, carefully mixed and placed has hitherto been found frost resistant. The use of vibrators to increase the density of the concrete serves to increase its surface strength, and may add much to its power of frost resistance.

The great object to be pursued in the future, in making concrete for hydraulic works, must be the attainment of a material which offers adequate resistance to external effects. It is only necessary to remember the large amounts that frequently have to be spent on reconstruction, owing to defects in the original construction of major hydraulic works, to perceive how vital it is that research and practice should combine in doing their utmost towards improving the quality of concrete.