SYMPOSIUM ON ARCH DAMS

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(OVER)

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ARCH DAMS: THEIR PHILOSOPHY

Andre Coyne,[1] Hon. M. ASCE
(Proc. Paper 959)

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June 1956 at Knoxville, Tennessee.

Since the last symposium on masonry dams (April 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be printed from the Symposium. So far, two papers have been approved: "Arch Dams: Their Philosophy," (Proc. Paper 959) by Andre Coyne, Hon. M. ASCE, and "Arch Dams: Trial Load Studies for Hungry Horse Dam," (Proc. Paper 960) by Robert E. Glover, M. ASCE, and Merlin D. Copen.

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

The Past

There is nobody who can flatter himself on being the inventor of the arch dam, and there is quite a chance we will never discover how the whole setup started.

The first historical specimens are to be found in Spain and Italy, a century apart.

In Spain, Almanza and Elche dams date from the 16th century. In Italy, Pontalto dam (Fig. 1) built in 1611 and raised several times since then, was mentioned by the late lamented Noetzli. The similarity of Pontalto dam with the bridge which is above it, is so obvious that it hardly needs to be pointed out, and it illustrates, better than any highbrow argument, how an arch dam is a kind of bridge overturned in an upstream direction. The oldest known examples are just like this and if the comparison seems a bit far-fetched today

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1. Inspector General, Cons. Engr., A. Coyne and J. Bellier, Paris, France.

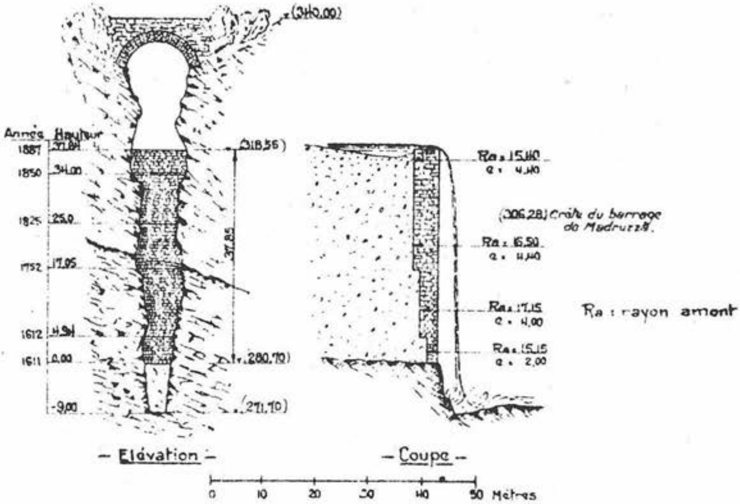

Fig. 1
Barrage de Pontalto

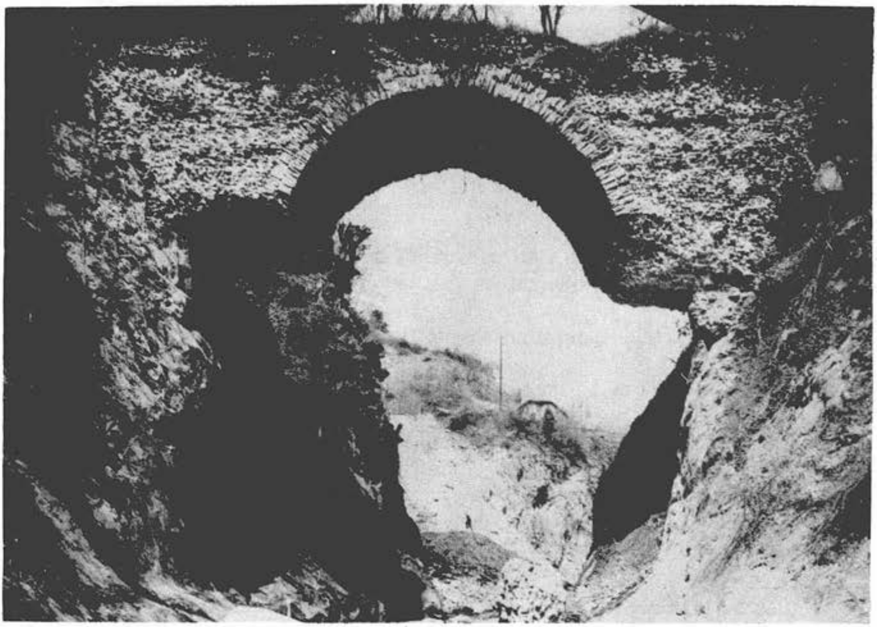

Fig. 2
Roman Bridge
(Photograph by: Ollière, La Mure)

it is because we vary the thicknesses and curvature so as to try and adjust our arches to the shapes of the valleys and the thrust of the water.

In France, the ancestor of arch dams is Zola dam near Aix-en-Provence, built in 1843. Zola, who designed it in 1839, was the father of the famous novelist. He really designed two dams, one 30 m (98 ft) high and the other 48 m (157 ft) high, on the same basic principles, but died before seeing his projects realized. Only the first dam was built, to a height of 36 m (118 ft), after his death.

At that time massive walls were the only known solution. Their dimensions were worked out by rough and ready empirical methods without taking any account at all of lateral support. The unquestionable merit of Zola was the fact he was daring enough to count upon an arch effect resulting from a curved setting out. Accurate calculation of the arch effect was beyond him - and of course beyond the technical knowledge of his times - but in his notes he underlines the safety given to his dams by their curvature and by the strength of the banks. He thus justified himself, against Academic criticism, for using lesser thicknesses than those admitted as standard in these days.

Then it is only just to mention in this historical review, the American pioneers who, a little bit nearer our days, had the courage of constructing arches, lots of arches. The most daring of these are, curiously enough, the oldest: Bear Valley and Upper Otay.

The great Jorgensen and his arches with a constant angle deserve special homage, for all modern Engineers are aware of having drawn their inspiration from his works.

Whence the Strength and Safety of Arch Dams?

The comparison just made of an arch dam to a bridge overturned in an upstream direction needs two explanations.

Firstly, contrarily to what happens in bridges, the weight and the load, instead of acting on the same plane, act more or less at right angles in dams.

Then, this time as in bridges, the thrust reactions of dams, and generally speaking their stabilising stresses, increase with the load. Contrarily to what happens in gravity dams, it is the structure itself that acts, producing practically automatically its own reactions and stresses, right up to the point strictly necessary for maintaining equilibrium.

This is why it has been said that arch dams work as self-sealing plugs, becoming stronger and more taught as the thrust of the forces bearing down on them increases. In hard fact, the total and general crushing of their constituent materials is their only yield point.

It goes without saying however that this faculty works in different ways, for arch dams are hyperstatic structures: even more so than encastered bridges because of the number of their peripheral connexions.

In other words, arch dams find many different ways - perhaps even an infinity - of solving the stability and strength problems required of them. There is on the one hand the inevitably unpredictable behavior of the foundations, where accurate advance estimates are difficult and on the other, the continually changing conditions created by different heads of water and above all varying temperatures. Arch dams, like the Roman bridge which to this day overarches the River Drac in France (Fig. 2) deal with all this by drawing on their hyperstatic reserves.

How then can we be surprised that such a simple principle has never betrayed the confidence placed in it - perhaps instinctively, and that no accident has been seen as yet in a type of structure which actually seems to be invulnerable?

What is more, it was enough, in the case of a structure justly reputed vulnerable, like Ternay gravity dam (Fig. 3), to give it a slight curvature (radius 400 m (1310 ft)), to save it from destruction. This without any research or calculation. The fact that it was the curvature alone that saved Ternay is all the more certain when we realize that all the other dams with the same cross-section but a straight setting-out have failed as a result of uplift.

There is however one failure of an arch dam on record. It was the one we built for experimental purposes, precisely so as to test it to failure (Figs. 4 and 5).

It was a simple concrete arch 3 m (10 ft) high, 20 m (66 ft) downstream radius and only 20 cm (8 in) thick, encastered at each end in excellent quality rock. Its upper part was covered over with a concrete roof with a watertight device. Thus was created a closed space in front of the cliff, where pressure could be built up to the limit tolerated by the arch. After several progressive tests when various measurements were made, a test to failure was applied. Failure started where the arch was thicker than elsewhere, and gradually extended, whereas in the parts of normal thickness, the concrete worked at about 300 kg/cm2 (4,300 lb/in2) without showing any visible signs of excessive strain. So the experiment revealed both the great strength of arches and the drawbacks of any awkwardly applied stiffening which, far from strengthening structures, is a cause of weakness. Elasticity is the essential quality of a good arch.

Apart from this exception, the conclusions which were valid twenty-five years ago, when Noetzli, with a keen sense of prophecy and perfect comprehension of his subject, founded his Study-Group for Arch Dam Investigation, are still valid today.

There is no known failure of an arch dam.

Evolution of Basic Criteriums

Since Noetzli, however, the criteriums applied have undergone a notable evolution, specially as far as the working stresses applied are concerned.

About twenty years ago, the average stresses, calculated by the tube formula for the great prototypes: Pacoïma, Diablo Dam, Ariel Dam, Marèges, Santa Luzia, were about 25 to 30 kg/cm2 (400 lb/in2). Today, they run up to 50, 60, 70 kg/cm2 (700, 850, 1,000 lb/in2) at Rossens, Val Gallina, Tignes, Salamonde, La Palisse, Malpasset, etc. (Fig. 6).

Exceptionally, average stresses exceeding 100 kg/cm2 (1400 lb/in2) were applied in the case of le Gage Dam.

With the average theoretical stresses, effective compressive stresses have risen from 30 to 40 kg/cm2 (400 to 550 lb/in2) (Marèges) to 60 and 70 kg/cm2 (850 and 1,000 lb/in2) (Salamonde) and even 110 or 120 kg/cm2 (1550 and 1700 lb/in2) at le Gage. A few words on this record dam will not be out of place here.

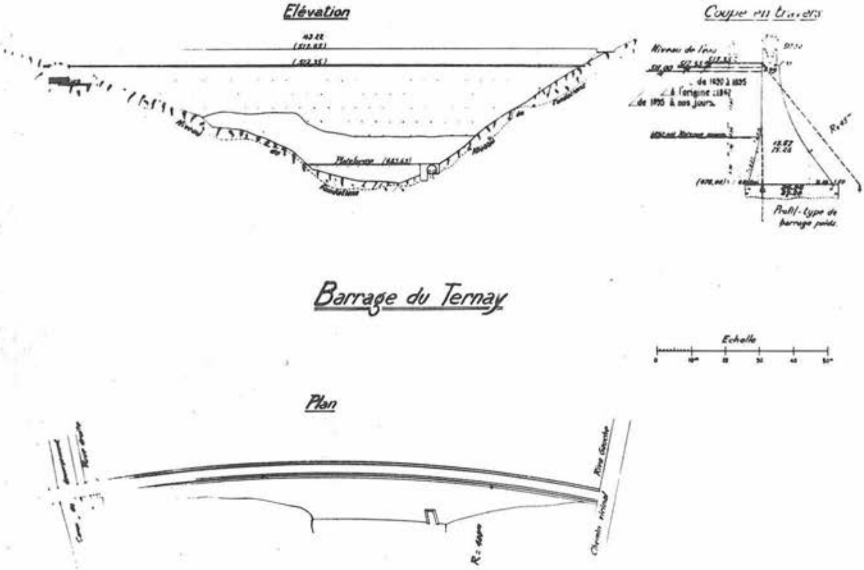

Fig. 3
Barrage du Ternay

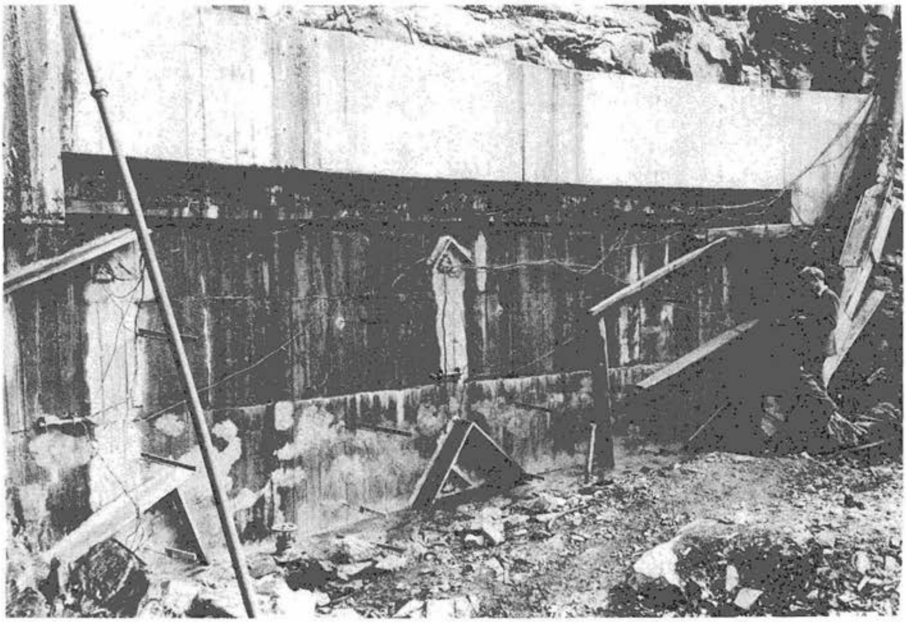

Fig. 4
Experimental arch dam before test to failure.

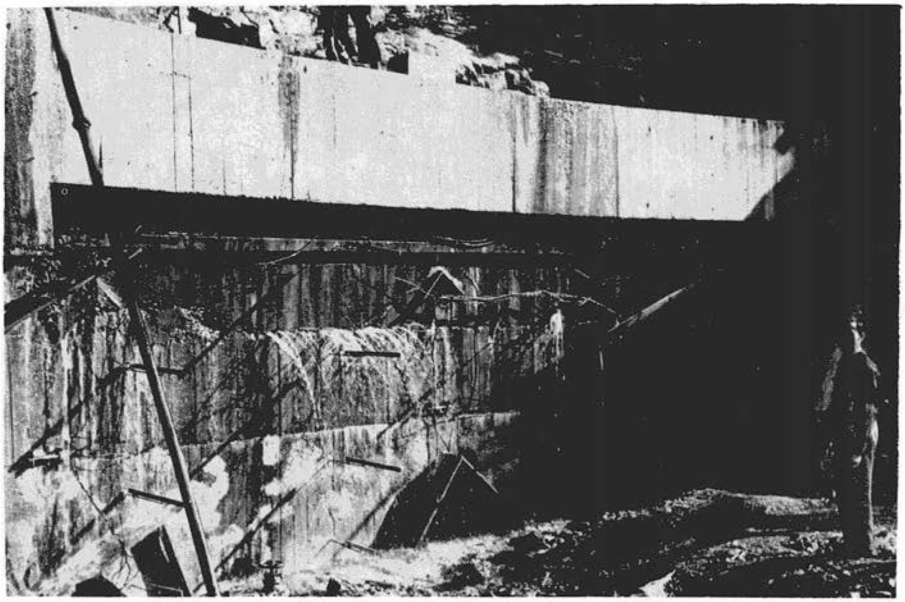

Fig. 5
Experimental arch dam after test to failure.

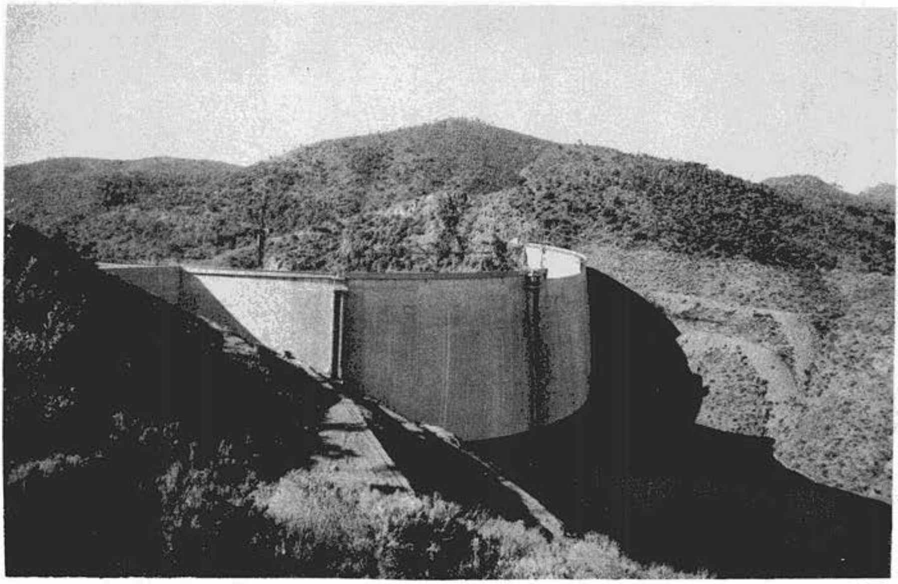

Fig. 6
Malpasset Dam
(Photograph by: Harand, Paris)

Le Gage Dam

A very thin arch, 38 m (125 ft) high (Figs. 7 and 8) built on one of the higher tributaries of the Loire in the framework of the Montpezat scheme, where water coming from the upper part of the Loire catchment area which would normally have flowed into the Atlantic Ocean, was diverted into a river running into the nearby Mediterranean. The object of this operation was to secure higher and more direct head. As the location was suitable, le Gage was made a sort of experimental dam, by designing it with the exceptionally high stresses mentioned above.

The first practical consequence was a reduction, also exceptional, of concrete volume for the dam. It is only 18/100 of what it would have been for a gravity dam at the same site, with the same excavation depth. We all know that for modern arch dams the proportion is still of the order of 1:3, i.e. twice as high as at le Gage.

The reduction in volume thus realised secured all the more economy because the working site of le Gage was only a few kilometers from the larger dam of La Palisse, then under construction. So as the quantities required were small, ready-mixed concrete was transported by road to the smaller dam. No question of borrow-pits or special mixing plant and the resulting expense. Over and above the economy in concrete unit-prices, there was a considerable saving of time. The dam was completely constructed within a single summer season.

Deflections and stresses were of course frequently measured when the reservoir was filled. It was thus ascertained that they were at no point above the maxima expected by the designers, but that the localisation of these maxima was however different. Better still, le Gage provided a remarkable example of the variety of ways arch dams have at their disposal for solving the elastic stability problems they are up against. Measurements have for instance shown that on filling the reservoir the dam foundations underwent very marked changes resulting from local subsidences of the rock, of excellent quality however on the whole. The distribution of stresses altered several times running and in some places the concrete freed itself from excessive extension by cracking, without any serious consequences.

It is thus not surprising that the true behavior of the structure was markedly different from that foreseen in the calculations, even those made on the basis of the most complex methods of adjustment, for the assumptions did not correspond to the actual conditions, which moreover, varied as the filling of the reservoir went on. In last analysis, it would seem that the fairly simple method of plunging arches gave the most satisfactory forecast of stress distribution in this arch.

Evolution of Shapes of Arch Dams

Parallel to the progress realized in stress values, shapes have thinned down, and have, further, evolved in very different directions.

Sometimes the lower part of the arch overhangs on the upstream side, a method applied for the first time twenty years ago at Marèges. This dome shape is the most suitable one where extension has to be fought, particularly at the encastrement of central cantilevers as in the case of Salamonde (Fig. 9), Cabril and many Italian arch dams.

Sometimes the arch slopes in a downstream direction as for Enchanet and Couesque (Fig. 10), so as to reduce the radius of the lowest part, deliberately

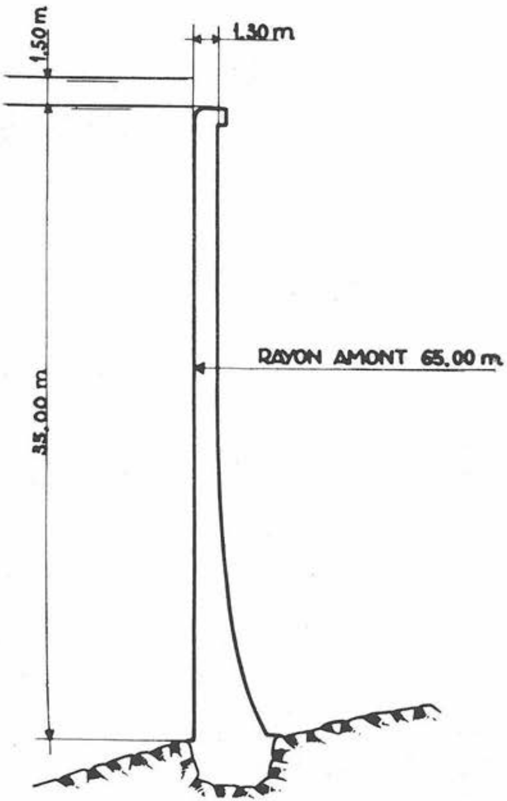

Fig. 7
Barrage du Gage

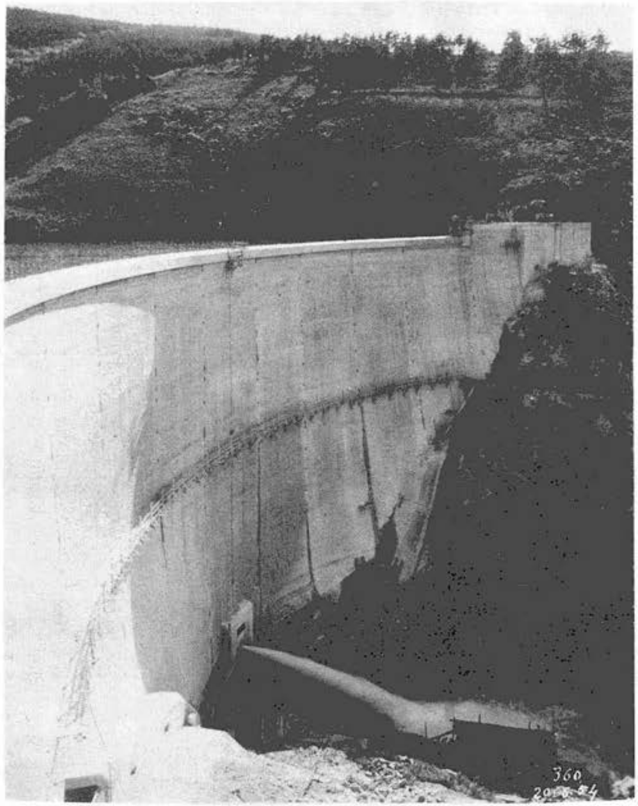

Fig. 8. Le Gage Dam
(Photograph by: H. Baranger, Paris)

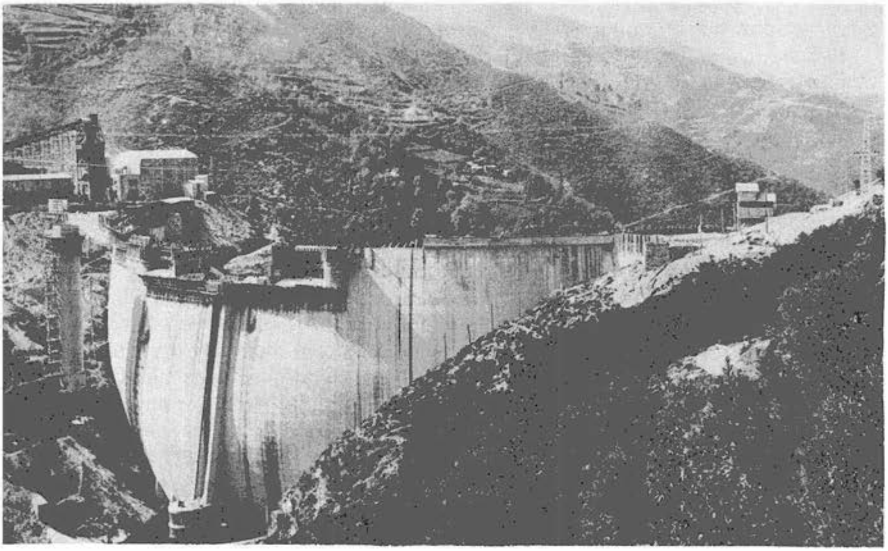

Fig. 9
Salamonde Dam

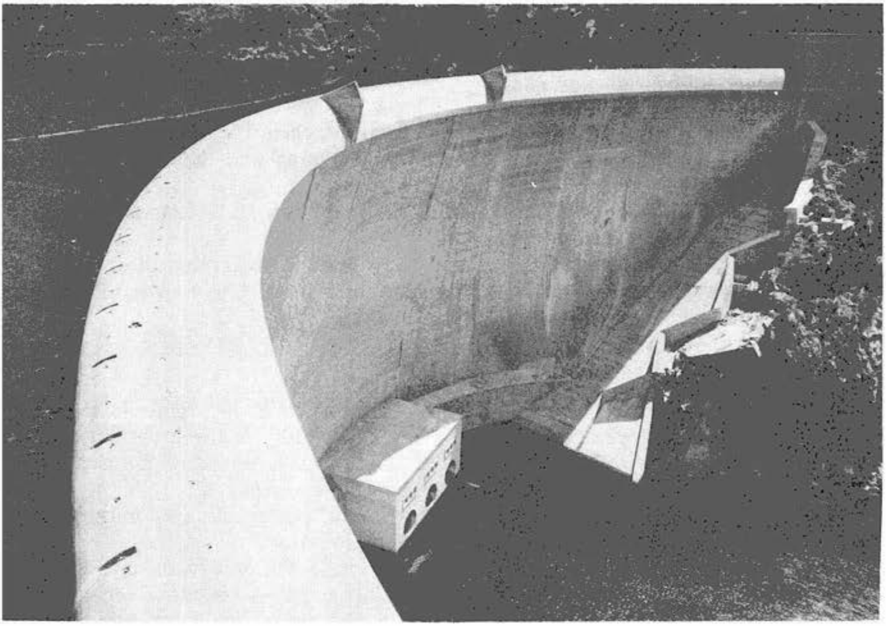

Fig. 10
Couesque Dam
(Photograph by: H. Baranger, Paris)

accepting breakage of the bottoms of the central cantilevers, which results in a crack subsequently plugged by grouting, as at Enchanet.

Shapes are sometimes simplified to their ultimate limit, for facilitating construction, as at Pont-en-Royans; for reasons concerning the foundations, as at Bioge (Fig. 11) and la Mandraka; or for the spillway, as in the case of Grangent, with its 5000 m3/s (178,000 cf/s) capacity crest spillway.

Then on the contrary for other reasons but still with the same objective of economy, we have complications of structure, such as local stiffening of the arch by adjoined or incorporated appurtenant structures (intakes or outlets); or else the elastic deformations are thwarted by blocking the toe of the dam with an appurtenant structure or even a large powerstation carrying a ski-jump spillway, as at l'Aigle (Fig. 12), Saint-Etienne-Cantalès and Chastang; or else a large opening is made right in the middle of the stress-flux, in order to provide a bottom sluice so as to eliminate temporary diversion tunnels as at Marèges and l'Aigle or to serve as a flood evacuator as at Castelo do Bode, Chastang (Fig. 13) and la Roucarié. Or else, even the mass itself is hollowed out and the power house placed in the orifice as at Monteynard.

Finally, as growing success lead to even greater daring, and progressively justified itself, arches were built even on locations which would have been considered unsuitable not so very long ago.

As at Bin el Ouidane, great asymmetry was accepted. Suspect foundations were tolerated with of course due precautions if the ground was hollow: grouting, as at Castillon; if the banks appeared unstable: strengthening with cables as at Castillon and la Chaudanne; if the ground was relatively soft: widening of the base as at Bort; if there was little or no hope of reaching sound rock at an economic price: deepening of the underground part of a thrust block, as at la Mandraka.

Looking back on these dams, it is clear that they are very different from each other.

They have however one common characteristic; they are all arches.

There is no doubt whatsoever that this fact explains why they hold out so perfectly.

Because an arch is curved, the difficulty would seem to be not to make it hold up, but to knock it down.

Serious accidents do not even occur: there are two examples of arch dams where a buttress was washed away, while the arch itself stood up to the pressure of the water: Moynie and Lake Lanier dams.

Economics of Arch Dams

A simple adjustment of the radial deflections at the crown tends to show that the dome-shape, overhanging on the upstream side, is the most rational one, at least as far as the thwarting of extension is concerned. This conclusion is immediately confirmed by membrane or model tests.

It is this shape that gives maximum economy of materials with maximum economy of stresses, particularly on the extension side.

Maximum economy of materials rarely corresponds, however, to true economy, specially for small or average size structures, because of the special formwork requirements and particularly because of the obligatory slowing up of construction through complicated shapes and layouts or through excessive thinness.

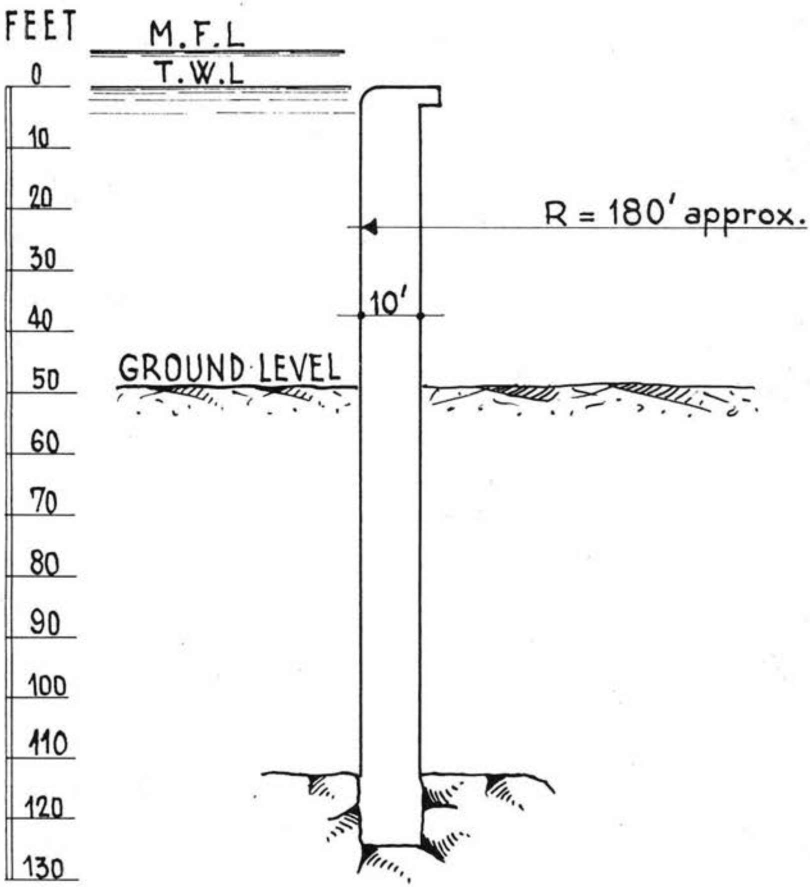

Fig. 11
Bioge Dam
Cross-Section

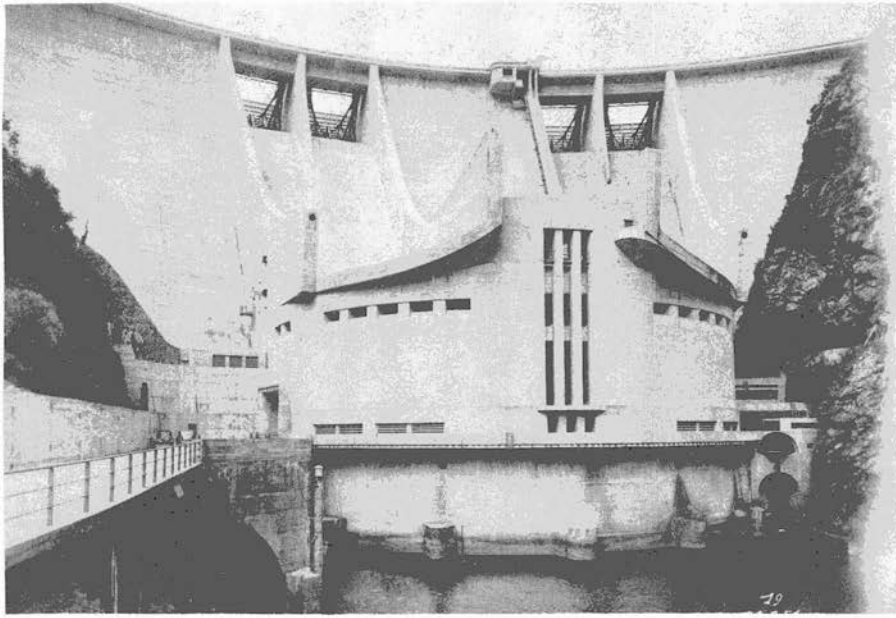

Fig. 12. L'Aigle Dam
(Photograph by: H. Baranger, Paris)

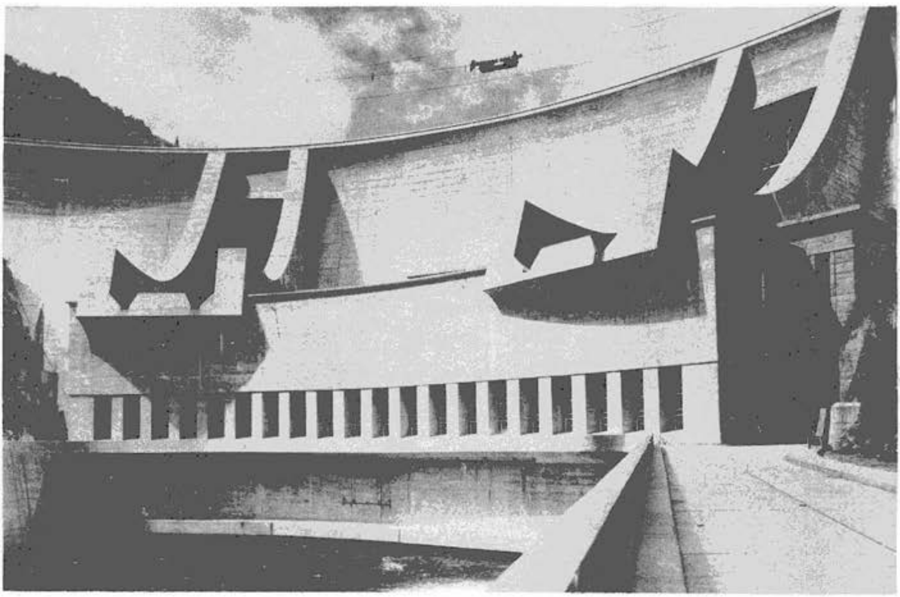

Fig. 13. Chastang Dam
(Photograph by: H. Baranger, Paris)

In any case, experience proves that extension and even cracks in arch dams are not dangerous.

It follows that economy in stresses, particularly at each end of a dam, and economy of materials are far from being always predominant considerations. There are even cases where it is better to keep them to a merely secondary status. Thus in the case of small or average dams, even if they are daring as at Le Gage, it is often preferable to keep to simple shapes, even at the cost of running the risk of fissuration.

On the contrary, extra amounts of volume are accepted as in the United States for instance, where gravity dams win the day from an economic standpoint.

Generally speaking, as far as arches are concerned, striving after economy in volume is most certainly a paying proposition in the case of large-scale dams and those whose shapes are more than usually out of the ordinary.

This is the case of Kariba arch dam on the Zambezi (Fig. 14). Its adoption for a very wide valley will result in great economy as compared to a gravity dam and will above all greatly simplify the question of the diversion of the river.

Calculation

The calculation of arch dams is difficult, owing to the very reasons whereby they resist all the various forces so well: that is to say, their great hyperstatic capacities, which enable them to compensate both for any unexpected foundation defects and for inadequate dimensions.

It would seem, after comparing the stresses obtained by calculation to those actually measured hitherto, that before we can hope even to begin to approach reality by calculation, we must go very far, for reality escapes our grasp; particularly as far as thermal effects are concerned, and it is more or less unpredictable where foundation reactions are concerned.

The United States engineers who systematized and developed so extensively the Trial Load Method had the merit of being the first to take account of the deformability of foundation rock. It is this that has given the method greater accuracy than others. Nevertheless, many French experiences, followed by European ones, have revealed that this deformability could readily be very much greater in the actual foundations as a whole than on a sample of rock or according to tests carried out within a limited area on the damsite and that it depends on the site, the banks and the elevation.

Then, the more daring, the thinner, the more loaded the dam, the greater the importance of local rock defects. The experience of Le Gage shows that these defects can cause such foundation changes that several series of calculations would have had to be carried out, on different support assumptions, so as to cover all the facts.

All this is enough to discourage the toughest Engineer, when we think of the extreme complexity of even the standard Trial Load calculations. In any case, the question as to whether it is all worth while arises, for we have another quicker, more reliable and cheaper method of investigation at our disposal: structural models.

Structural Model

The Italian and Portuguese schools of thought in the matter of small-scale models have got Designers into the habit of thinking that such models provide

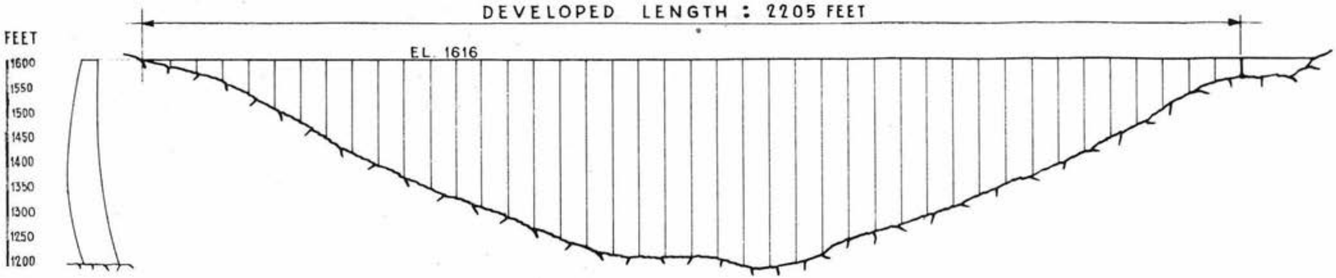

Fig. 14. Kariba Dam
Developed Upstream Elevation

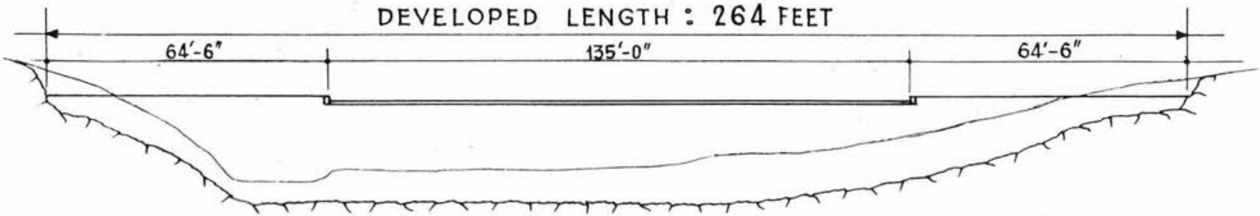

Fig. 15a. Moulin Ribou Dam
Developed Downstream Elevation

sufficient certainty for going boldly ahead, without wasting too much time - even with the help of electronics - on solving equations. Equations in any case have one great failing: if the results actually achieved are not consonant with the results expected, it will be difficult to summon up the courage to start all over again.

So the best thing to do is to put up with simplified calculations, giving results within two limits and to get from them the courage for making up one's mind for choosing a given design. This is then immediately elaborated on a model, but with the clearly defined intention of retouching this model several times over so as to gain little by little on volumes and stresses, as there is only a very slight chance of the first model working out right from the start.

This method has borne its fruit in modern progress.

Present-day applications are very different from those fashionable a few years ago, consisting in using the models simply to confirm the correctness of the method of calculation adopted, and specially that of the Trial Load Method, then a subject of controversy. Today, over-exacting calculations, should they be necessary, particularly for calming the troubled consciences of a few people, are only undertaken as a final check, after all the dimensions have been established in the laboratory.

Future Prospects

It is this method which has enabled us to go further and further in the use of wide valleys - even valleys wide at their bases - without any fear of excessive deflections in the central cantilevers, as in the cases of Pievo de Cadore, Kariba.

There is no doubt at all that it will enable us to go yet further in this direction, though we had hoped to be able to rely on calculations alone for designing a particularly long but low dam, with two hinges: Moulin Ribou (Fig. 15).

In the case of Roselend arch dam (Fig. 16), apart from some rough calculations on plunging arches based on a method I inaugurated about twenty-five years ago, the work was completely based on models. The principles of this structure are now well known and can be summed up as follows:

The arch rises high up above the gorge and it comprises two parts composing a single monolith - the lower part is a standard arch, with the usual abutments on the banks; above this there is an arch without any abutments, but sloping crosswise along a long oblique line. Here the plunging arch effects draw down to the foundations the thrust of the water and the dead weight, as if the abutments were somehow incorporated in the arch itself.

Another factor of progress for arch dams is the excellent quality and performances of modern concrete, such as large aggregate concrete and specially gap-grading concrete, containing up to 60 per cent of 100 to 250 mm (4 in to 10 in) aggregate, the strength of which, at one year old, attains more than 350 kg/cm2 (5,000 lb/in2) at moderate cement contents (220 kgs per cubic meter) (370 lb per cu. yd.). This allows for working stresses exceeding 100 kg/cm2 (1,400 lb/in2) for the usual applications.

The progress already accomplished in multiple arch dams, which share the advantages of pure arches, and the great future opening out before these structures, deserve special mention. Some designers in Europe and Africa, who are up against the problem of saving materials because of transport difficulties and expensive cement, and still having the advantage of relatively cheap labour, still favour this type of dam.

The best example is that of Oued Mellègue (Fig. 17), completed recently. I hope, however, to go one better very shortly.

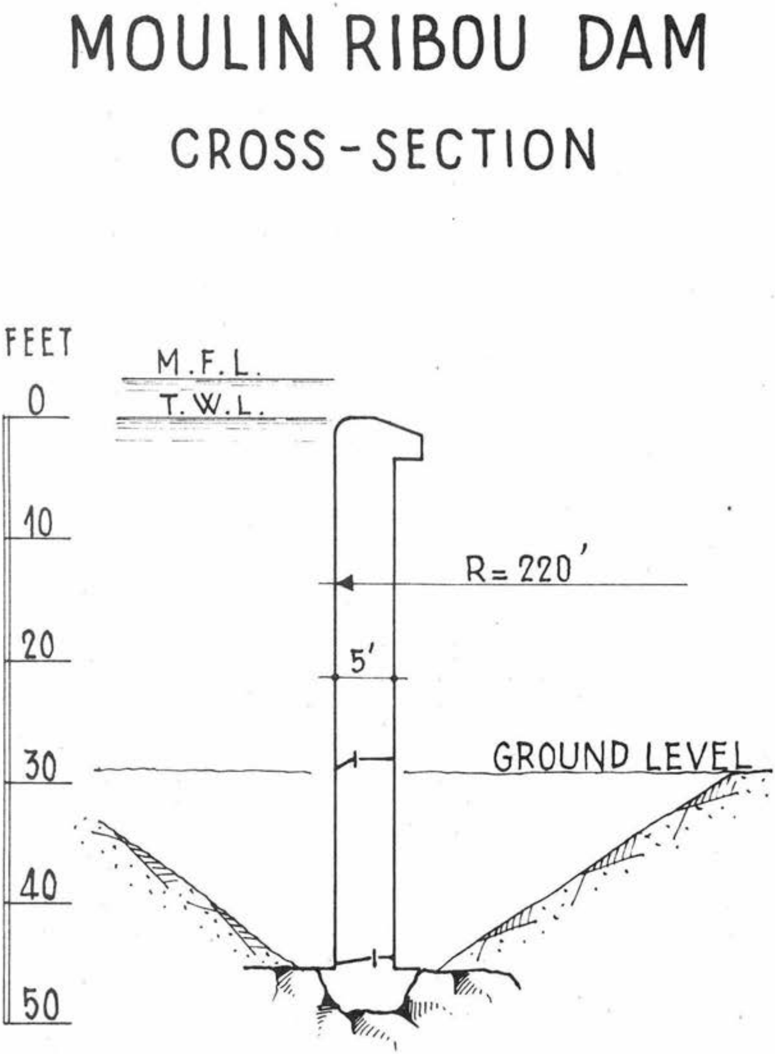

Fig. 15b
Moulin Ribou Dam
Cross-Section

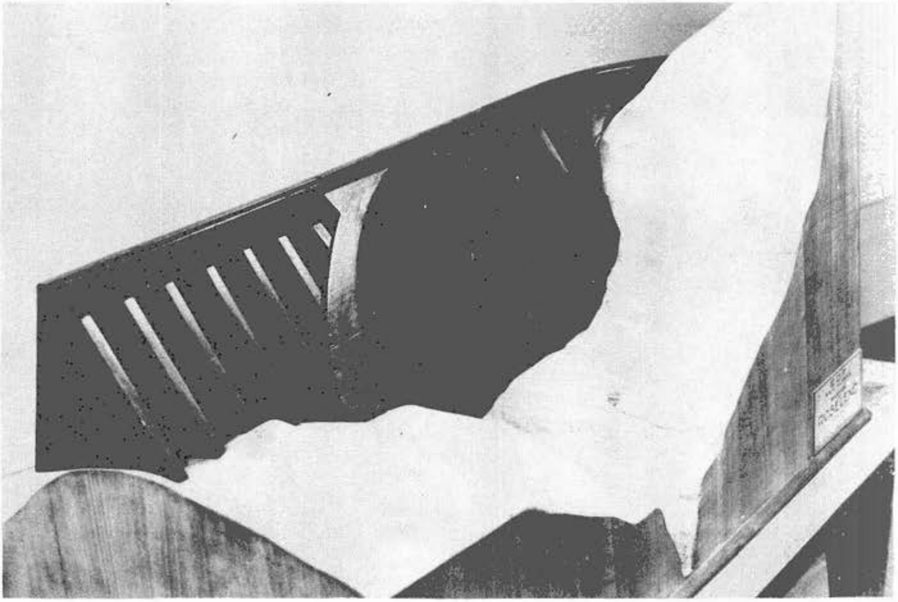

Fig. 16
Model of Roselend Dam

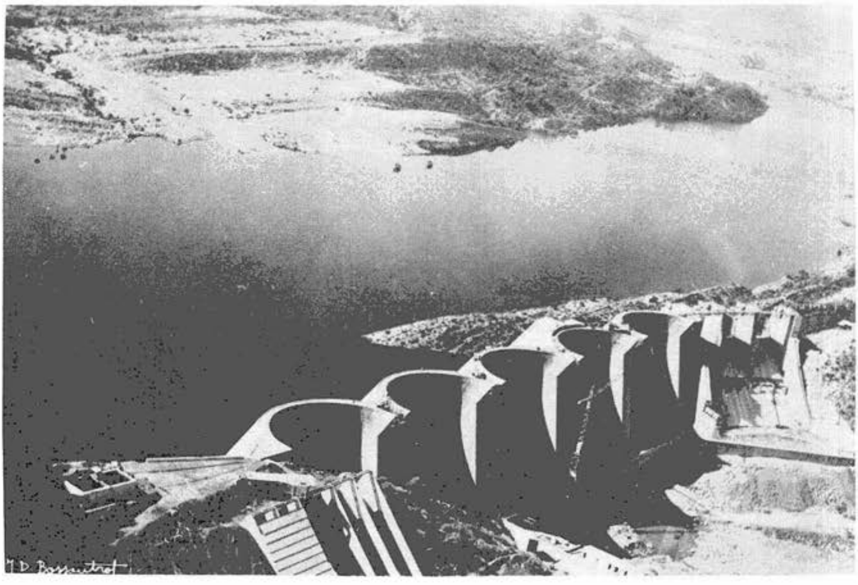

Fig. 17
Oued Mellègue Dam
(Photograph by: J. D. Bossoutrot, Tunis)

Discussion of "ARCH DAMS: THEIR PHILOSOPHY"

by Andre Coyne
(Proc. Paper 959)

G. S. SARKARIA,[1] J.M., ASCE.—That the arch dam, though ancient in concept, is yet in an active stage of development at the present time is the main theme of this interesting paper by M. Coyne. Studying this and the other papers presented at the A.S.C.E. Symposium on Arch Dams, June 1956, the writer is impressed by the diversity of methods of design, analysis, and experimentation adopted by various engineers and organizations for obtaining arch dams most suited to their requirements.

All arch dams have one property in common: curvature. Otherwise there appear to be numerous types of arch dams which differ from each other substantially in their structural and shape characteristics. M. Coyne, for example, has named these different types in his paper: "very thin arch," dome shaped or overhanging arch, arch sloping downstream, "long but low dam, with two hinges," and multiple arch. Other authors have used terms like single-curvature(1) and double-curvature(2) arch dams. Terms like arch-gravity, variable-thickness arch and constant-angle arch dams are of more common usage. There are other arch dams with ungrouted radial contraction joints, an ungrouted "perimetral joint,"(3) or a "horizontal sliding joint."(4)

The writer believes it is desirable to classify various types of arch dams in order to appreciate the different designs discussed by M. Coyne and other contributers to the Symposium. The two major classifications are:

1. Shape Classification: Terms such as constant-thickness, variable-thickness and constant-angle refer to the shape of the arch. Other designations such as single-curvature, double-curvature, dome or overhanging type signify the shape characteristics of the dam in two directions.

2. Structural Classification: Arch dams with horizontal or peripheral sliding or keyed joints, and those with radial contraction joints keyed and/or grouted, represent distinctly different structural types.

It is necessary to adopt a structural classification that will clearly distinguish between different types of structures. The examination of merits and suitability of the different structural types to a particular situation should form as much a part of design and analyses as the adoption of the final arch dimensions. The writer proposes the following preliminary structural classification:

a. Monolithic Arch Dams: This type includes arch dams in which all contraction joints, whether horizontal, vertical, or peripheral are grouted.

b. Simple Arch Dams: Arch dams that are designed as horizontal elastic arch slices fixed at the abutments and where a horizontal sliding joint is provided between the dam and its base. Such dams are not three-

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dimensional monolithic structures, since bending moments can not be transferred across the sliding joint to the base of the dam. Matilija Dam(4) in California is an example of this type.

c. Hinged Arch Dams: Arch dams that have vertical (radial) contraction joint that are keyed but not sealed with grout, are hinged structures. Dams with peripheral hinged joints, such as Osiglietta Dam(3) in Italy, should also be classified as hinged arch structures. The main advantage(3) claimed for hinged arch dams is, "the stresses in the structure were lower and better distributed."

d. Cantilever Arch Dams: Such arch dams have radial (vertical) contraction joints that are neither keyed nor grouted. The arch voussoirs are vertical cantilevers that wedge against each other when subjected to reservoir water pressure.

M. Coyne has mentioned that progress in the design of arch dams is dependent upon factors like shape of the dam-site, stiffness of the arch, the allowable average stresses, and the quality and performance of concrete. Comparing the various designs of arch dams described in the papers submitted for the Symposium, the writer has no doubt that many other engineers like himself must be asking the question "Why is there so much difference between the various designs?" The writer feels that an analysis of the factors that influence the design of arch dams should help to answer the above question to some extent. Some of these major factors are:

a. Shape of canyon at dam-site.
b. Structural type of arch dam.
c. Height of the structure.
d. Forces acting against the dam, and the competence of the foundation and abutments.
e. Allowable stresses.
f. Methods of design and analysis.

These factors are discussed in brief in the following paragraphs.

Canyon Shape at Dam-site:

It has long been a matter of common belief amongst dam designers that economical arch dams cannot be built at a site where the crest length to maximum height ratio exceeds 5. M. Coyne states that the use of structural models "has enabled us to go further and further in the use of wide valleys—even valleys wide at their bases—without any fear of excessive deflections ...." The design of an arch dam is affected not only by the crest length to height ratio but also by the shape of the canyon and length of the peripheral contact. In order to systematically investigate influence of canyon shape, it is necessary that a standard classification of canyon shapes is referred to by all designers. In collaboration with Mr. F. D. Kirn, the writer has proposed(5) a classification, which if adopted, should eliminate the confusion caused by references to general terms like wide, narrow, and U-shaped canyons. This classification is shown in the writer's Fig. (1).

An effort was also made to establish a simple yet appropriate criterion based on shape of canyon at a dam-site, that would indicate the suitability or otherwise of the site for an arch dam. A "canyon-shape factor" defined as the ratio of the foundation and abutment perimeter to the maximum height of the dam, was therefore proposed.(5) Refering to the writer's Fig. (2), the canyon-shape factor is:

$$K = \frac{b + H (\sec \psi_1 + \sec \psi_2}{H}$$

The notations are explained in the sketch.

Two dam-sites having the same crest-length to height ratio are likely to have different shape characteristics. The two canyons shown in the writer's Fig. (3) have the same B to H ratio, but Profile I and II have canyon-shape factors of 4.5 and 5.3 respectively. The difference between the canyon-shape factors indicates that an arch dam for Profile II would be much more massive than the one for Profile I, provided both the dams are designed according to the same criteria and by the same method.

To illustrate this point further, canyon profiles and crown cantilever sections of Hungry Horse Dam and the proposed Yellowtail Dam(5) are shown in the writer's Fig. (4). Hungry Horse site has a wide V profile and its canyon-shape factor computed from the developed profile is 4.6. Yellow tail dam-site has a composite U-V shaped canyon with comparatively steeper abutment slopes, and its canyon-shape factor is 3.5. Both these dams were designed by the U. S. Bureau of Reclamation using the trial load method. The loading conditions and design criteria were reasonably similar. The influence of canyon shape is dramatically evident from comparison of the crown cantilever sections for the two dams.

Canyon-shape data for 20 dams is presented in the writer's Table I. It includes both arch and straight-gravity dams. For arch dams developed profiles along arch center lines were used in obtaining canyon-shape factors. The writer has also included some of the dams described by M. Coyne and other contributers to the Symposium. Undoubtedly some of the data are approximate as sufficient details are not available to the writer.

Comparing the canyon-shape factors for the arch and gravity dams listed in Table I, it should be noted that for arch dams, canyon-shape factors vary between 2.2 and 4.6, the average being 3.4. Canyon-shape factor for Kariba Dam proposed by M. Coyne is 6. Since the site for this dam is exceptionally wide compared to those conventionally considered suitable for arch dams, its canyon-shape factor is not considered representative. Canyon-shape factors for most of the gravity dams are well above 6. If any gravity dams have been built in canyons with shape factors less than 5, it is presumed that considerations other than shape of site profile must have disfavoured the adoption of an arch dam.

From this empirical comparison it can be concluded that sites with canyon-shape factors greater than 5 are unsuitable for building economical arch dams. The author, however, proposes arch dams for sites which would be considered too wide by most designers and may, therefore, change the above empirical limit of canyon-shape factor for arch dams.

Structural Types and Height:

The writer has already briefly discussed the desirability of classifying arch dams according to their designed and built-in structural characteristics. Height of dam is also a common denominator in all types of arch dams, and all schools of thought feel that arch dams are the type that can be economically built to heights for which gravity or earth may not be feasible.

Methods of Design, Allowable Stresses, and Forces Acting on Dam:

The author has mentioned, although briefly, three or four current design concepts and methods. He shares with many other designers the dread of "extreme complexity" of elaborate analytical methods. So he suggests that

> "the best thing to do is to put simplified calculations, giving results within two limits and to get from them the courage for making up one's mind for choosing a given design. This is then immediately elaborated on a model. . . Today, over-exacting calculations, should they be necessary, particularly for calming the troubled consciences of a few people, are undertaken as a final check, after all the dimensions have been established in the laboratory."

The writer finds this statement rather paradoxical. M. Coyne does not discredit "over-exacting" analytical calculations as incorrect, for to satisfy "some troubled consciences," if necessary, he does consider analytical methods good enough to check the finished product of laboratory experiments. Without meaning to minimise the importance of structural models, the writer believes it is necessary to deprecate the tendency amongst many designers to reject detailed analytical analyses as "lengthy," "cumbersome," time-consuming, and expensive.

Arch dam design will greatly benefit if reasonably exact analytical methods and not too "over-exact" structural model tests compliment each other. After all, analytical methods and scale models often work under similar, though occasionally unrealistic, design assumptions. Analytical methods can often be modified and shortened within similar limits of accuracy, to be economically comparable with model tests.

The controversy as to which method of design is better can be detected as an undercurrent in many of the papers submitted to the Symposium. Often, it is not the method of design, but the design forces, allowable stresses, and the quality of concrete that determine the section finally adopted for an arch dam. Some design criteria require that no tensile stresses should occur in the structure, anywhere. Describing the very thin Le Gage Dam, M. Coyne states "in some places the concrete freed itself from excessive extension by cracking, without any serious consequences." Also, "experience proves that extension and even cracks in arch dams are not dangerous." Evidently it is a matter of judgment as to how much tension and cracking can be safely allowed in an arch dam, and on this may depend the adopted thickness of the dam.

M. Coyne also points out the improvement in allowable average stresses in concrete over a period of years. This is another factor that should not be lost sight of when comparing two dams designed by different methods. For example, it would not be correct to compare the sections of Hungry Horse Dam(6) where the maximum allowable compressive stress is 750 lb/in^2 against those for Malpasset Dam for which the allowable stresses are of the order ot 850 to 1000 lb/in^2 as mentioned by the author. All other factors being comparable, this disparity in allowable stresses alone would indicate a proportional difference in the sections.

Having described some of the factors that should not be ignored in comparing various types of arch dams and methods of design, the writer would like to express the wish that in appreciation of the healthy arguments and spirit of rivalry generated by the papers submitted by M. Coyne and others, dam designers would evolve design concepts and criteria that are a compromise between the various points of view and thus useful and acceptable to apparently opposing viewpoints.

TABLE I

CANYON-SHAPE CHARACTERISTICS OF MASONRY DAMS

No.	Dam	Location	Max Height in feet	Shape of Canyon	Canyon-shape Factor K	Remarks (Profile)
(a) ARCH DAMS						
1.	Arrowrock	U.S.A.	350	Composite U-V	4.3	Symmetrical
2.	Buffalo Bill	U.S.A.	325	U	2.2	
3.	Cabril	Portugal	442	Wide V	3.5	Unsymmetrical
4.	Castelo Do Bode	Portugal	378	Composite U-V	3.4	
5.	Hoover	U.S.A.	726	Composite U-V	2.5	
6.	Hungry Horse	U.S.A.	564	Wide V	4.6	Unsymmetrical
7.	Kamishiiba	Japan	370	Wide V	3.9	
8.	Kariba	Africa	400	Composite U-V	6.0	Proposed
9.	Ponte Racli	Italy	164	Wide V	3.1	
10.	Ross	U.S.A.	655	Wide V	3.5	Final stage height
11.	Santa Giustina	Italy	495	U	2.4	Symmetrical
12.	Tumut Pond	Australia	265	Wide V	3.2	Proposed
13.	Venda Nova	Portugal	318	Wide V	3.5	Unsymmetrical
14.	Yellowtail	U.S.A.	520	Composite U-V	3.5	Proposed
(b) STRAIGHT GRAVITY DAMS						
15.	Bhakra	India	680	Wide V	3.4	Under construction
16.	Fontana	U.S.A.	480	Composite U-V	5.2	
17.	Friant	U.S.A.	319	Wide and flat	12.2	
18	Grand Coulee	U.S.A.	510	Wide and flat	9.5	
19.	Kortes	U.S.A.	240	Composite U-V	3.0	
20.	Norris	U.S.A.	265	Wide V	7.0	

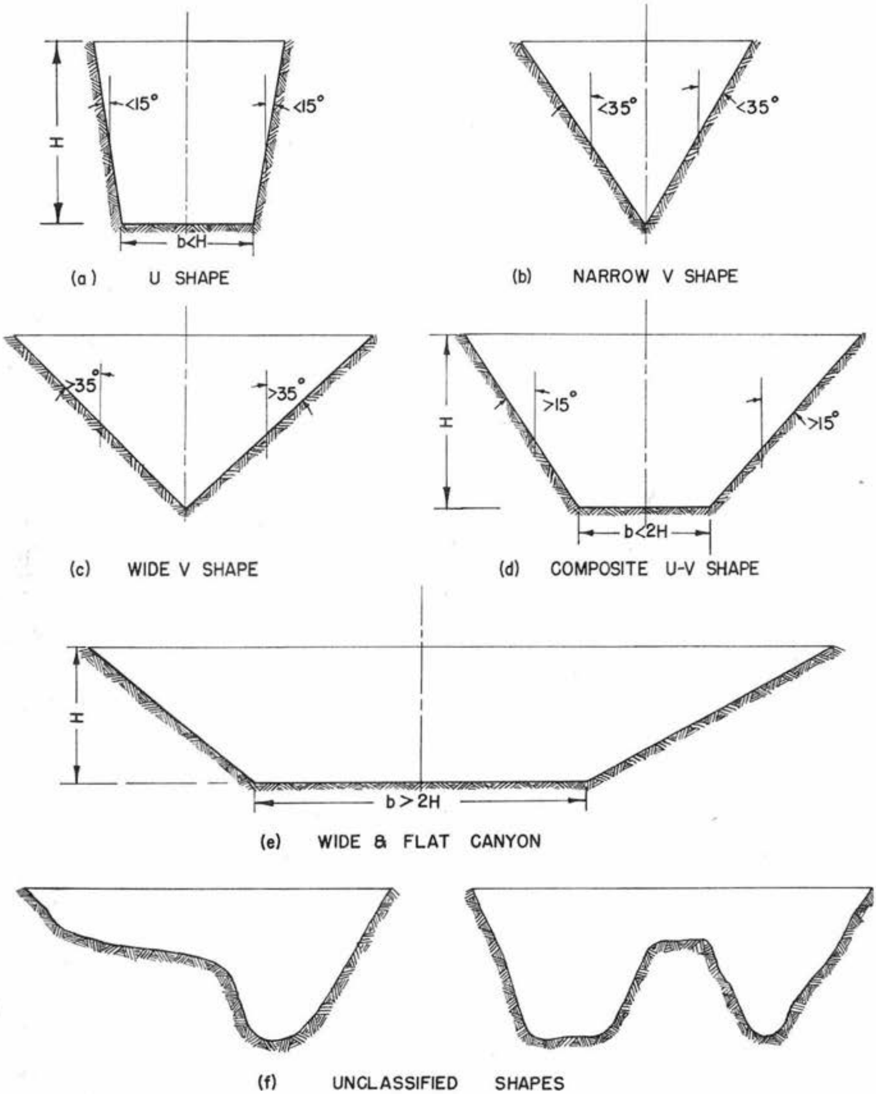

H
<15°
<15°
b<H
(a) U SHAPE
<35°
<35°
(b) NARROW V SHAPE
>35°
>35°
(c) WIDE V SHAPE
H
>15°
>15°
b<2H
(d) COMPOSITE U-V SHAPE
H
b>2H
(e) WIDE & FLAT CANYON
(f) UNCLASSIFIED SHAPES

FIG. 1

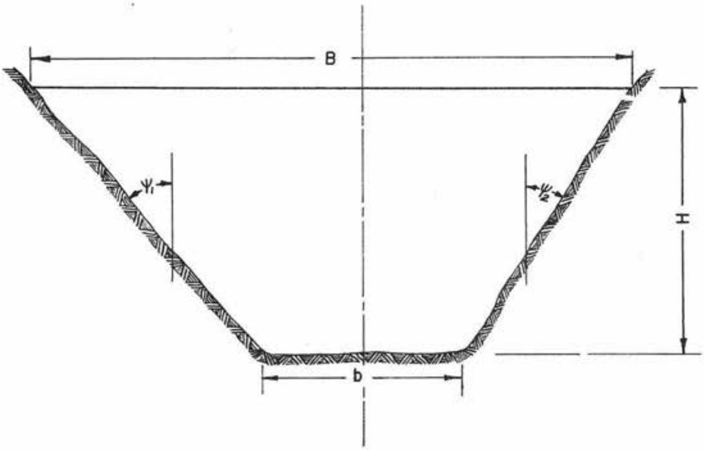

B
H
b
ψ_1
ψ_2

FIG. 2

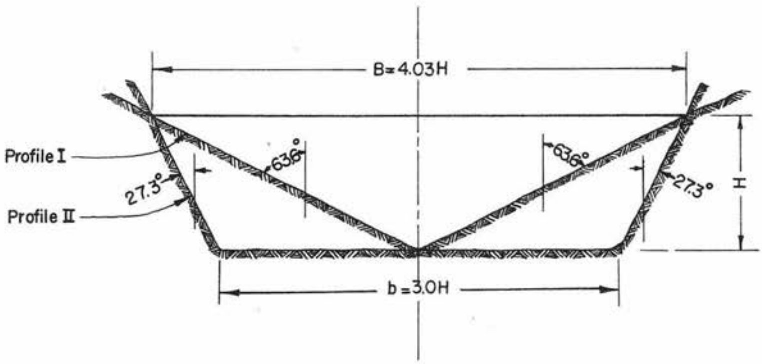
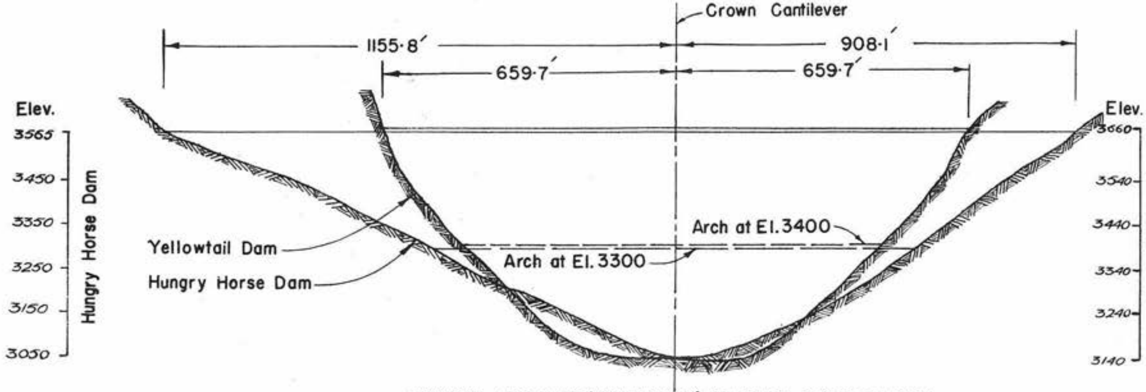

B=4.03H
Profile I
Profile II
63.6°
63.6°
27.3°
27.3°
H
b=3.0H

FIG. 3



CANYON PROFILES (DEVELOPED) ON ARCH CENTER-LINES
(Looking Downstream)

CROWN CANTILEVER SECTIONS

REFERENCES

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3. Semenza, Carlo, "Arch Dams: Development in Italy" Paper 1017, Journal of the Power Division, Proc. ASCE, June 1956, p. 1017-36.
4. Perkins, W. A., "Analysis of Arch Dams of Variable Thickness," Trans ASCE Vol. 118, 1953, p. 766.
5. Kirn, F. D. and Sarkaria, G. S., "Influence of Canyon Shape on the Design of Concrete Dams," Civil Engineering & Public Works Review, London, March 1955.
6. Glover, R. E. and Copen, M. D., "Trial Load Studies for Hungry Horse Dam." Paper 960, Journal of the Power Division, Proc. ASCE, April 1956.

GEORGE E. GOODALL,* M. ASCE.—The author properly begins his very interesting paper with a brief mention of 16th Century arch dams in Spain and Ponte-Alto Dam in Italy. It seems probable that the earliest origin of arch dams reaches still further back in history. About fifteen years ago, the writer encountered an article in "Irrigacion en Mexico" in which it was stated that arch dams had been constructed by the Arabs in Africa before the Christian Era.

The development of arch dam designs in the United States, as well as in other countries, has undergone a constant evolution. A large part of the construction of arch dams in the United States has been done in California. Probably the oldest of this latter group was the old Bear Valley Dam. As the progress of California was so largely dependent on irrigation, a great many arch dams were constructed due to the inability of local interests to finance more costly types of dams for water storage. As a consequence, by the time the California Legislature passed an act in 1929 placing all dams in the State under the jurisdiction of the State Engineer, there were more than eighty single arch dams and twenty-two multiple arch dams in existence. The writer spent more than three years in the office of the State Engineer, charged with the responsibility of analyzing the dams in both of these groups. It was evident that older arch dams had been analyzed solely by the "cylinder" formula. The engineers responsible for these designs, realizing the shortcomings and inaccuracies of the "cylinder" formula, limited "cylinder" stresses to about 300 p.s.i. With the aid of the Cain formulae,[1] augmented by the later work of B. F. Jacobson,[2] Dr. Vogt,[3] and others, newer designs were made in which the "cylinder" formula was used only for a rough preliminary with the result that higher cylinder stresses became the rule.

In the early days of the California State Supervision of Dams, maximum

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* Cons. Civ. Engr., Sacramento, Calif.
1. Trans. ASCE, Vol. LXXXV, p. 233.
2. Trans. ASCE, Vol. 90, p. 475.
3. Trans. ASCE, Vol. 93, p. 1272.

arch stresses, as computed by the Cain Formulae, were limited to 600 p.s.i. in compression, but tensile stresses not to exceed 100 p.s.i., even though occuring at contraction joints, were permissible. In more recent years, these limitations have been modified and in the case of Donnell's Dam, now under construction on the Stanislaus River, California, the maximum compressive stress, as calculated by the Vogt Formulae, is 850 p.s.i., including the effects of a 20 degree F. temperature drop. In adapting the Vogt Formulae for this work, the modulus of elasticity of the abutment rock was taken as 1.5 times the modulus of elasticity of the concrete. Inasmuch as the effect of abutment deformation is negligible for arches having a small ratio of t/r, the arches in the upper portion of the dam would show the highest stresses as computed by the "cylinder" formula. In this specific instance the maximum "cylinder" stress is 766 p.s.i., 77' below crest of dam. From this elevation down toward the base, "cylinder" stresses progressively decrease. Donnell's Dam,[4] when completed, will be 485' high and due to the great economy of the arch dam, will cost slightly under 10-million dollars.

The author's statement that "because an arch is curved, the difficulty would seem to be not to make it hold up, but to knock it down" is not surprising to anyone who has had considerable experience with many of the existing older arch dams. The old Bear Valley Dam and Upper Otay Dam, both in California, which have long been considered classic examples of boldness, would seem to have had considerable influence in the author's statement quoted above. Two of the older arch dams in California had arches that were not even circular in plan but rather roughly in the shape of a spiral. They have no axis of symmetry although the loads imposed thereon are symmetrical. However, they have been in service for many years. Another example, if any be needed to back up the author's statement, is one arch dam 178' in height, whereat the actual excavation contours for the lower half of the height of the dam diverge in a down stream direction. Yet this dam in the disasterous flood of December, 1937, was overtopped by 16.2' and did not fail.

Figure 1 shows plan and abutment details of an arch dam in California approximately 50' in height which has no abutment whatsoever at the right end of the arch. The unbalanced "cylinder" thrust at the end of the arch is 4,650,000 pounds. The spillway is inadequate and the entire arch has been overtopped, yet this structure still stands.

The writer does not mention these obviously horrible examples as something that should be repeated or even allowed to influence design in any way. These have been mentioned merely to reinforce, if any such reinforcement be needed, the author's contention that the arch inherently is a very safe structure. His reference to an experimental dam model which failed under "cylinder" stress of 4300 p.s.i. adds further weight to the reported test data on models performed at Lake Cushman Dam.[5]

Where topographic and geologic considerations are favorable, there can be no question as to the economy of the arch dam. In the case of Donnell's Dam, when the site was first explored, a competent geologist estimated that the depth of alluvial fill in the stream channel at the site would be in excess of 75'. Preliminary designs and estimates showed that a rock fill dam would cost in excess of $1,000,000 more than a concrete arch. Diamond drilling of the site showed that the depth of the alluvium was about 200' where the arch

4. Engineering-News Record, August 16, 1956, p. 42.
5. Trans. ASCE, Vol. 90, p. 553.

dam crossed the channel section. It is obvious that excavation through this depth of alluvium for any type of dam other than a thin arch dam, would have been so costly as to have rendered the entire project uneconomical.

Much has been written in the past thirty-five years about the calculation of stresses in an arch dam. The farther one goes, the more complex the analyses become and the claims of accuracy of stress determination seem to grow in direct ratio to complexities of the mathematics involved. It has been well stated by Dr. Fredrik Vogt that "a rough approximation based on correct assumptions with regard to shrinkage, temperature changes, yielding of foundation, etc., is more valuable than a highly refined computation based on incorrect assumptions.[6] In none of the papers expounding methods of stress determination has the writer even seen an attempt to calculate temperature stresses in the arches of a dam which considers the effect of the non-linear temperature variations experienced throughout the thickness of any arch ring such as those measured at the Englebright (Upper Narrows) Dam.[7] Concrete placement in this dam was completed in December 1940 and the reservoir filled to spillway elevation in five days. At the end of the filling period, the Carlson strain meters embedded near the faces of the dam at the abutments in general indicated no change in stress between the conditions of no load and full load. Some years later, after the complete dissipation of the heat of hydration of the cement, measured stresses began to follow definite patterns. The meters adjacent to the up stream face were fairly stable as far as temperature was concerned and followed the center temperature. Temperatures near the down stream face varied throughout wide ranges with the atmospheric temperature, but showed a time lag with increase in depth from the face of dam. The time lag mentioned above refers to the diurnal temperature variations. The arch ring 118' below spillway has a thickness of 59.25'. For this arch ring the time lag of seasonal temperature variation appears to be about four months. After complete dissipation of the heat of hydration, maximum temperatures at the center line of the arch ring were observed to be about December 1, and minimum temperatures about May 15. The termperature 1' from the down stream face could vary any where from below 50 degrees to in excess of 90 degrees. As a result of these temperature variations, the measured stresses showed considerable variation from the calculated stresses and frequently the sign of the measured stress reversed from that of the calculated stress.

The writer has made inumerable and voluminous stress analyses of many arch dams. It is certain that many indeterminate factors can not be evaluated. Two of the most important indeterminate factors that can not be evaluated, either in stress calculations or model studies, are the non-linear temperature variations refered to above and the effect of contraction joints, grouted or not grouted. It would seem fitting to refer to conclusion to the closing statement of the late William Cain, M. ASCE, in his discussion of B. F. Jacobson's classic paper "Stresses in Thick Arches of Dams" where he wrote, "He seems well aware that an exact solution of the arch dam is not to be looked for, so that all an engineer can do is to examine the various influences and combine them to effect a practical solution."[8]

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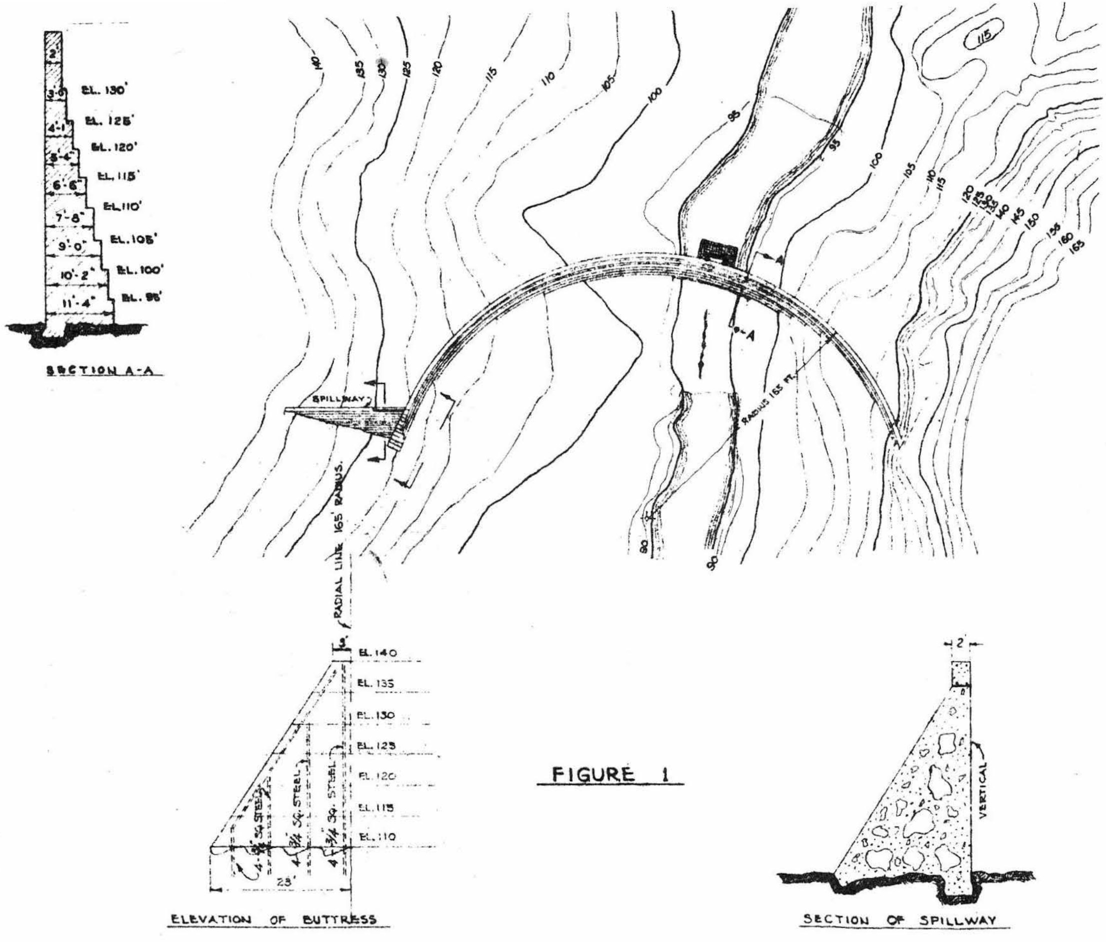
6. Trans. ASCE, Vol. 93, p. 1272.
7. Journal of the American Concrete Institute, September 19, 1947, p. 65.
8. Trans. ASCE, Vol. 90, p. 547.



SECTION A-A
EL. 130'
EL. 125'
EL. 120'
EL. 115'
EL. 110'
EL. 105'
EL. 100'
EL. 95'
SPILLWAY
RADIUS 165 FT.
RADIAL LINE 165' RADIUS.
EL. 140
EL. 135
EL. 130
EL. 125
EL. 120
EL. 115
EL. 110
3/4 SQ. STEEL
23'
ELEVATION OF BUTTRESS
VERTICAL
SECTION OF SPILLWAY

FIGURE 1

Discussion of
"ARCH DAMS: THEIR PHILOSOPHY"

by Andre Coyne
(Proc. Paper 959)

ANDRE COYNE,[1] Hon. M. ASCE.—The remarks of G. S. Sarkaria and George E. Goodall complement most auspiciously the writer's paper on the Philosophy of Arch Dams. Their closeness together gives them yet further interest, for they each cast light upon the subject in very different and even opposite ways.

The writer agrees most willingly with the first discusser, G. S. Sarkaria, that it is most desirable to tidy up the classification of arch dams.

Classification is useful—not to say necessary—both for those who teach and for those who learn, for it helps to get new data into the mind and to keep them in the memory. It forms a framework for exhibiting or for retaining the essential landmarks. From this point of view the pursuit of the simplest and most comprehensive classification therefore presents unquestionable utility. The same is true of the research, analysis and codification of the rules followed and hallowed by great builders up to the present time.

It is here the part it plays stops. A part which, it may be said consists merely in organizing the conquered territory.

Now, to judge by the number and diversity of the examples purposely proposed by the writer, there are still territories awaiting conquest, which will not fail one day to tempt some innovating mind.

In the technical sphere conquest, the realisation of progress, means being able or having the courage to do that which has not as yet been done. It means emerging from the rigid conventional framework or at least extending it, even bursting forth from it, in a word, ceasing to consider it an imprescriptible law of the Medes and Persians. This is of course the idea of the discusser, when he concedes that the present empirical limit of the canyon-shape factor for arch dams can be changed in the future.

Far from "not being considered representative" today's exception, Kariba for instance, can become tomorrow's rule.

This is the reason why the writer himself has always shared the opinion that as things are at present, formal and imperative regulations do more harm than good in the sphere of arch dams, particularly where the conditions of adaptation to the site, methods of calculation and permissible stresses—specially tensile stresses—are concerned.

This all the more if, as the discusser suggests, such criteria result "from a compromise between the various points of view."

On the contrary, the greater the part given to diversity of individual inspiration the better the result, taking into account the variety of sites and working potentialities which the earth offers unceasingly to designers.

Doing without the impassable barriers built up by the regulations boils down to relying principally upon the gifts for observation, the know-how and

1. Inspector General, Cons. Engr., A. Coyne and J. Bellier, Paris, France.

and the good judgment of designers; a thing which is only apparently dangerous for no ruling on construction has ever prevented people from committing accidentally fatal fundamental mistakes.

It can even be maintained without being paradoxical that the illusion of being covered by regulations and exaggerated attachment to tradition can hamper designers in establishing correct diagnoses by inhibiting their capacity and even their desire for apprehending reality. This is the major risk. Precedent has been called humourously the momentum of an early start in a wrong direction.

G. E. Goodall on his side, recalls in a timely way the vigour and fecundity of the great current of pragmatism started in the United States of America by the first arch dam builders under the empire of "holy necessity," that is to say lack of money.

The writer is particularly grateful to him for bringing grist to his mill by quoting fresh examples which the discusser declares to be "obviously horrible," of arch dams which have stood up obstinately to many kinds of causes of destruction, of which some could incidentally, have been obviated.

These examples again, have to be interpreted with care, in the light of modern knowledge of the mechanical behaviour of such structures and the quality of their materials. Here again, the "sixth sense" of diagnosis, the wisdom and good judgment of the designer are the decisive and irreplaceable factors. This is one of the grandeurs of his profession.

The writer associates himself to the full with the very sensible remarks made by the discusser and by Dr. Frederik Vogt on the proper use of calculations, and on the differences revealed by experience between actual and computed stresses. It is for this reason that he persists in believing that it is from the experimental method rather than from analysis that fresh progress can be expected in a sphere which is in full evolution.

The writer was not aware that traces of arch dams built in Africa before the Christian Era had been found. Should such relics exist, they ought not to be attributed to the Arabs, who only started invading Africa in the VIIth Century A.D.

Henri Goblot, a French Engineer residing in Iran, has just informed the writer of his discovery near the town of Koum, of an old masonry dam, 26 m high, curved or rather polygonal in plan. It is well-preserved but now empty. According to local information it was probably built in the middle of the thirteenth century.

Although its shape is not perfect this dam is certainly one of the ancestors of arch dams, the oldest we know up to date.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: TRIAL LOAD STUDIES FOR HUNGRY HORSE DAM

R. E. Glover,[1] M. ASCE, and Merlin D. Copen[2]
(Proc. Paper 960)

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June 1956 at Knoxville, Tennessee.

Since the last symposium on masonry dams (April 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be printed from the Symposium. So far, two papers have been approved: "Arch Dams--Their Philosophy", (Proc. Paper 959) by Andre Coyne, Hon. M. ASCE, and "Arch Dams: Trial Load Studies for Hungry Horse Dam," (Proc. Paper 960) by Robert E. Glover, M. ASCE, and Merlin D. Copen.

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

SYNOPSIS

A brief history of the development of the Trial Load method for stress analysis of arch dams is first given and the application of these procedures to the design of the Hungry Horse Dam is then described in detail. The Kirchhoff uniqueness theorem of the Theory of Elasticity is used to show that the radial, tangential and twist adjustments of a complete Trial Load analysis are adequate to meet all the requirements for a correct stress analysis of the structure under the loading conditions assumed. However, local stress concentrations may profit from study by photoelastic means and the results of strain gaging may indicate that some factors should be included which are now ignored. Stress distributions as obtained by a radial adjustment only and by

Note: Discussion open until September 1, 1956. Paper 960 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 2, April, 1956.

1. Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo. Retired.

2. Engr., Concrete Dams Section, Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

a complete Trial Load analysis are compared. An analysis based upon the abutment configuration of the dam, as built, is compared to the results of the design analysis based upon the abutment configuration developed from the preliminary investigations at the site to show the influence of such changes upon the maximum stress in the dam.

Development of the Trial Load Method at the Bureau of Reclamation

Before describing the results of the stress computations for the Hungry Horse Dam it may be well to trace briefly, the development and characteristics of the method by which they were obtained.

The U. S. Reclamation Service, which was the predecessor of the present Bureau of Reclamation came into existence on June 17, 1902 when the act creating it became effective. This organization began the design of arch dams by the Trial Load method almost immediately(17) and before the summer of 1905 was passed they had completed the design of two of the worlds great dams by this process. These two dams were the Pathfinder and Shoshone (now Buffalo Bill) dams in Wyoming. Because the canyons in which these two dams were to be built were very similar, one analysis was used for both of them. The analysis was made by the Consulting Engineers George Y. Wisner, M. ASCE and Edgar T. Wheeler, M. ASCE. The "Feature History of the Shoshone Dam"(3) carries the following statement: "No decision as to the type of dam was reached until the summer of 1905. At this time the late George Y. Wisner, Consulting Engineer, acting under recommendation of the Board of Engineers for the Reclamation Service, presented a report embodying a design for high and short masonry dams which was adopted for the Shoshone and Pathfinder sites. The type of structure as adopted was the result of careful study by Mr. Wisner, assisted by Mr. Edgar T. Wheeler, and the subject is covered in a report by Mr. Wisner entitled: 'Investigation of Stresses in High Masonry Dams of Short Spans' which has been published,(2) with diagrams, in the technical journals".

The account of their work(2) shows a very clear comprehension of the structural action in arch dams. They adjusted the arch and crown cantilever deflections only but recognized the need for a complete radial adjustment and computed deflections at points between the crown and abutment to investigate the remaining discrepancies at these points. They accounted for temperature changes as well as the loads produced by water pressure, and were aware of foundation deformations and twist action. Both of these dams were completed and in service by January 16, 1910 and are still in excellent condition. At the time it was completed, the Shoshone Dam was the highest in the world. The next advance was made by the Bureau Engineers Julian Hinds, M. ASCE, C. H. Howell, M. ASCE and A. C. Jaquith who developed procedures for making a complete radial adjustment.(7) The Gibson Dam in Montana was the first to be designed on this basis. Shortly thereafter the senior author added the twist and tangential adjustments and when the model testing work organized by Mr. I. E. Houk, M. ASCE, had produced deflection measurements on the model of Gibson Dam, an analysis was made(9) to determine whether the Trial-Load procedure, as then developed, was adequate. It was found that the twist and tangential adjustments were needed and that when these were added a close agreement between observed and computed displacements was obtained.(9) This result was confirmed by later model testing.(14) In making these computations, foundation deformations were accounted for by using the formulas developed by Dr. Fredrik Vogt,[5] M. ASCE and by noting the peculiar boundary condition at the top of the dam[1-11] which was brought to our attention by Dr. H. M. Westergaard, M. ASCE.[21] The Owyhee Dam, in Oregon, was the first dam to be designed by the Trial Load procedure which included twist and tangential adjustments. The computation procedures were later improved by R. S. Lieurance who replaced summations with integrals. These were tabulated[12] by Mr. F. D. Kirn using series formulas[11-12] devised for this purpose.[12] The additional water pressures applied to dams during earthquakes[8] were treated by Dr. H. M. Westergaard. The mathematical theory of the conduction of heat in solids was used for preparation charts for temperature changes in the concrete due to external temperature changes[10] and for cooling of concrete dams by embedded pipes.[15] If an arch ring does not meet the abutment along a radial line the arch is considered to terminate at a radial line and the deformations of the triangular portion of concrete lying between this radial line and the actual abutment line are included by using formulas developed by Dr. Westergaard from strain-energy considerations. All of these developments found application in the design of the Hungry Horse Dam.

When the radial, tangential and twist adjustments are made the analysis satisfies the Kirchhoff uniqueness theorem[6] of the Theory of Elasticity as nearly as it is possible to do so with an analysis based upon prismatic elements which extend through the dam from the upstream to the downstream face.[11] Such an analysis may be said to be complete. The Trial Load method shares this type of element with the ordinary slab and thin shell theories.

If the prismatic element described above is further subdivided into approximately cubical elements cut out by surfaces normal to a radius it will be found that continuity conditions in thick dams are not completely satisfied near the upstream and downstream faces even though the adjustments are perfectly made at the midpoint of the prism. In Bureau practice it has been customary to use special means[13] to account for the departures from the linearity of stress distribution postulated for the prismatic element. Photoelastic methods should also be effective for these purposes. The way in which the three adjustments account for the three displacements and the three rotations experienced by the prismatic element as the dam passes from the unstrained to the strained state is shown in the following table:

Table I

Effects of the adjustments

Adjustment	Radial displacement	Tangential displacement	Vertical displacement	Rotation about the radius	Rotation about the tangent	Rotation about the vertical
Radial	*					
Tangential		*	*	*		
Twist					*	*

The Kirchhoff uniqueness theorem not only gives assurance that there can be but one stress system for any given load situation but tells by implication, what conditions must be met if this unique stress system is to be found. These conditions are the following: (1) The condition of equilibrium must be satisfied everywhere. (2) The condition of continuity must be satisfied throughout the structure and (3) the boundary conditions must be satisfied. The Trial Load method meets these requirements through the three adjustments described in Table I. This method is therefore complete in the sense that no other adjustments are required. The purpose of a complete Trial Load analysis is to find that distribution of stress which must be present in the dam under the assumed conditions of use. It is not a short cut method in any sense and its use will require an expenditure of considerable time and effort. However, it has advantages as a design tool for large arch dams where sound economy, and the safety of lives and investments are paramount.

If these purposes are to be attained, care must be taken to see that the actual conditions are imposed upon the analysis. It is especially necessary to scrutinize all stereotyped assumptions and to discard them if they do not accord with the facts. The shortcomings of one of these were disclosed by the strain-meter data obtained from the Shasta Dam.(18) These measurements showed conclusively that the stresses are strongly influenced by the construction procedure in spite of the fact that accepted design methods generally ignored these factors. It will be noted in the descriptions which follow that a careful accounting was made in the Hungry Horse Trial Load studies for effects of the construction procedure. Of particular importance were the load temperature and structural conditions at the times of grouting. Cooling by embedded pipes was used to bring the concrete temperature to a predetermined level before the grouting in each stage was done.

It may be well to point out that strain gage data from dams in service will not generally conform to the results of a study such as has been described. The reason for this is that the designer generally chooses a maximum possible set of loadings as a basis for his design whereas the service conditions will generally be less severe. In the present case maximum water loadings, temperature, earthquake and ice loadings were used as a design basis but, in service, water levels will generally be less than maximum, ice may not be present and earthquakes will seldom occur. It should not be surprising, therefore, if the strain gage data indicate stresses differing from those obtained from the Trial Load analyses. Another factor which influences the stresses in dams is temperature change. The temperature changes associated with the march of the seasons affect the temperatures of the concrete throughout the thinner dams and penetrate to depths of about 50 feet in the thicker dams. The daily changes and the erratic fluctuations caused by changes of the weather are important near the surface but do not penetrate as deeply as the yearly changes. Potentially, these changes are capable of producing stresses comparable to those due to the water load. In the present design allowances were made for changes in the mean temperature of the arch rings but no accounting was made for variable temperature distributions through the arch rings. It is possible that the results of strain-gagings will show that this factor is of sufficient importance so that some accounting should be included for it in the stress analysis.

Temperature Control

In order to obtain the most effective structural action in an arch dam it is desirable to do the grouting, which makes it behave as a structural entity, at a time when the mean temperature is at its lowest point in the yearly cycle of temperature change.[10] If the dam is so large that it will not come to thermal stability with its surroundings during the construction period then an embedded pipe cooling system may be used to bring the concrete temperatures under control.[15] At the Hungry Horse site river water was used for cooling and the temperatures were brought down somewhat below the estimated final configuration to improve structural action in the dam. If the grouting is done about the first of March the condition of minimum mean temperature is usually obtained since the mean concrete temperatures lag behind the external changes by about one-eighth of a year.[10]

In those cases where winter stops the placement of concrete the exposed top surfaces of the blocks are subjected to severe temperature stresses which have a tendency to split the top of the block. If this happens the crack so formed has a tendency to propogate up through the new concrete when placement is resumed.

An innovation was tried at the Hungry Horse Dam by insulating the tops of some of the blocks with planer shavings.[16] This proved to be very effective and the concrete temperatures in the blocks so treated did not reach the freezing point during the winter. The use of thermal insulation for maintaining favorable curing conditions in concrete placed in cold weather has since proved its effectiveness on other jobs.[20]

Design Study

The Hungry Horse Dam has been constructed on the South Fork of the Flathead river, the site for the dam being about nine miles southeast of Columbia Falls, Montana. Construction was completed in 1952. The appearance of the completed dam is shown in figure 1. Figure 2 shows a general plan of the dam and a profile along the arch center-lines. Cross sections of the Cantilever elements used in the design studies are shown on figure 3.

Estimates of the thermal properties, the density and the structural and elastic properties of the concrete to be used in the dam were obtained from the Bureau Laboratories. A special study was made to determine the maximum ice thrust which might be expected to be exerted on the dam. The estimated schedule for placing concrete, temperature variations in the concrete and times of grouting the contraction joints were furnished by the Temperature Control group in the Dams Division.

The design of the dam has a uniform thickness of 35 feet along the top arch ring, a uniform thickness of 55.86 feet at elevation 3500, and a uniform thickness of 81.0 feet at elevation 3450. Below elevation 3448.5 the arch rings are variable in thickness from their crowns to the abutments. The length of the dam along its crest was estimated to be 2,060 feet corresponding to a half central angle of the top arch ring of 50 degrees. The dam, as analyzed, is 321 feet thick at the base of the crown cantilever and its maximum height was assumed to be 515 feet. Since these studies were made, however, excavations at the base of the dam have been completed and the official height of the structure has been determined to be 564 feet. This is the distance from the top of the dam to the lowest point in the foundation for which mass concrete prices apply. The effect of these changes was evaluated by a supplemental Trial Load study, to be described later, which also accounted for the actual construction procedure.

Basic Design Data Used in Analyses

Trial Load stress studies made for the proposed design of Hungry Horse Dam were based on the following design data:

a. Crest of dam, elevation 3565.
b. Base of crown cantilever, elevation 3050.
c. Normal high water surface, elevation 3560.
d. Reservoir water surface during times of maximum drawdown, elevation 3250.
e. Increase in horizontal pressure due to silt accumulations, if any, were not included in the analyses.
f. Effects of tailwater were neglected.
g. Ice pressure, 5 tons per linear foot.
h. Thickness of ice sheet, 2.25 feet.
i. Maximum horizontal earthquake assumed to have an acceleration of one-tenth of gravity, a period of vibration of one second, and a direction of vibration normal to the axis of the dam at the line of centers.
j. Effects of vertical earthquake were not included in the analyses.
k. Sustained modulus of elasticity of concrete in tension and compression, 3,000,000 lb/in^2.
l. Sustained modulus of elasticity of foundation and abutment rock 3,000,000 lb/in^2.
m. Modulus of elasticity of concrete in shear, 1,250,000 lb/in^2 reduced to 1,000,000 lb/in^2 in calculating the detrusions caused by radial shears to allow for the nonlinear distribution of shearing stresses between the upstream and downstream faces of the dam.
n. Poisson's ratio for concrete, 0.20.
o. Poisson's ratio for foundation and abutment rock, 0.20.
p. Unit weight of concrete, 150 pounds per cubic foot.
q. Unit weight of water, 62.5 pounds per cubic foot.
r. Coefficient of thermal expansion of concrete 0.000,005,97 feet per foot per degree Fahrenheit.

Basic Assumptions Used in Analyses

The following basic assumptions were used in the Trial Load analyses:

a. For purpose of making the analyses, the arch rings above elevation 3400 were assumed to have radial abutments and below this elevation they were assumed to have triangular abutments.
b. Arch rings above elevation 3400 are symmetrical about their crowns and are nonsymmetrically loaded. Below elevation 3400, the arch rings are nonsymmetrical about their crowns and are nonsymmetrically loaded.
c. Foundation and abutment rock formations at the site of Hungry Horse Dam have adequate strength to safely carry the loads transmitted by the dam.
d. The concrete in the dam will be homogeneous, uniformly elastic in all directions, and strong enough to carry the applied loads with stresses well below the elastic limit.

e. The dam will be thoroughly keyed into the foundation and abutment rock throughout its contact with the canyon profile so that arches may be considered as fixed with relation to the abutments, and cantilevers as fixed with relation to the foundation.

f. Contraction joints in the dam will be thoroughly grouted according to the grouting schedule and it is assumed that these joints will remain grouted during the life of the structure.

Loading Conditions and Studies Made

Stresses determined from the Trial Load studies include effects due to the construction and grouting programs, horizontal earthquake, ice pressure and cooling of the concrete prior to grouting the contraction joints. Assumptions, loading conditions, and analyses made to include these effects were as follows:

A. Stage 1

(a) Dam built to elevation 3300.
(b) Contraction joints from foundation to elevation 3300 grouted when the reservoir is empty.
(c) Analysis made: Gravity analysis, dead load carried by cantilevers.

B. Stage 2

(a) Crest of dam raised from elevation 3300 to elevation 3440.
(b) Reservoir water surface raised to elevation 3325.
(c) Contraction joints above elevation 3300 ungrouted.
(d) All loads carried by cantilever action above elevation 3300 and by arch and cantilever action below elevation 3300.
(e) Analysis made: Complete Trial Load analysis for portion of dam below elevation 3300. This analysis includes effects due to tangential shear and twist, the reservoir water load to elevation 3325, and the weight of the concrete above elevation 3300. Temperatures used in the study are shown in column 1 of Table 2.

C. After the concrete in the lifts between elevations 3300 and 3400 has been cooled to the required closure temperatures it was assumed that contraction joints in this portion of the dam would be grouted when the reservoir water surface is at elevation 3325.

D. Stage 3

(a) Crest of dam raised from elevation 3440 to elevation 3565.
(b) Reservoir water surface raised from elevation 3325 to elevation 3425.
(c) Contraction joints above elevation 3400 ungrouted.
(d) All loads carried by cantilever action above elevation 3400 and by arch and cantilever action below elevation 3400.
(e) Analysis made: Complete Trial Load analysis for portion of dam below elevation 3400. This analysis includes effects due to tangential shear and twist; raising the reservoir water surface from elevation 3325 to elevation 3425; and the weight of the concrete above elevation 3440. Concrete temperatures used in the analysis are listed in column 2 of Table 2.

E. After the concrete in the lifts between elevations 3400 and 3565 has been cooled to the required closure temperatures it was assumed that contraction joints in this portion of the dam would be grouted when the reservoir water surface is at elevation 3425.

F. Stage 4 Completed Dam

(a) Arch and cantilever action occurs in entire dam.
(b) Reservoir water surface raised from elevation 3425 to elevation 3560.
(c) Analysis made: Complete Trial Load analysis of entire dam. The analysis includes effects due to tangential shear and twist, raising the reservoir water surface from elevation 3425 to elevation 3560, horizontal earthquake and ice pressure. Concrete temperatures used in the analysis are shown in column 3 of Table 2.

G. Stresses in the completed dam were obtained by superimposing stresses calculated from the analyses listed under stages 1, 2, and 3 on the stresses determined from the analysis for stage 4.

Table 2

EFFECTIVE CONCRETE TEMPERATURES
USED IN DESIGN STUDY

Elevation	Stage 2 Temp. ° F.	Stage 3 Temp. ° F.	Stage 4 Temp. ° F.
3565			-1.4
3500			+4.3
3450			+6.0
3400		+3.3	+2.7
3350		+3.0	+2.6
3300	+4.3	+2.4	-1.4
3250	+3.1	+2.4	-0.5
3200	+3.0	+2.0	0
3150	+3.7	+2.4	-1.1

Plus sign means temperature rise.
Minus sign means temperature drop.
Temperatures in Table 2 reflect the effects of subcooling the concrete to 38° F. prior to closure of contraction joints.

Autogenous Shrinkage

Specifications for Hungry Horse Dam required that a predetermined per cent of fly ash, or a pozzolan of similar characteristics, by weight of Portland cement be used to make the concrete for the dam.

Laboratory tests, of concrete specimens containing fly ash and cement, indicated that such concrete may be subjected to autogenous shrinkage.

Autogenous shrinkage occurring in an arch dam after the joints are groutec will affect stresses in the dam in the same manner as if a temperature drop occurs in the concrete. However, in a report to the Chief Engineer by the

Board of Consultants on "The Use of Portland Pozzolan Materials in Hungry Horse Dam" dated July 29, 1948, it is stated on page 4, second paragraph that:

"Pending the proof that such autogenous shrinkage will occur with the combination of materials which are finally selected, it is recommended that any effects of autogenous shrinkage be neglected in the design calculations".

Effects of autogenous shrinkage were therefore omitted from the analyses discussed herein.

Later results(19) from the tests did indicate some autogenous shrinkage but, although there is considerable scatter among these results, it appears that this shrinkage generally came early enough so that substantial stability was reached at the times when grouting was done. The recommendation, therefore, seems to have been justified.

Radial Trial Load Analysis

The results of a Trial Load adjustment of the radial deflections only is included for purpose of comparison. Such a comparison is useful since it shows the effects of the tangential shear and twist adjustments and other refinements of the analysis on the computed stresses in the dam. The effects of construction procedures and the grouting program were not included in this analysis. This study was based upon the following conditions:

a. Reservoir water surface elevation 3560.
b. Ice sheet assumed to exert a horizontal pressure of 5 tons per linear foot at elevation 3558.75.
c. Effects of tailwater and uplift are not considered.
d. Temperature of concrete at time of grouting contraction joints assumed to be 38^{o} Fahrenheit.
e. Temperatures used in analysis are minimum stable temperatures modified by effects of sub-cooling.
f. Earthquake assumption- Dam moves upstream and downstream horizontally in the direction of the line of centers. Increased water pressure acts equally on all cantilevers. Period of vibration 1.0 second. Acceleration 0.1 gravity. Effects of vertical acceleration are not included.
g. Modulus of elasticity of concrete and abutment rock 3,000,000 pounds per square inch.
h. Poisson's ratio of concrete and abutments; 0.2.
i. Unit weight of concrete 150 pounds per cubic foot.
j. Coefficient of thermal expansion of concrete 0.000,005,97 feet per foot per degree Fahrenheit.

The stresses obtained from this study and their modification by the twist and tangential adjustments and other refinements of the complete design study are included in Tables 3 and 4. The effects of Poisson's ratio were not included in the adjustments of either of these studies.

The stresses in Table 3 act in the horizontal plane of the arch and those in Table 4 act in the vertical plane of the cantilever. However, in both cases, when the surface is inclined to the radial or horizontal planes, on which the stresses were originally computed, a factor was applied to obtain the maximum stress at the surface. This factor is the reciprocal of the square of the cosine of the angle by which the surface departs from the normal to the plane on which the stress was originally computed. Although a comparison of the values in Tables 3 and 4 will show that these direct stresses were generally decreased by the twist and tangential adjustments a further comparison should be made. Since the twist and tangential adjustments bring shearing stresses on horizontal and vertical planes into the analysis it becomes possible to compute principal stresses and the maximum principal stress, so obtained is sometimes greater than either of the direct stresses given in the Tables 3 and 4. The two greatest principal stresses obtained from the analysis which includes the tangential and twist adjustments are on the downstream face. On the left abutment a stress of 731 pounds per square inch occurs at elevation 3200. On the right abutment a stress of 724 pounds per square inch occurs at elevation 3250. The highest stress obtained from the radial adjustment analysis is 571 pounds per square inch acting horizontally at elevation 3400 on the right abutment. In this case, therefore, while the stresses on horizontal and vertical planes were generally decreased by the tangential and twist adjustments the maximum computed stress was increased. This increase comes from the shear stresses introduced by the tangential and twist action. The maximum stress obtained from the complete Trial Load study is about 22 per cent higher than the maximum stress obtained from the radial adjustment only.

The maximum compressive principal stress at the upstream face, obtained from the complete analysis, is 299 pounds per square inch and occurs at the right abutment at elevation 3150. Small tensile principal stresses of 30 pounds per square inch or less occur at elevations 3300, 3350, 3400 and 3450 at the right abutment and at elevations 3150 and 3450 at the left abutment.

A compressive stress of 750 pounds per square inch was the maximum to be permitted in the design.

Final Trial Load Study

Since a substantial increase in excavation at the abutments of the arches was necessary over that assumed in previous studies, and there were variations in the construction program from that assumed, it was considered essential that a study be made of the stress conditions in Hungry Horse Dam as it was constructed.

A complete Trial Load analysis of the dam was made, including the construction and grouting program, effects of nonsymmetrical arches and triangular abutments where applicable. Data on concrete placing, grouting and temperature changes were obtained from the Temperature Control Unit. Thermal coefficient, modulus of elasticity and unit weight of concrete, modulus of elasticity of foundation and abutment rock were provided by the laboratory from actual test data. Data pertinent to the final study are shown in Table 5.

A plan of the dam as it was analyzed, the maximum section and a developed profile are shown on figure 2.

Basic Design Data Used in Analysis

a. Crest of dam, elevation 3565.
b. Base of crown cantilever, elevation 3050.
c. Normal reservoir water surface, elevation 3560.
d. Increase in horizontal pressure due to silt accumulations if any, were not included in the analysis.
e. Effects of tailwater were not included.

TABLE 3

EFFECTS OF TANGENTIAL SHEAR AND TWIST
ON ARCH STRESSES AT CROWN AND ABUTMENT SECTIONS
EFFECTS OF POISSON'S RATIO NOT INCLUDED

Elev.		Stresses are in pounds per square inch								
		Left Abutment			Crown			Right Abutment		
		Radial analysis	Complete analysis	Effects of tangential shear and twist	Radial analysis	Complete analysis	Effects of tangential shear and twist	Radial analysis	Complete analysis	Effects of tangential shear and twist
3565	Ext.	+ 300	+ 89	- 211	+ 340	+ 296	- 44	+ 301	+ 125	- 176
	Int.	+ 167	+ 122	- 45	+ 126	+ 172	+ 46	+ 165	+ 90	- 75
3500	Ext.	+ 242	+ 102	- 140	+ 440	+ 338	- 102	+ 210	+ 107	- 103
	Int.	+ 358	+ 202	- 156	+ 157	+ 257	+ 100	+ 391	+ 323	- 68
3450	Ext.	+ 170	+ 130	- 40	+ 479	+ 368	- 111	+ 114	+ 135	+ 21
	Int.	+ 456	+ 245	- 211	+ 140	+ 221	+ 81	+ 513	+ 323	- 190
3400	Ext.	+ 143	+ 188	+ 45	+ 510	+ 374	- 136	+ 4	+ 96	+ 92
	Int.	+ 411	+ 245	- 166	+ 89	+ 155	+ 66	+ 571	+ 405	- 166
3350	Ext.	+ 99	+ 189	+ 90	+ 518	+ 366	- 152	- 80	+ 104	+ 184
	Int.	+ 385	+ 307	- 78	+ 20	+ 104	+ 84	+ 557	+ 375	- 182
3300	Ext.	- 5	+ 163	+ 168	+ 500	+ 329	- 171	- 87	+ 96	+ 183
	Int.	+ 455	+ 335	- 120	- 29	+ 34	+ 63	+ 534	+ 406	- 188
3250	Ext.	- 69	+ 121	+ 190	+ 478	+ 302	- 176	- 114	+ 121	+ 235
	Int.	+ 479	+ 345	- 134	- 74	+ 10	+ 84	+ 521	+ 402	- 119
3200	Ext.	- 67	+ 122	+ 189	+ 393	+ 267	- 126	- 67	+ 135	+ 202
	Int.	+ 406	+ 311	- 95	- 71	- 12	+ 59	+ 403	+ 348	- 55
3150	Ext.	- 32	+ 114	+ 146	+ 291	+ 228	- 63	- 38	+ 130	+ 168
	Int.	+ 256	+ 200	- 56	- 76	- 38	+ 38	+ 268	+ 268	0

Ext. means stress at extrados of arch.
Int. means stress at intrados of arch.
Tension stresses are indicated by minus sign in table.

TABLE 4

EFFECTS OF TANGENTIAL SHEAR AND TWIST
ON STRESSES IN CROWN CANTILEVER AND CANTILEVERS D AND E
EFFECTS OF POISSON'S RATIO NOT INCLUDED

Elev.		LOADING CONDITIONS-B Stresses are in pounds per square inch								
		Cantilever - E			Crown Cantilever			Cantilever - D		
		Radial analysis	Complete analysis	Effects of tangential shear and twist	Radial analysis	Complete analysis	Effects of tangential shear and twist	Radial analysis	Complete analysis	Effects of tangential shear and twist
3565	U.	0	0	-	0	0	-	0	0	-
	D.	0	0	-	0	0	-	0	0	-
3500	U.	+ 53	+ 58	+ 5	+ 57	+ 61	+ 4	+ 52	+ 51	- 1
	D.	+ 64	+ 59	- 5	+ 60	+ 56	- 4	+ 66	+ 68	+ 2
3450	U.	+ 63	+ 72	+ 9	+ 71	+ 76	+ 5	+ 62	+ 67	+ 5
	D.	+ 136	+ 129	- 7	+ 125	+ 118	- 7	+ 137	+ 130	- 7
3400	U.	+ 66	+ 80	+ 14	+ 79	+ 86	+ 7	+ 69	+ 79	+ 10
	D.	+ 203	+ 184	- 19	+ 180	+ 170	- 10	+ 202	+ 189	- 13
3350	U.	+ 67	+ 86	+ 19	+ 86	+ 97	+ 11	+ 77	+ 93	+ 16
	D.	+ 275	+ 250	- 25	+ 243	+ 226	- 17	+ 264	+ 246	- 18
3300	U.	+ 69	+ 94	+ 25	+ 93	+ 112	+ 19	+ 88	+ 112	+ 24
	D.	+ 350	+ 318	- 32	+ 305	+ 279	- 26	+ 330	+ 304	- 26
3250	U.	+ 72	+ 103	+ 31	+ 105	+ 129	+ 24	+ 102	+ 136	+ 34
	D.	+ 432	+ 396	- 36	+ 365	+ 330	- 35	+ 398	+ 366	- 32
3200	U.	+ 82	+ 118	+ 36	+ 122	+ 153	+ 31	+ 124	+ 164	+ 40
	D.	+ 512	+ 476	- 36	+ 418	+ 373	- 45	+ 466	+ 438	- 28
3150	U.	+ 95	+ 137	+ 42	+ 146	+ 183	+ 37	+ 152	+ 196	+ 44
	D	+ 578	+ 545	- 33	+ 463	+ 408	- 55	+ 518	+ 503	- 15
3050	U.				+ 204	+ 255	+ 51			
	D.				+ 542	+ 463	- 79			

U. means stress at upstream face of cantilevers.
D. means stress at downstream face of cantilevers.
Tension stresses are indicated by minus sign in table.

f. Ice pressure, 5 tons per linear foot with a sheet of ice 2.25 feet thick.
g. Maximum horizontal earthquake has an acceleration of one-tenth gravity, a period of vibration of one second, and a direction of vibration parallel to the line of centers.
h. Effects of vertical earthquake acceleration were not included.
i. Sustained modulus of elasticity of concrete in tension and compression, 3,940,000 pounds per square inch.
j. Sustained modulus of elasticity of foundation and abutment rock 4,400,000 pounds per square inch.
k. Modulus of elasticity of concrete in shear, 1,641,667 pounds per square inch reduced to 1,313,333 pounds per square inch in computing the detrusions caused by radial shears to allow for the nonlinear distribution of shearing stresses between the upstream and downstream faces of the dam.
l. Poisson's ratio for concrete and abutment rock, 0.20.
m. Unit weight of concrete, 150 pounds per cubic foot.
n. Unit weight of water, 62.5 pounds per cubic foot.
o. Coefficient of thermal expansion of concrete, 0,000,005,3 feet per foot per degree Fahrenheit.

Basic Assumptions Used in Analysis

a. All arch rings were assumed to have radial abutments except those below elevation 3400 which were assumed to have triangular abutments on the right side.
b. Arch rings above elevation 3400 are symmetrical about their crowns and are nonsymmetrically loaded. Below elevation 3400, the arch rings are nonsymmetrical about their crowns and nonsymmetrically loaded.
c. Foundation and abutment rock formations at the site of Hungry Horse Dam have adequate strength to safely carry the loads transmitted by the dam.
d. The concrete in the dam is homogeneous, uniformly elastic in all directions, and strong enough to carry the applied loads with stresses well below the elastic limit.
e. The dam is thoroughly keyed into the foundation and abutment rock throughout its contact with the canyon profile so that arches may be considered as fixed with relation to the abutment, and cantilevers as fixed with relation to the foundation.
f. Contraction joints were thoroughly grouted according to the grouting schedule and it is assumed that these joints will remain grouted throughout the life of the structure.

Loading Conditions

In determining the stresses from the Trial Load analysis, every effort was made to duplicate as nearly as possible the actual conditions at the damsite and the construction and grouting program followed as the dam was built. The final excavation, effects of earthquake, ice pressure and concrete cooling were included in the study. The loading conditions and the analyses made to include these effects are as follows:

Table 5
Pertinent data relating to the Hungry Horse Dam

Elevation - - feet	Water pressure - lb/ft^2	Pressure due to earthquake* lb/ft^2	Date grouting completed	Average Temperature at closure °F	Average elevation of concrete at closure	Water surface elevation at closure	Range of concrete Temperature** °F Max	Min
3565	0	0	4-17-53	38.0	3565	3376	59.5	33.5
3500	3750	811	4- 3-53	37.6	3565	3373	53.5	41.0
3450	6875	1178	3-30-53	37.6	3565	3371	50.5	43.0
3400	10000	1472	5- 9-52	38.4	3446.5	3292	48.1	43.7
3350	13125	1713	5- 4-52	38.6	3438.75	3285	47.3	43.9
3300	16250	1911	4-26-52	37.3	3430.5	3246	46.8	44.0
3250	19375	2069	4-18-52	37.7	3421.25	3208	46.4	44.1
3200	22500	2190	4-13-51	38.7	3233.5	None	46.2	44.2
3150	25625	2275	4- 5-51	38.4	3228.5	None	46.0	44.2
3050	31875	2343						

* Computed by procedures of U.S.B.R. Engineering Monograph No. 11.

** Range of mean temperatures of concrete at the elevations stated. The effects of solar radiation have been included in these estimates.

a. Stage 1

(1) Dam built to elevation 3225.
(2) Reservoir empty, joints ungrouted.
(3) Analysis made: Gravity analysis, dead load carried by cantilevers.

b. Stage 2

(1) Top of dam raised from elevation 3225 to elevation 3450.
(2) Reservoir water surface at elevation 3250.
(3) Dam grouted to elevation 3200.
(4) Analysis made: Radial Trial Load analysis below elevation 3200, including water load to elevation 3250 and the weight of concrete above elevation 3225. Temperatures used are shown in Table 6.

c. Stage 3

(1) Concrete raised from elevation 3450 to elevation 3565.
(2) Reservoir water surface raised from elevation 3250 to elevation 3375.
(3) Dam grouted to elevation 3400.
(4) Analysis made: Radial Trial Load analysis below elevation 3400, including raising reservoir water surface from elevation 3250 to elevation 3375 and the weight of concrete above elevation 3400. Temperature changes used in this analysis are shown in Table 6.

d. Stage 4

(1) Dam grouted to elevation 3565.
(2) Reservoir water raised from elevation 3375 to elevation 3560.
(3) Analysis made: Complete Trial Load analysis of entire dam, including tangential shear and twist effects for Stages 2, 3, and 4, raising reservoir water surface from elevation 3375 to elevation 3560, horizontal earthquake and ice pressure. Temperatures used in this analysis are shown in Table 6.

Table 6

EFFECTIVE CONCRETE TEMPERATURES
USED IN FINAL TRIAL LOAD STUDY

	Stage 2	Stage 3	Stage 4	Final
Elevation	Temp. °F	Temp. °F	Temp. °F	Temp. °F
3565			-4.5	33.5
3500			+3.4	41.0
3450			+5.4	43.0
3400		+3.6	+1.7	43.7
3350		+6.4	-1.1	43.9
3300		+6.7	0	44.0
3250		+6.3	+0.1	44.1
3200	+3.3	+2.0	+0.2	44.2
3150	+4.6	+2.0	-0.8	44.2

Plus sign means temperature rise.
Minus sign means temperature drop.
Temperatures in Table 6 reflect the effects of subcooling the concrete prior to closure of contraction joints.

e. Stresses in the completed dam were obtained by combining the forces and moments from all four stages and computing the desired stresses therefrom.

Comparison of Stresses in Dam As Constructed With Proposed Design

Comparison of Arch Stresses

In Table 7 is shown a comparison of arch stresses in Hungry Horse Dam, as it was constructed, with the proposed design study. The table is self explanatory. It should be noted that the dam as constructed has a maximum arch compressive stress of 535 pounds per square inch as compared to 405 pounds per square inch in the design study. Small tensile stresses found at the crown intrados of the design study are not present in the dam as constructed. Tensile stresses are found at the abutments of the top arch where compression occurred in the design study; this being due probably to a greatly increased temperature drop in the top portion of the dam. Variations will be noted in the arch stresses throughout the dam but, other than those noted, are not of great importance.

Comparison of Cantilever Stresses

Table 8 shows a comparison of cantilever stresses in Hungry Horse Dam, as it was constructed, and as estimated in the proposed design study. The table indicates that, in general, the stresses at the upstream face are higher in the dam as constructed than in the proposed design, while at the downstream face they are lower. The maximum cantilever stress in the dam decreased from 545 pounds per square inch to 521 pounds per square inch in the completed dam. All of the cantilever stress changes were small and of minor importance.

Comparison of Principal Stresses

A comparison of principal stresses for the dam as constructed with the proposed design is shown in Table 9. Orientations of the principal stresses are shown on figure 5. The maximum compressive principal stress is 631 pounds per square inch in the dam as constructed, compared with 731 pounds per square inch in the proposed design. The maximum tensile principal stress in the dam as constructed is 79 pounds per square inch compared with 30 pounds per square inch in the proposed design.

SUMMARY

The amount of excavation required in the lower part of Hungry Horse damsite was more extensive than was originally anticipated. This resulted in a change of arch and cantilever properties which in turn caused a redistribution of loads throughout the dam. The construction and grouting program as well as the depth of water in the reservoir at the time of grouting varied from the original assumptions, resulting in changes in load distribution. The results of these changes on the stresses are shown in the tables included herein.

An over-all appraisal indicates an increase in maximum principal tensile stresses at the right abutment extrados in the lower part of the dam, and a

TABLE 7

COMPARISON OF ARCH STRESSES
AS CONSTRUCTED (STUDY 3) WITH PROPOSED DESIGN (STUDY 2-A)

Elev.		Left abut.		3/4		1/2		1/4		Crown		1/4		1/2		3/4		Right abut.	
		Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A
3565	Ext.	-72	+89	+17	+52	+111	+160	+205	+209	+277	+296	+279	+246	+217	+175	+77	+61	-44	+125
	Int.	-43	+122	+50	+57	+147	+159	+188	+240	+197	+172	+200	+204	+160	+150	+175	+123	-40	+90
3500	Ext.	+41	+102	+170	+174	+252	+246	+314	+291	+377	+338	+351	+326	+315	+283	+200	+205	+114	+107
	Int.	+157	+202	+223	+218	+273	+232	+285	+273	+243	+257	+253	+243	+243	+213	+291	+240	+199	+323
3450	Ext.	+105	+130	+177	+196	+242	+234	+326	+315	+393	+368	+380	+353	+306	+301	+229	+216	+141	+135
	Int.	+212	+245	+253	+237	+279	+274	+262	+255	+220	+221	+213	+220	+234	+232	+282	+278	+421	+323
3400	Ext.	+143	+188	+152	+196	+228	+224	+328	+306	+374	+374	+357	+351	+300	+295	+261	+209	+16	+96
	Int.	+233	+245	+271	+272	+248	+278	+192	+217	+161	+155	+219	+177	+207	+222	+242	+294	+535	+405
3350	Ext.	+139	+189	+157	+164	+223	+207	+317	+304	+370	+366	+340	+334	+272	+263	+187	+181	+94	+104
	Int.	+293	+307	+287	+280	+243	+248	+170	+165	+122	+104	+142	+120	+179	+191	+251	+260	+380	+375
3300	Ext.	+137	+163	+177	+155	+246	+196	+328	+277	+373	+329	+344	+307	+272	+246	+196	+186	+128	+96
	Int.	+329	+335	+301	+275	+239	+212	+160	+101	+113	+34	+134	+65	+188	+160	+254	+243	+336	+406
3250	Ext.	+133	+121	+174	+144	+240	+201	+305	+268	+330	+302	+299	+285	+246	+237	+189	+187	+110	+121
	Int.	+329	+345	+272	+223	+192	+141	+118	+55	+88	+10	+115	+33	+169	+108	+218	+198	+265	+402
3200	Ext.	+142	+122			+214	+201			+264	+267			+209	+223			+134	+135
	Int.	+283	+311			+131	+81			+56	-12			+126	+69			+264	+348
3150	Ext.	+136	+114			+209	+192			+248	+228			+218	+210			+163	+130
	Int.	+214	+200			+100	+15			+52	-38			+97	+12			+203	+268

Stresses are in pounds per square inch.
(+) Indicates compression. (-) Indicates tension.
Ext. = extrados of arch.
Int. = intrados of arch.

TABLE 8

COMPARISON OF CANTILEVER STRESSES AS CONSTRUCTED (STUDY 3) WITH PROPOSED DESIGN (STUDY 2-A)

Elev.		H		G		F		E		Crown		D		C		B		A	
		Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A	Study 3	Study 2-A
3565	U	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	D	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3500	U	+30	+33	+43	+50	+52	+50	+58	+58	+63	+61	+64	+51	+56	+49	+50	+40	+41	+30
	D	+93	+89	+77	+69	+66	+68	+59	+59	+53	+56	+52	+68	+61	+70	+68	+81	+79	+92
3450	U	+31	+30	+54	+60	+68	+63	+76	+72	+85	+76	+85	+67	+74	+66	+64	+48	+49	+29
	D	+173	+181	+149	+140	+128	+136	+116	+123	+103	+118	+104	+130	+120	+132	+135	+157	+150	+183
3400	U			+77	+81	+83	+73	+88	+80	+100	+86	+96	+79	+97	+82	+95	+68		
	D			+235	+226	+199	+209	+176	+184	+151	+170	+165	+189	+175	+195	+190	+235		
3350	U			+104	+107	+99	+83	+99	+86	+115	+97	+105	+93	+128	+101	+138	+97		
	D			+281	+274	+262	+281	+236	+250	+201	+226	+229	+246	+211	+254	+203	+284		
3300	U					+120	+101	+114	+94	+134	+112	+123	+112	+160	+128				
	D					+335	+357	+301	+318	+248	+279	+291	+304	+265	+314				
3250	U					+148	+121	+132	+103	+156	[illegible]	+149	+136	+196	+162				
	D					+416	+458	+370	+396	+291	+330	+354	+366	+332	+390				
3200	U							+155	+118	+180	+153	+177	+164						
	D							+448	+476	+333	+373	+427	+438						
3150	U							+180	+137	+201	+183	+203	+196						
	D							+521	+545	+382	+408	+500	+503						
3050	U									+242	+255								
	D									+483	+463								

Stresses are in pounds per square inch.
(+) Indicates compression.
U = Upstream edge of cantilever.
D = Downstream edge of cantilever.

TABLE 9

COMPARISON OF PRINCIPAL STRESSES AS CONSTRUCTED (STUDY 3) WITH PROPOSED DESIGN (STUDY 2-A)

Elev.		Left abutment				Right abutment			
		Study 3		Study 2-A		Study 3		Study 2-A	
		σP_1	σP_2	σP_1	σP_2	σP_1	σP_2	σP_1	σP_2
3565	U	0	-72	0	+89	0	-44	0	+125
	D	-43	0	+122	0	-40	0	+90	0
3500	U	+18	+45	+9	+121	-1	+149	+7	+126
	D	+204	+51	+235	+62	+306	-22	+336	+85
3450	U	-5	+141	-7	+167	-23	+212	-30	+195
	D	+352	+33	+357	+70	+515	+56	+412	+94
3400	U	+34	+180	+41	+222	-73	+182	-27	+182
	D	+441	+52	+387	+64	+622	+101	+549	+140
3350	U	+45	+198	+61	+236	-21	+253	-27	+229
	D	+482	+67	+523	+36	+522	+38	+615	+22
3300	U	+45	+221	+50	+234	-9	+310	-30	+258
	D	+547	+80	+618	+30	+547	-14	+711	-18
3250	U	+45	+236	+21	+222	-7	+313	+8	+275
	D	+595	+83	+705	+48	+587	-79	+724	+5
3200	U	+58	+249	+5	+237	+43	+313	+24	+294
	D	+611	+83	+731	+68	+631	-73	+719	+14
3150	U	+68	+247	-2	+252	+99	+267	+27	+299
	D	+595	+70	+684	+24	+619	-21	+704	+1
3050	U					+243	0	+256	0
	D					0	+483	-1	+464

Stresses are in pounds per square inch.
(+) Indicates compression, (-) Indicates tension.
U = Upstream face of dam. D = Downstream face of dam.

general decrease in the higher compressive stresses. Tensile stresses were found at the abutments of the top arch; which were probably caused by a larger temperature drop in the concrete than was assumed in the design study. The maximum tensile stress of 79 pounds per square inch and compressive stress of 631 pounds per square inch are well within allowable limits.

ACKNOWLEDGMENTS

It is not possible, in the brief account given here, to mention all those who have contributed to development of Trial Load procedures at the Bureau of Reclamation. This process has benefited from the thought given it by many Engineers. Not the least of these contributions has been made by those who, by the maintenance of the required technical skills, have made possible an effective application of these methods when they were needed.

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NOTE: References are continued on page 960-25.

FIGURE 1. HUNGRY HORSE DAM

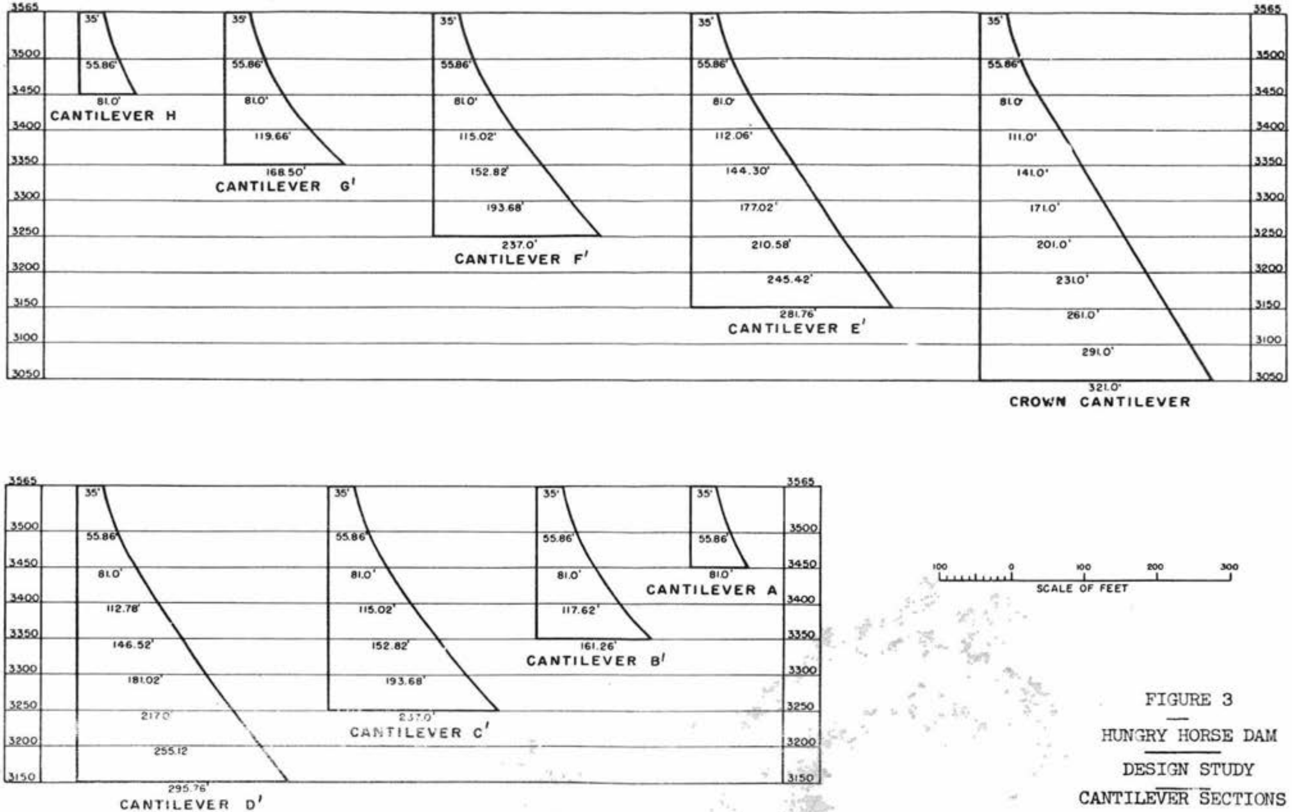

FIGURE 3

HUNGRY HORSE DAM

DESIGN STUDY

CANTILEVER SECTIONS

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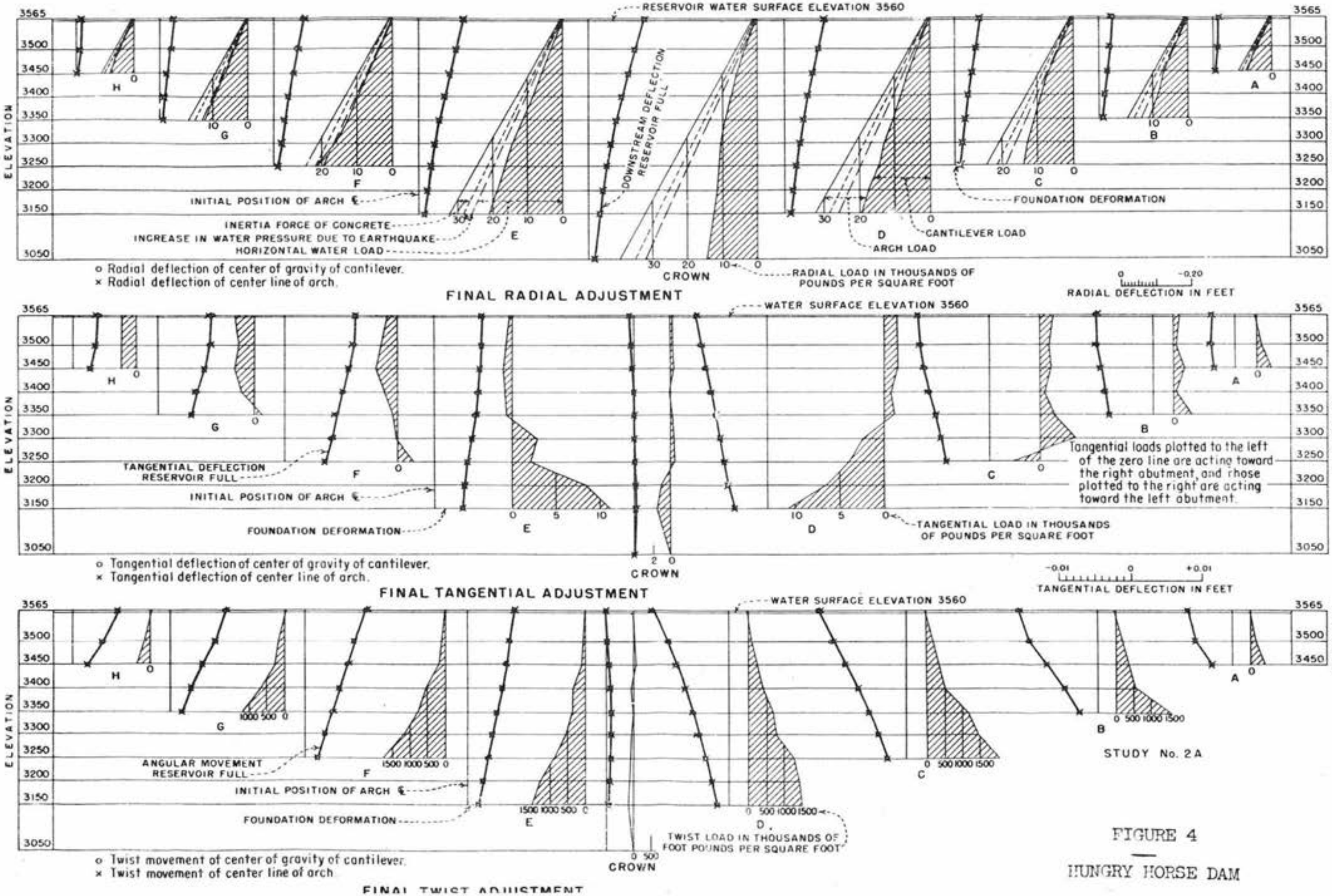
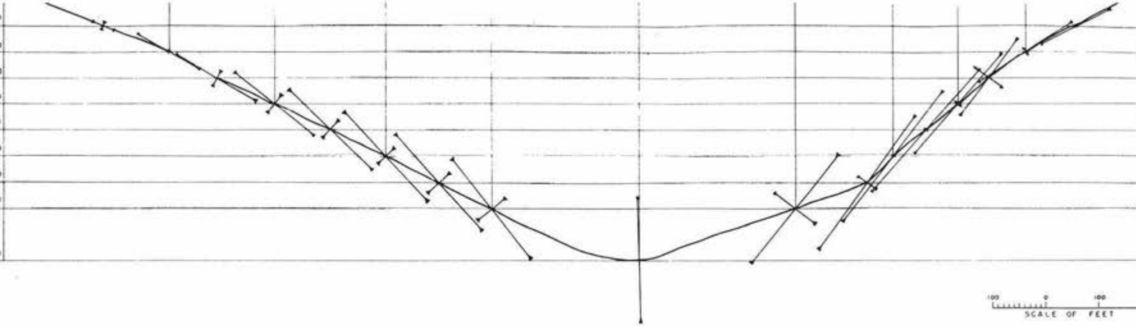


FIGURE 4

HUNGRY HORSE DAM

σP_1	σP_2	α	Elev.
+18	+45	23°03'	3500
-5	+141	29°49'	3450
+34	+180	30°25'	3400
+45	+198	38°29'	3350
+45	+221	43°38'	3300
+45	+236	47°11'	3250
+58	+249	48°24'	3200
+68	+247	52°03'	3150
			3050

Maximum shear stresses occur at the downstream face except at elevations 3565 and 3500 where it occurs in the interior of the section



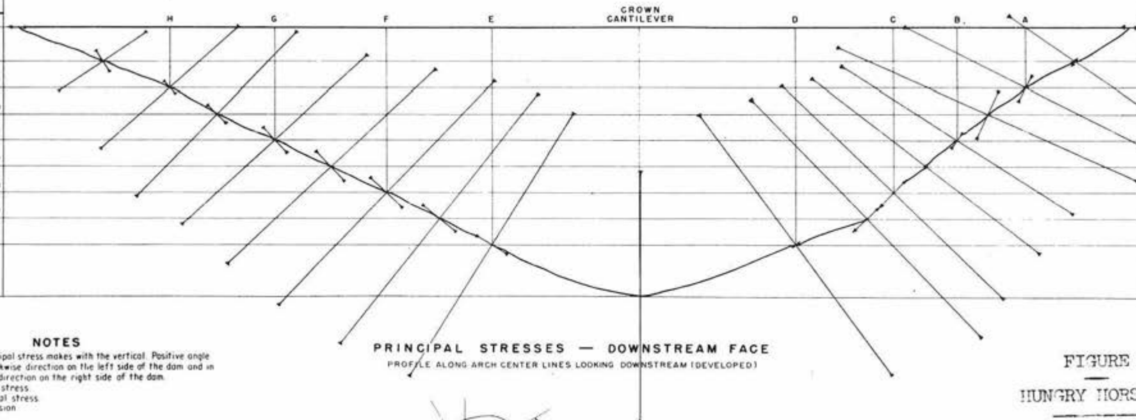
Elev.	σP_1	σP_2	α
3500	1	+149	28°57'
3450	-23	+212	33°27'
3400	-7.3	+182	53°48'
3350	-21	+253	49°34'
3300	-9	+310	49°01'
3250	-7	+313	52°47'
3200	+43	+313	54°31'
3150	+99	+267	51°55'
3050	+243	0	0°42'

Maximum shear stresses occur at the downstream face except at elevation 3565 where it occurs in the interior of the section.

PRINCIPAL STRESSES — UPSTREAM FACE

PROFILE ALONG ARCH CENTER LINES LOOKING DOWNSTREAM (DEVELOPED)

PRINCIPAL STRESSES			Δ Max.	
σP_1	σP_2	α	Shear	Elev.
-43	0	90°00'	-8	3565
+204	+51	56°21'	+40	3500
+352	+33	48°32'	+101	3450
+441	+52	44°07'	+174	3400
+482	+67	47°29'	+194	3350
+547	+80	46°53'	+233	3300
+595	+86	43°38'	+273	3250
+611	+83	37°59'	+298	3200
+595	+70	31°37'	+296	3150
				3050



	PRINCIPAL STRESSES			Δ Max.
Elev.	σP_1	σP_2	α	Shear
3565	-40	0	90°00'	-8
3500	+306	-22	55°12'	+48
3450	+515	+56	63°08'	+99
3400	+622	+101	65°50'	+158
3350	+522	+38	57°15'	+226
3300	+547	-14	52°08'	+270
3250	+587	-79	45°56'	+316
3200	+631	-73	43°45'	+332
3150	+619	-21	36°12'	+316
3050	0	+483	89°54'	+213

PRINCIPAL STRESSES — DOWNSTREAM FACE

PROFILE ALONG ARCH CENTER LINES LOOKING DOWNSTREAM (DEVELOPED)

NOTES

α = Angle first principal stress makes with the vertical. Positive angle measured in a clockwise direction on the left side of the dam and in counter clockwise direction on the right side of the dam.
σP_1 = First principal stress.
σP_2 = Second principal stress.
⟼ = Compression
⟷ = Tension
Δ = Maximum horizontal shear stress at rock abutment planes
+ Indicates downstream shear
See Drawing No. 447-D-2030 for constants, assumptions and loading conditions.

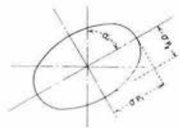


FIGURE 5

HUNGRY HORSE DAM

AS CONSTRUCTED

PRINCIPAL STRESSES INCLUDING EFFECTS OF TANGENTIAL SHEAR AND TWIST

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: PORTUGUESE EXPERIENCE WITH OVERFLOW ARCH DAMS*

A. C. Xerez**
(Proc. Paper 990)

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams held in April, 1939, much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be included in the Symposium. So far, nine papers have been approved: "Arch Dams: Their Philosophy" (Proc. Paper 959) by Andre Coyne; "Arch Dams: Trial Load Studies for Hungry Horse Dam" (Proc. Paper 960) by R. E. Glover and Merlin D. Copen; "Arch Dams: Portuguese Experience with Overflow Arch Dams" (Proc. Paper 990) by A. C. Xerez; "Arch Dams: Theory, Methods, and Details of Joint Grouting" (Proc. Paper 991) by A. Warren Simonds: "Arch Dams: Santa Giustina Single-Curvature Arch Dam" (Proc. Paper 992) by Claudio Marcello; "Arch Dams: Measurements and Studies on Santa Gustina Dam" (Proc. Paper 993) by Claudio Marcello; "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam" (Proc. Paper 994) by Claudio Marcello; "Arch Dams: Isolato Double-Curvature Arch Dam (Proc. Paper 995) by Claudio Marcello; and "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs" (Proc. Paper 996) by Claudio Marcello.

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

Note: Discussion open until November 1, 1956. Paper 990 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 3, June, 1956.

* Paper presented at the Symposium on Arch Dams of the ASCE, Power Div., June, 1956.

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SYNOPSIS

An important part of every dam, affecting both safety and cost, is the arrangement by which flood waters by-pass the dam. This paper discusses Portuguese experience with arch dam spillways. Discussed are the problems involved in the various types and presented are actual examples.

The delicate problem of deciding upon the layout of the appurtenant works of a dam is intimately related to that of flood discharge. The problem arises as soon as it becomes necessary to estimate the maximum flood and to establish its hydrograph. The difficulties in obtaining these elements are very well known to hydraulic engineers who appreciate the responsibilities involved as regards the safety of the dam.

The examination of this safety should be faced not only from the point of view of strength of the structure; it must be taken further, bearing in mind, particularly, the question of flood discharge. It frequently happens that engineers who are not fully aware of the problem of flood discharge are led to arrive at solutions which they feel to be the most convenient when, should the problem be studied in all its extent and due consideration paid to the hydraulic problem, their conclusions might be very different. These considerations are related to the more general problem of the choice of the most convenient type of dam for the particular site conditions, having in mind both its safety and its cost. This problem was dealt with at the last Congress on Large Dams at which the conclusion was reached that the choice is always dependent on many factors which require individual analysis by the designer in each case.

In this connection it should be stressed that one of the most important factors is that of flood discharge which can introduce determining factors of such a nature as to alter completely the type of dam which would otherwise have been chosen. Thus it can happen that, for a certain type of valley, where a thin arch dam could be built to advantage, a solution has to be chosen involving the building of other type of dam. This will be the case if the floods to be discharged are so large that this solution would give the best overall technical and economical results when the dam and flood discharge arrangements are considered together as one whole. It appears that American engineers are well aware of this aspect of the problem because of the large floods which must be foreseen in the case of their own schemes. The reasons will therefore be well appreciated for drawing attention to this matter here.

From what has been written above, i.e., the important determining factors introduced by flood discharge, the preference which designers may have for arch dams because of their inherent reserve of strength, must be considered in conjunction with this problem particularly when large floods have to be handled. Therefore it seems justified that designers call upon the cooperation of hydraulic engineers in order to arrive at solutions which will permit of the maximum advantage being taken from the arched form. This will be possible when an arch dam discharging over the crest can be built because in such cases can the dam and flood discharge together show economic advantages over the gravity dam.

Another factor arises if the dam forms part of a hydroelectric scheme where the power station is located at its foot as this will influence the layout of the accessory installations as well as the choice of the best type of dam.

General Considerations Concerning Overflow Arch Dams

The problem of flood discharge over arch dams can be tackled by the designer in various aspects, all depending on the flow to be discharged, the height of the dam and its type. The solution to such problems varies from the thick arch dam with overfall spillway to the thin arch dam with free flood discharge.

If the flood is small no technical difficulties arise in designing overflow arch dams even in the case of high dams, and, from an economical point of view, this solution is clearly the best.

If the flood is large and the valley has topographical and geological characteristics which warrant an arch dam, then the choice of a solution for the best dam and flood discharge taken together as a unit—bearing also in mind other determining factors such as, for instance, a power station located at the foot of the dam—requires very careful study, case by case.

Where the power station is not located up against the dam itself and the valley is suitable for a thin arch dam the best technical and economical solution consists in freely discharging the flood over all or part of the crest, controlled by flood gates particularly in the latter case, providing the flow per unit length of crest is not excessive and the dam is of medium height.

When the arch dam is of the thick type, advantage can be taken of the slope of its downstream face for flood discharge although this will require the nappe to be forced to follow its shape. In this case also, if the width of the valley permits, it will be possible to locate the power station at the foot of the dam close to the spillway.

When the flow per unit length of crest is relatively high, over 80 m3/sec. per meter of crest (860 cu. ft./sec. per foot of crest), or the height of the dam exceeds 100 metres (328 ft.), the problem must be very carefully studied, specially in the case of free discharge where it becomes very delicate even for lower values than these. In such cases, although a solution involving flood discharge over the crest of an arch dam would be the most favourable from an economic point of view, solutions requiring independent spillways must also be considered. The possibility of an overflow gravity dam should also be studied as, hydraulically, it may offer greater safety.

The Hydraulic Problem and Model Tests

As far as the design of the crest is concerned, the hydraulic problem of flood discharge over a dam may be easily solved by analytical methods if the discharge is free. It cannot however be so readily solved by analysis in cases where the nappe has to be forced to follow the shape of the downstream face of the dam and difficulties become even greater if the water approaches the spillway asymmetrically.

The remaining problems requiring attention whether overfall be free or whether the nappe follows the shape of the downstream face including that very important one of energy dissipation downstream, are not open to solution by analysis and, as generally accepted, involve exhaustive model tests.

One conclusion to be drawn from the above is that the hydraulic model is an indispensable tool to the designer of dams when studying the various possible solutions; firstly using reduced scale models for preliminary studies and, later, larger models for the final and complete tests on the chosen solution. This procedure is fundamental for arriving at the best design. It is the writer's opinion that the method sometimes adopted, where the designer carries out his complete study in his drawing office and effects model tests only on the solution of his choice arrived at purely by analysis, without even directing or assisting at these model tests, is not justified.

The problem of downstream energy dissipation is the most difficult one connected with hydraulic tests. In many solutions involving overfall discharge, even in cases where the nappe follows the shape of the downstream face of the thick arch dams, the jet can be thrown on the bed of the river and hydraulic jump will not occur. The problem to be faced, therefore, is that of dissipating the energy of a jet of water falling through a given height as determined by the difference on elevation between the upstream and downstream water levels. This is the reason for the remarks made above to the effect that difficulties in arriving at safe solutions to the hydraulic problem increase as the flow per unit length of crest and the height of the dam increase. However, on the other hand, the narrower the valley, the greater will be the depth of the water in the river bed for higher discharges and this will facilitate energy dissipation.

This energy dissipation is the cause of erosion of the river bed and banks, mainly as a result of the well known phenomenon of uplift developed through cracks and diaclases in the rocks. Due to the nature of this phenomenon and as it is impossible to reproduce the actual nature of the rocks in the model, hydraulic tests cannot show the real results and, therefore, the problem must be approached by assuming the worst possible conditions so as to determine whether erosion is regressive in such a way as to cause damage to the dam foundations. In general, regressive erosion need not be feared and, therefore, the problem becomes one of analysing the pool formed in the bed of the river whose shape and depth will ultimately become stabilized. The resulting barrier across the river will have no adverse effect on the hydroelectric scheme except when formed downstream of the turbine tailrace, but in any case its removal presents no difficulties.

Structural Design Problem and Model Tests

If free discharge takes place over the whole length of the crest of a thin arch dam, no special structural design problems are involved. But, if the discharge is limited to one part of the crest, whether the dam be of the thin or thick arch type, then large intake openings, closed by gates, must be left in the arch. This brings about the problem of the re-establishment of the continuity of the arch effect through the reinforced concrete structure encasing these openings.

This problem may require the use of heavy reinforcement, not only for the purpose of ensuring structural continuity but also for supporting the gates and the resulting increase in cost must be taken into consideration when comparing this solution with others.

The study and design of these structures involve delicate problems of elasticity which are not easily approached by analysis unless simplifications be introduced in the computation. The technique of model tests, which has become so advanced throughout the world, is an indispensable working tool if used in conjunction with analytical studies but it must be borne in mind that it is up to the designer to conduct the studies in such a manner as to take full advantage of these two methods, i.e., model tests and analysis.

Some Recent Portuguese Overflow Arch Dams

Portuguese experience in the building of overflow arch dams is of interest because, having resulted from the work carried out in recent years, it has consequently benefitted from the experience gained in other countries. As a result of the national electrical development plan which is being carried out in Portugal, there are six hydroelectric schemes in operation where arch dams more than 60 metres (197 ft) high have been built. Five of these are overflow arch dams of various types.[10] Of the schemes now under construction, one of the dams is of the rock fill type 110 metres (361 ft) high and the other is a thin arch dam 90 m (295 ft) high. The latter is of the overflow type handling large floods per unit length of crest with special arrangements for the restitution of the water downstream of the dam.

This shows that in a large percentage of hydroelectric schemes, the type of overflow arch dam has been adopted. The reason for this was mainly one of economy, studies having shown that a satisfactory and safe solution to the hydraulic problem was possible. The chosen sites were suitable for the building of this type of dam and careful analytical studies and model tests had to be made with a view to arriving at the most convenient and safest solution in each case. The flood discharge over the six dams can be classified in distinct types according to their main characteristics and detailed reference is made to each below.

The design of the dam and of the spillway considered as a unit depends fundamentally on the flow to be discharged and on the layout of the accessory installations, including the power station when this is located near the dam.

The existence of a foot-of-dam type power station makes it extremely difficult to discharge floods over the dam unless an arrangement be adopted such as that which the French consulting engineer A. Coyne has used where the flood is discharged over the dam and power station together. This same engineer advised the adoption in Portugal of lateral overfall spillway in the case of the thick arch dams of Castelo do Bode and Venda Nova. The former dam has a height of 110 m (361 ft) with a foot-of-dam power station situated close to the spillway (Fig. 1).

This discharge over the downstream face of a thick arch dam is possible only within certain limits of slope and on condition that the nappe is forced to take the shape of this face by means of a especially designed intake opening

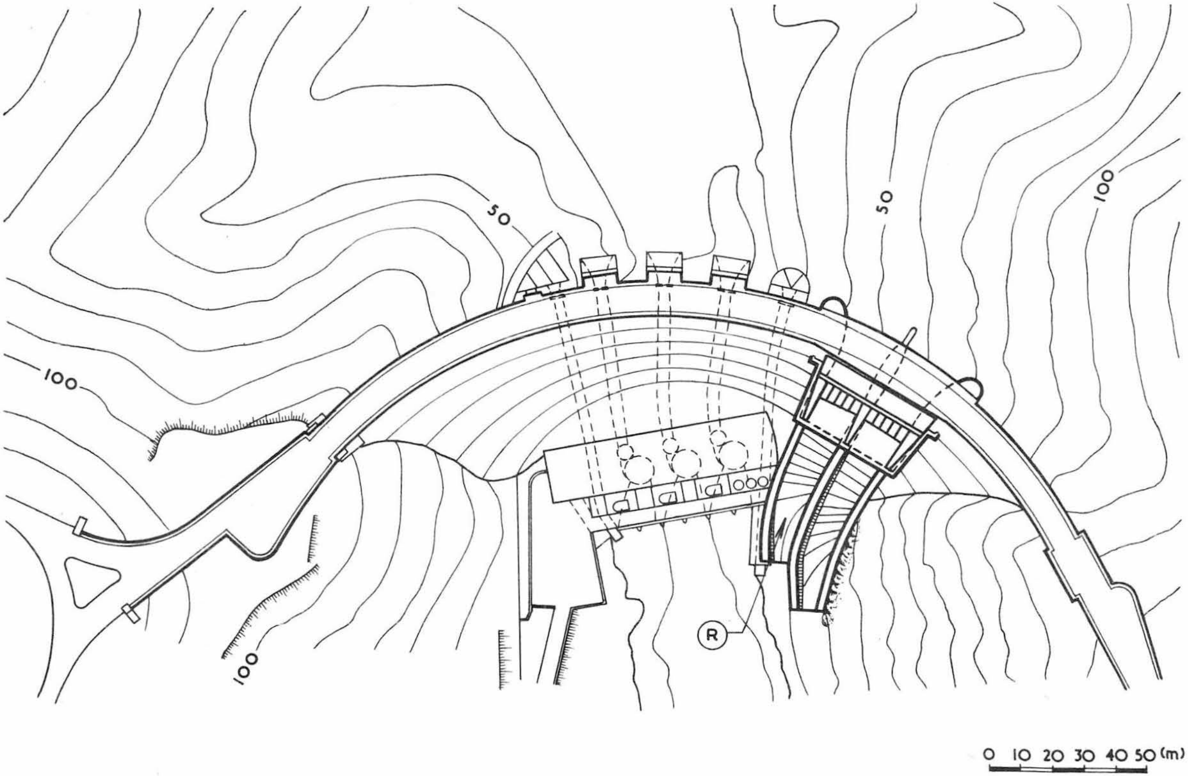


FIG. 1 — CASTELO DO BODE — PLAN OF THE DAM, POWER STATION AND SPILLWAY

fitted with control gates. In the case of Castelo do Bode, a solution was arrived at which will permit the discharge of 4,000 m3/sec (141,260 cu ft/sec through two openings each measuring 14.00 x 10.00 m (45.93 x 32.81 ft), fitted with radial gates (Fig. 2).

The restitution of the water downstream of the dam after leaving the spillway channels presented certain difficulties due to the limited space available between the power station and the left bank of the river and because of the necessity of ensuring that the jets would be directed along the axis of the river in such a way as to permit perfect aeration of the jets and ascertain that the banks would not be eroded. After exhaustive hydraulic tests, a satisfactory solution became possible by narrowing the jets; this permitted the width of the spillway channels to be restricted and allowed perfect energy dissipation in the air before the jets reached the river. This solution also made it easier to throw the jets in the desired direction. As regards the effect of the jets falling on the river bed, model tests showed that a natural pool is formed its shape becoming stabilised and that no regressive erosion takes place which might adversely affect the dam foundations. Actual experience during recent floods (Fig. 3) confirmed the conclusions arrived at the model tests.

Apart from the hydraulic problems mentioned above, the adoption of this solution required special analytical studies and model tests in relation to the reinforced concrete structure surrounding the discharge openings in the dam in order to maintain the continuity of the arch effect.

The above summarises the main aspects of one layout with a thick overflow arch dam having a power station located at its foot and which was the first one to be built in Portugal involving the discharge of large floods over a high dam.

Reverting to a layout with a power station situated close to the dam, the building of an overflow dam is simplified where it is possible to locate the power station laterally in one of the banks, leaving space enough between it and the dam to facilitate free flood discharge over its crest. This was the solution which was recently adopted in the case of the thin arch dam at Bouçã 60 m (197 ft) high (Figs. 4, 5, 6, 7), where 2,200 m3/sec (77,700 cu ft/sec) can be discharged over the whole crest with a nappe height of 3.5 m (11.48 ft), the water falling directly into a concrete lined basin. The crest is divided into three sections at two different levels; one central section and two lateral sections. The former is capable of discharging 300 m^3/sec (10,590 cu ft/sec) before the others come into operation so ensuring the formation of a stilling pool at the foot of the dam to facilitate energy dissipation.

The hydraulic model tests enabled the determination of the length of crest required for flood discharge, the correct level for the central and lateral sections and their best profil taking into account the most convenient area on which the jet should fall. These tests also permitted the study of the best method for dividing the nappe by means of piers situated on the crest in order to eliminate the well known vibratory effect which can be set up at the beginning of the discharge when the nappe is thin. By means of these tests it was further possible to design the shape of the basin to enable on one hand energy dissipation to take place satisfactorily and, on the other, to ensure

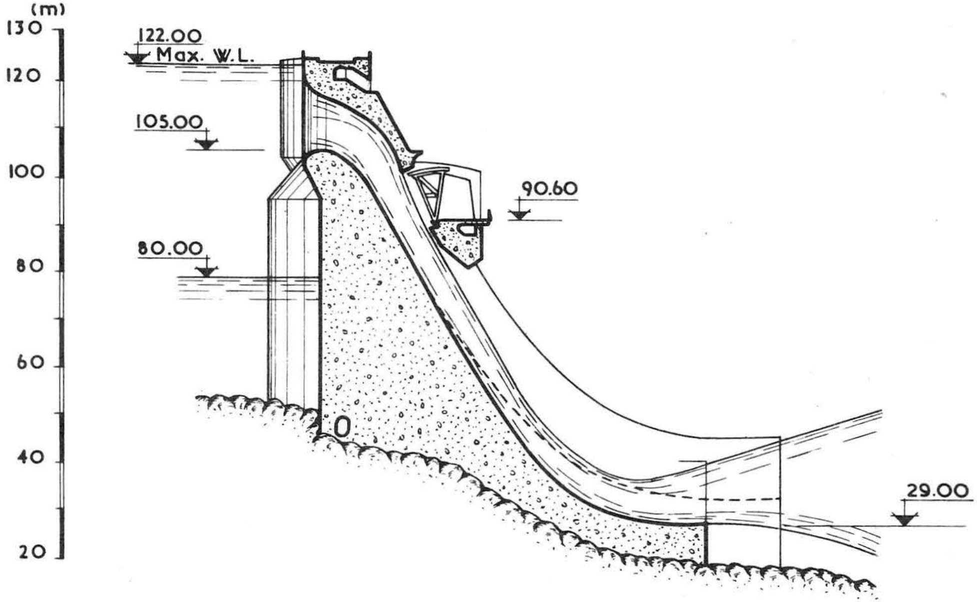


FIG. 2 – CASTELO DO BODE – SECTION THROUGH THE RIGHT SPILLWAY CHANNEL (R)

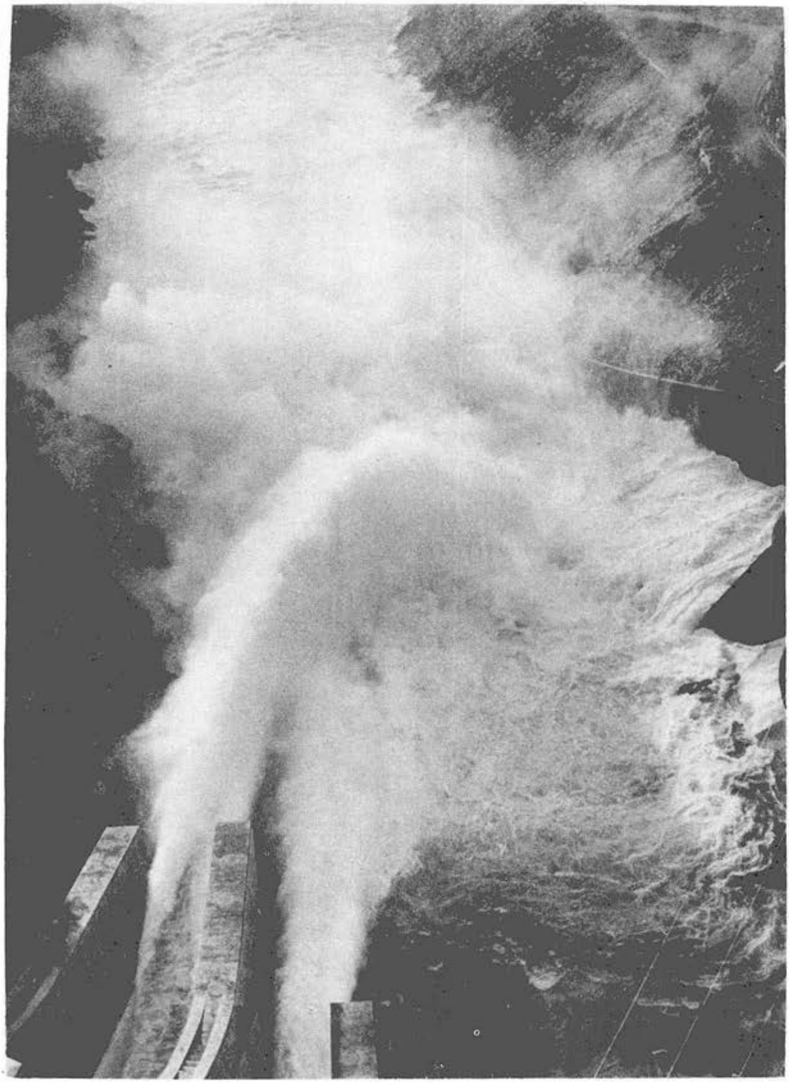


Fig. 3 - CASTELO DO BODE - Flood discharge of 1,100 m^3/sec. (38,850 cu.ft./sec.). Outlets of the channels, viewed from the crest of the dam.

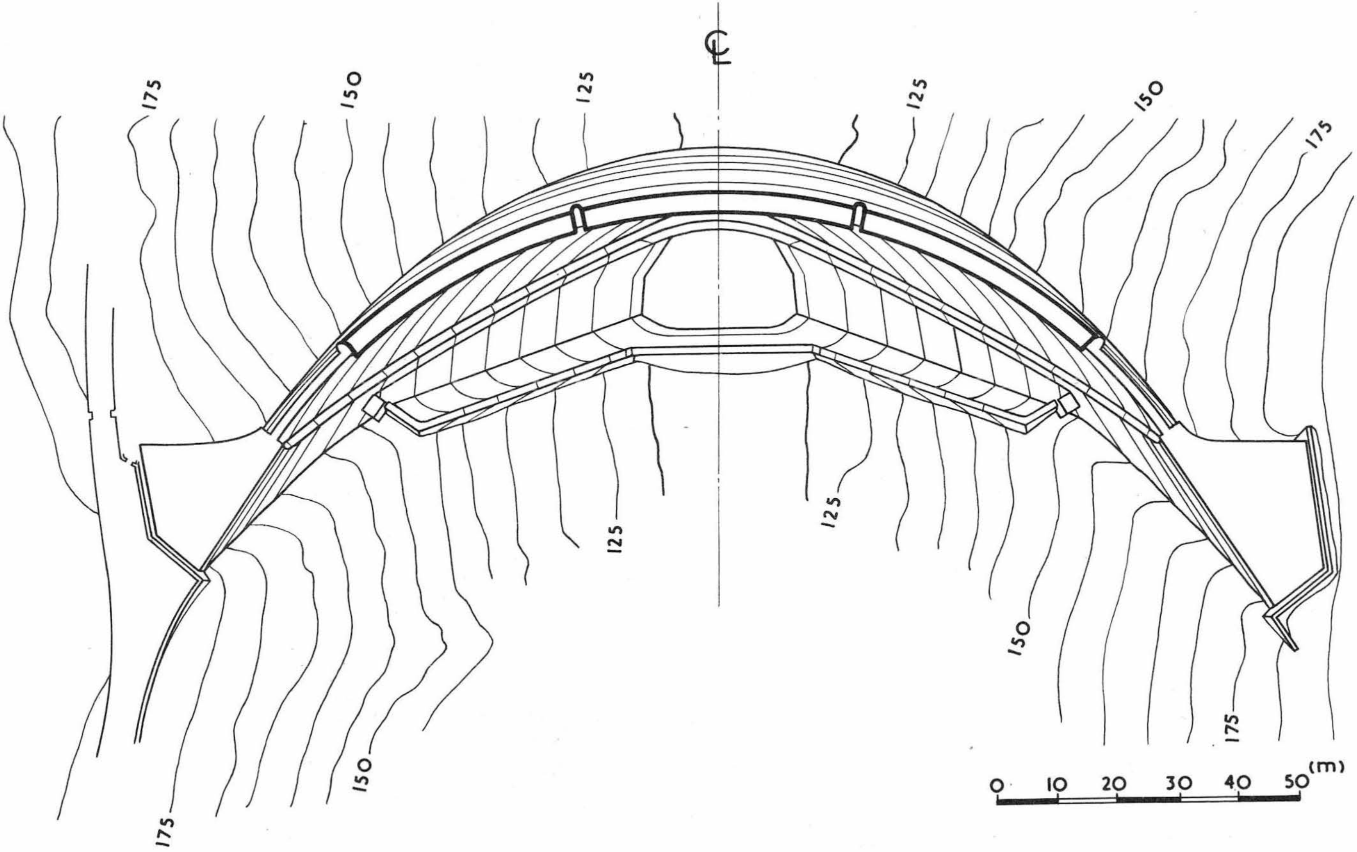


FIG. 4— BOUÇÃ — PLAN OF THE DAM AND SPILLWAY

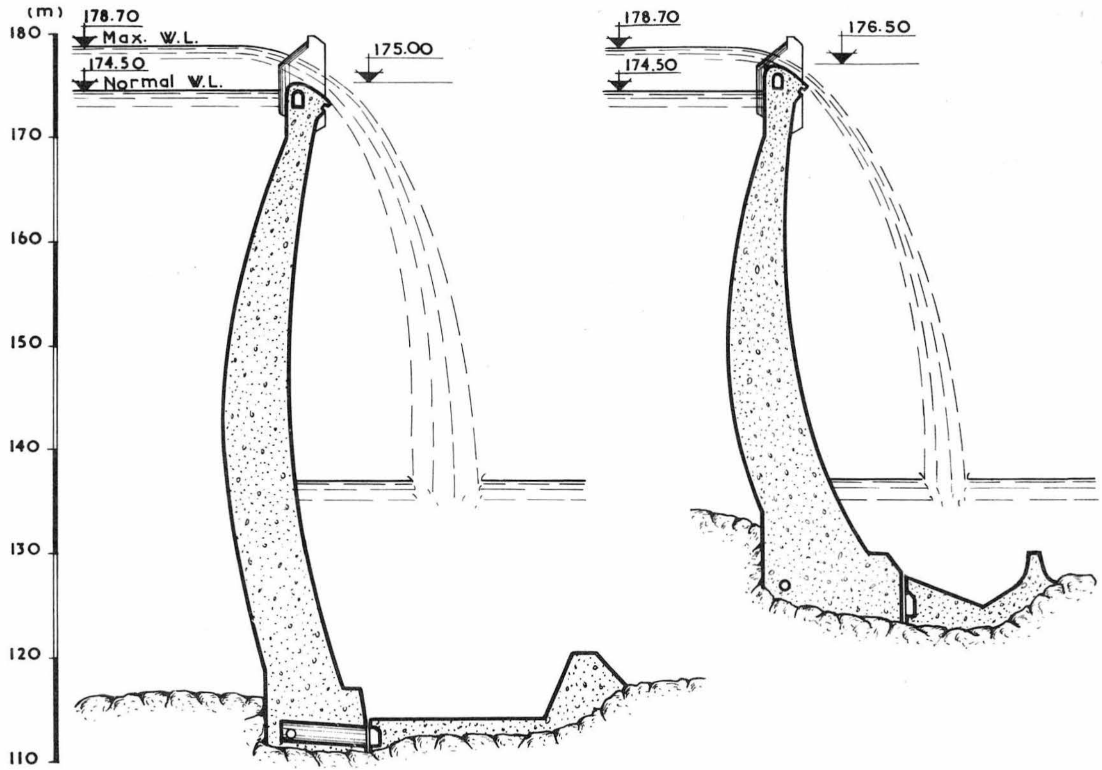


FIG. 5 — BOUÇÃ — CENTRAL AND LATERAL PROFILES OF THE DAM AND SPILLWAY

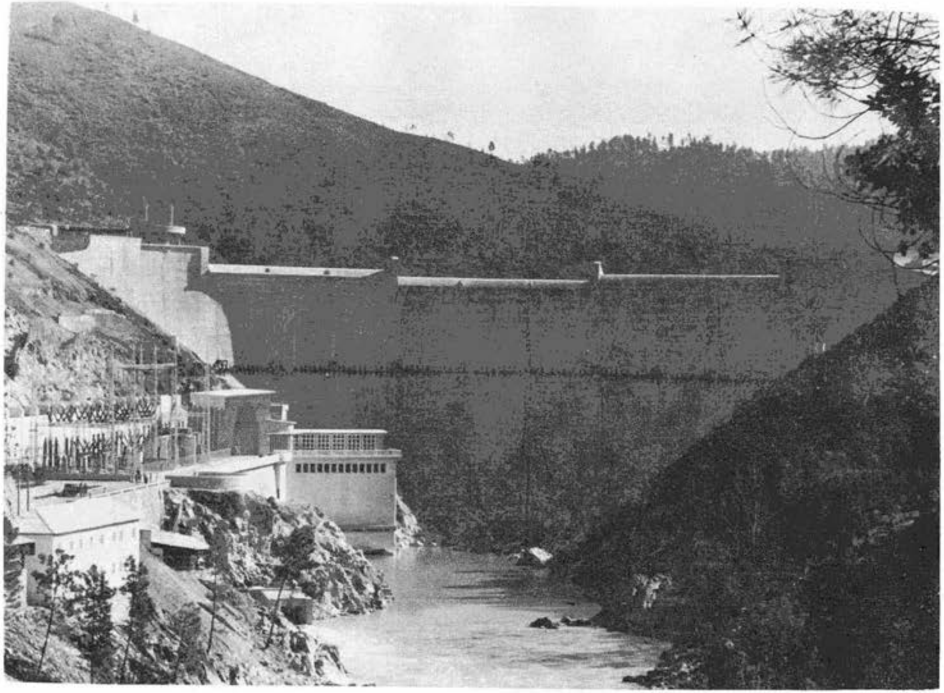


Fig. 6 - BOUÇÃ - Downstream view of the dam and power station.

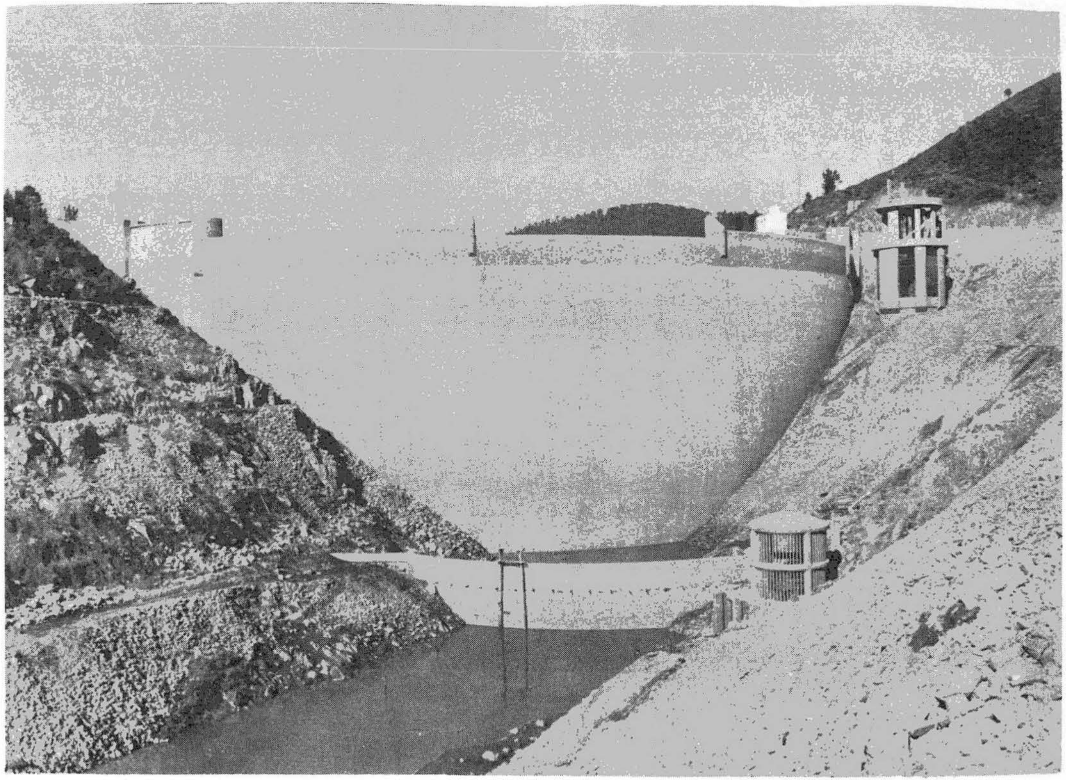


Fig. 7 - BOUÇÃ - Upstream view of the dam and intake structure.

that restitution downstream took place in such a way as to avoid undue disturbance in the vicinity of the power station under all flood conditions, mainly for the most frequently expected flow of about 500 m3/sec (17,660 cu ft/sec).

In last February, the river Zezere was in heavy flood and a maximum flow of 1,500 m3/sec (52,980 cu ft/sec) was experienced. During the discharge (Fig. 8) the absence of disturbance in the vicinity of the power station and the satisfactory conditions of energy dissipation were confirmed and coincided within the normal similarity limitations with the observations made during model tests (Fig. 9).

This type of flood discharge does neither affect the arch form nor does it bring about any special constructional problem except the one resulting from the convenience in giving geometric continuity to the downstream face and spillway basin and the establishment of an expansion joint between these two elements which have different elastic characteristics.

Flood discharge over a thin arch dam can also be restricted to its central portion using openings fitted with control gates, usually of the stoney type, to attend to space limitations. This third layout for the dam and spillway has been adopted at Salamonde and Caniçada (Figs. 10, 11) where the power stations are underground. The former has a height of 75 m (246 ft) and is capable of handling floods of 1,700 m3/sec (60,037 cu ft/sec), the corresponding values for the latter being approximately the same. This type of flood discharge does not demand special hydraulic tests as compared with the previous type nor are any special constructional problems involved downstream. The only care required is the consolidation and lining of a small area to receive the impact of the jet.

Free flood discharge over a thin arch dam offers a simple solution and a cheap construction can be used. However, when large floods per unit length of crest are discharged, this solution for dams of great height must be regarded with care from a safety point of view. This was the reason why this type of discharge was not adopted at Cabril[11] where the dam has a height of 135 metres (443 ft) and must handle floods of the order of 2,000 m3/sec (70,600 cu ft/sec). Although extensive tests were made on the model of an overflow dam, the solution of having independent flood discharge by means of two tunnels, one on each bank, was finally selected (Figs. 12, 13, 14).

Finally, mention will be made of the type of overflow arch dam arrangement most recently adopted in Portugal and which is now under construction. The problem put before the designers was the building of a dam about 90 m (295 ft) high in a gorge of well consolidated granite having a width, at the level of the crest, of little more than 100 m (328 ft) and where exceptional floods of 11,000 m3/sec (388,470 cu ft/sec) had to be contemplated. This is a case of having to deal with a large flow per unit length of crest where, due to the great height of the dam, a very large amount of energy has to be dissipated under flood conditions. The dam forms part of the hydroelectric project of Picote on the international section of the river Douro.

The difficulties in arriving at a satisfactory layout for the dam, spillway and power station can be appreciated from what has been written above bearing in mind the narrowness of the gorge and the convenience of taking full advantage of the geological and topographical conditions which are so favorable for the building of an arch dam. This is a typical case which conforms

Fig. 8 - BOUÇÃ - Flood discharge of 1,300 cu.m./sec. (45,910 cu.ft/sec.)

Fig. 9 - BOUÇÃ - Hydraulic model in operation (Tests directed and operated by the Civil Engineering Department of HIDRO-ELECTRICA DO ZEZERE).

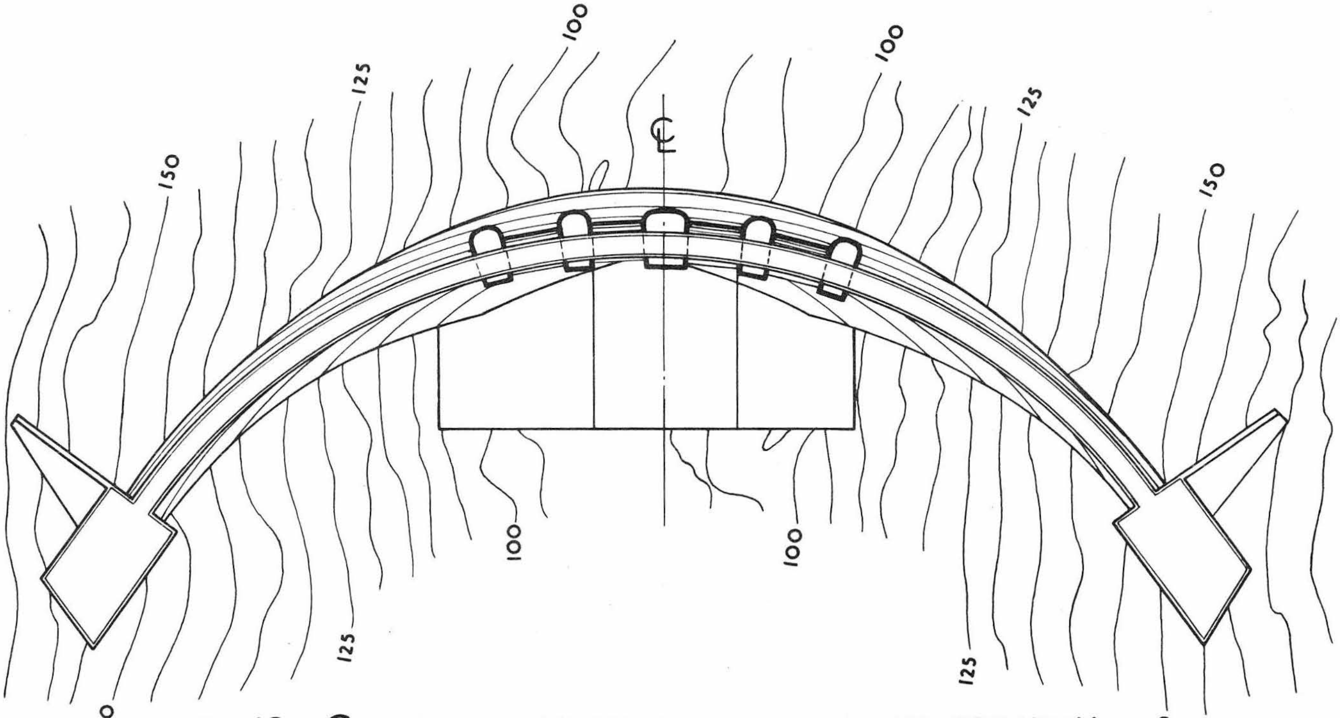
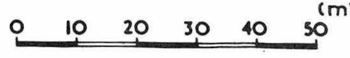


FIG.10 — CANIÇADA — PLAN OF THE DAM AND SPILLWAY



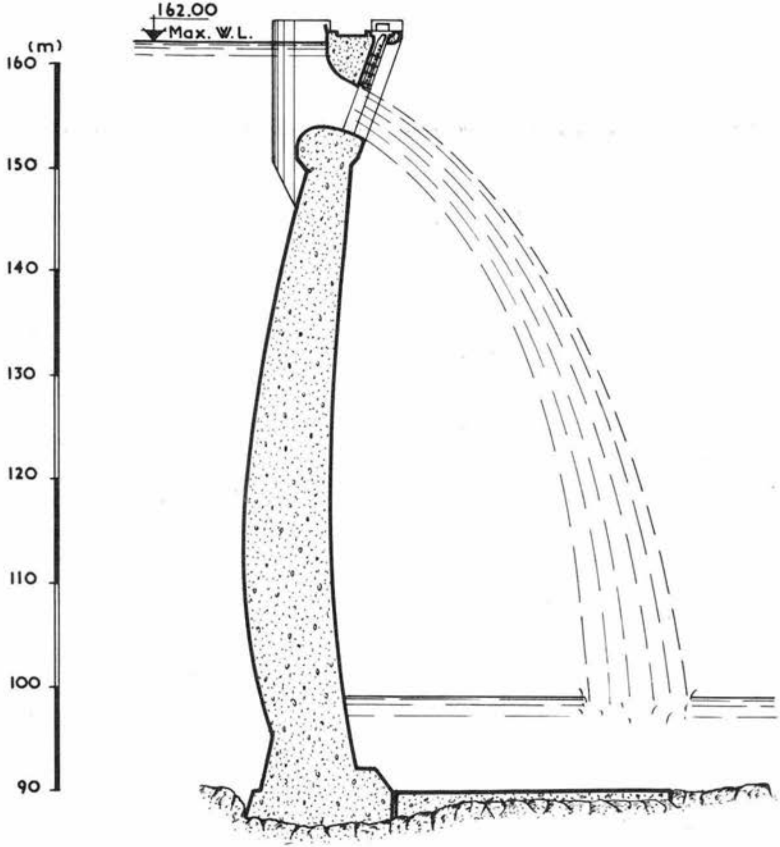


FIG. 11 — CANIÇADA — CENTRAL PROFILE OF THE DAM AND SPILLWAY

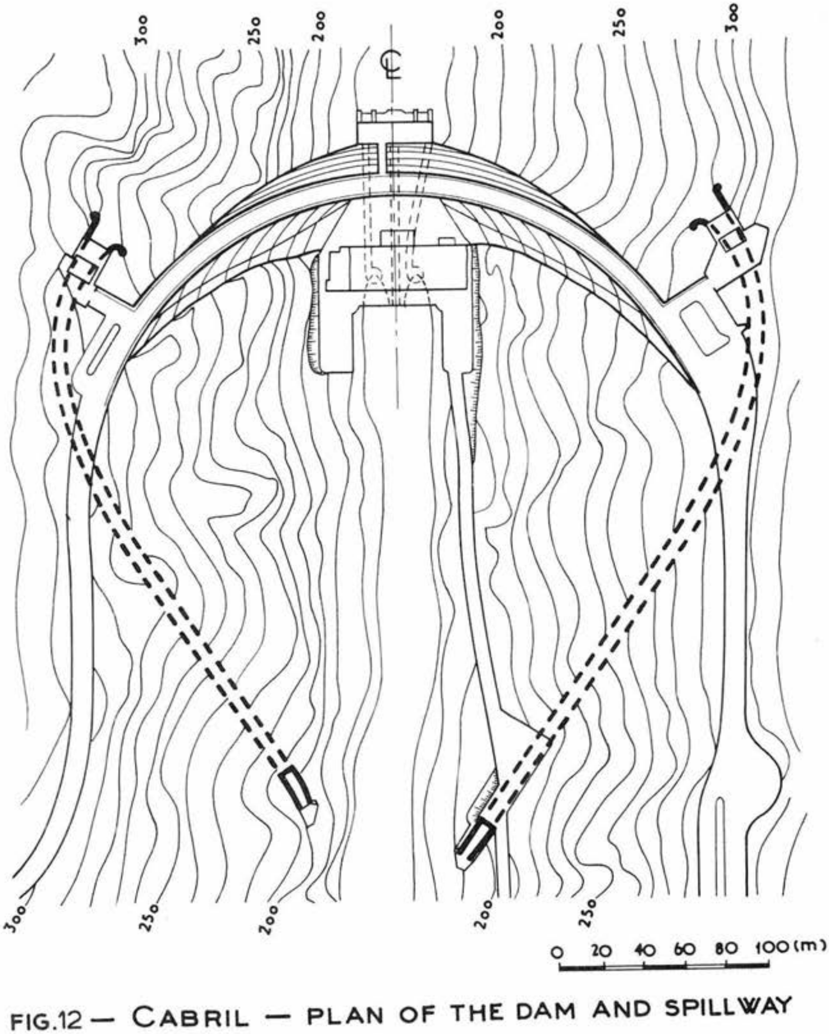


FIG.12 — CABRIL — PLAN OF THE DAM AND SPILLWAY

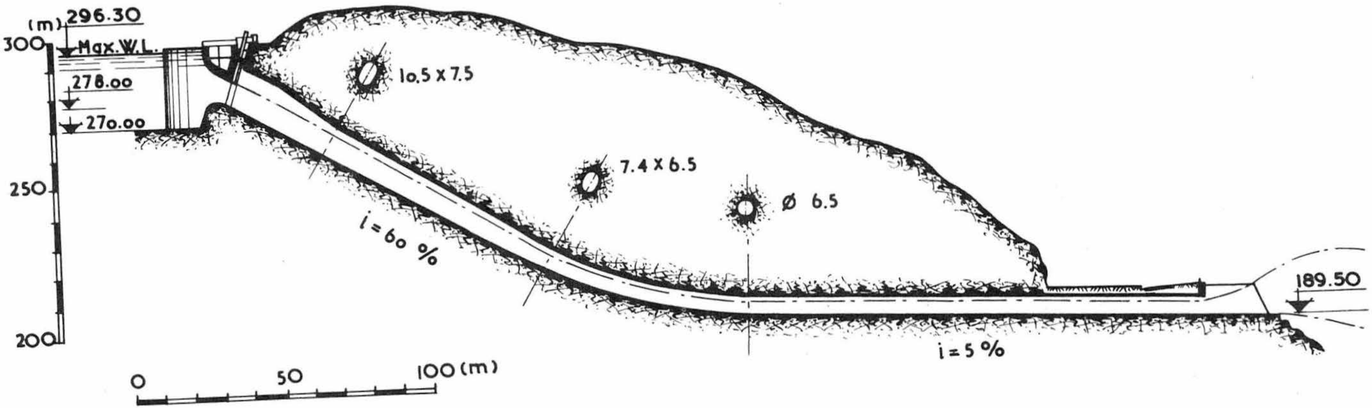


FIG.13 — CABRIL — SECTION THROUGH THE SPILLWAY TUNNEL

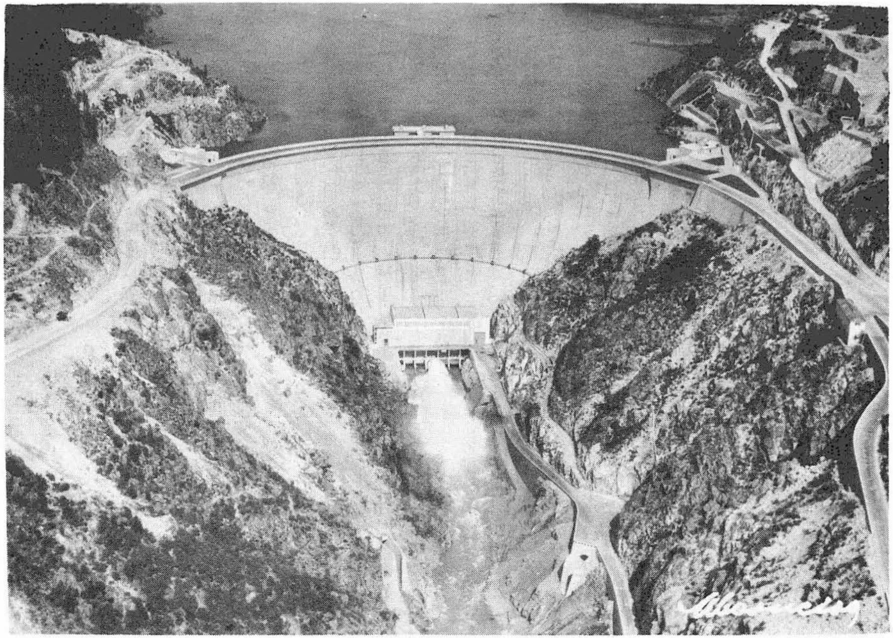


Fig. 14 - CABRIL - Downstream view of the dam, power station and outlets of the tunnels.

to the conclusions drawn at the 5th Congress on Large Dams when it was stated that the choice of the type of dam to build must always be considered case by case and depends on many factors besides the specific one of the actual design of the dam. In effect, the solution to be adopted for the dam and flood discharge arrangements in this case depends fundamentally on the location of the power station and on the problem of the restitution of the water from the turbines downstream.

Without discussing the various possible solutions which were considered before arriving at the final one it may be said that studies were conducted envisaging an underground power station with the restitution of the turbine water upstream of the flood discharge zone. This solution has the advantage of minimising the loss of head during floods and of making this restitution in a zone where there is no disturbance. It was possible to conjugate these requirements in a project involving the building of a thin arch dam where the floods are discharged over the whole crest through four openings each 20.00 x 15.00 m (65.62 x 49.21 ft). The throwing of the jet downstream is satisfactorily made by means of a ski-jump spillway* under which the station tunnel discharges (Figs. 15, 16).

As a result of this layout it was necessary to establish two independent structures having different elastic characteristics: the arch dam of the best possible design within the limits imposed by the topography of the site and with a crest shape in plan and profile determined by means of hydraulic model tests and conveniently arranged for the installation of the radial gates; and the ski-jump spillway which is to be supported by a buttress structure.

The ski-jump spillway design was carefully checked through elaborate hydraulic model tests (Figs. 17, 18). During these tests the shape of the spillway was thoroughly studied, specially the convergence in plan of the side walls, the inclination of the slab, the angle of throw and the special shape to be given to the dentated sill so as to have the best possible aeration of the jet and, consequently, the best energy dissipation. The jet is thrown centrally in the valley and is widely dispersed as a result of the action of the dentated sill. Velocity distribution in the river is satisfactory, the greater part of the flow being concentrated in the centre with lower velocities near the banks as required.

As regards constructional details particular reference should be made to the longitudinal expansion joint between the crest of the dam and the buttress slab which must ensure not only the continuity of flow but also the independent movements of the two structures. The crest of the dam supports the piers whose shape have been established to permit perfect hydraulic flow; these piers are reinforced concrete structures to which the control gates of the radial type are tied. Results of the model tests now in course will give an indication of the stresses in every part of this delicate zone of the arch structure to enable the best constructional solutions to be chosen.

Before finishing attention is drawn to the fact that although contribution of

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* After the above project had been designed the writer has learnt through the kindness of eng. Carlo Semenza of SADE (Italy) that an analogous solution for the flood discharge was studied, but not built, with regard to the Sottosella arch dam.

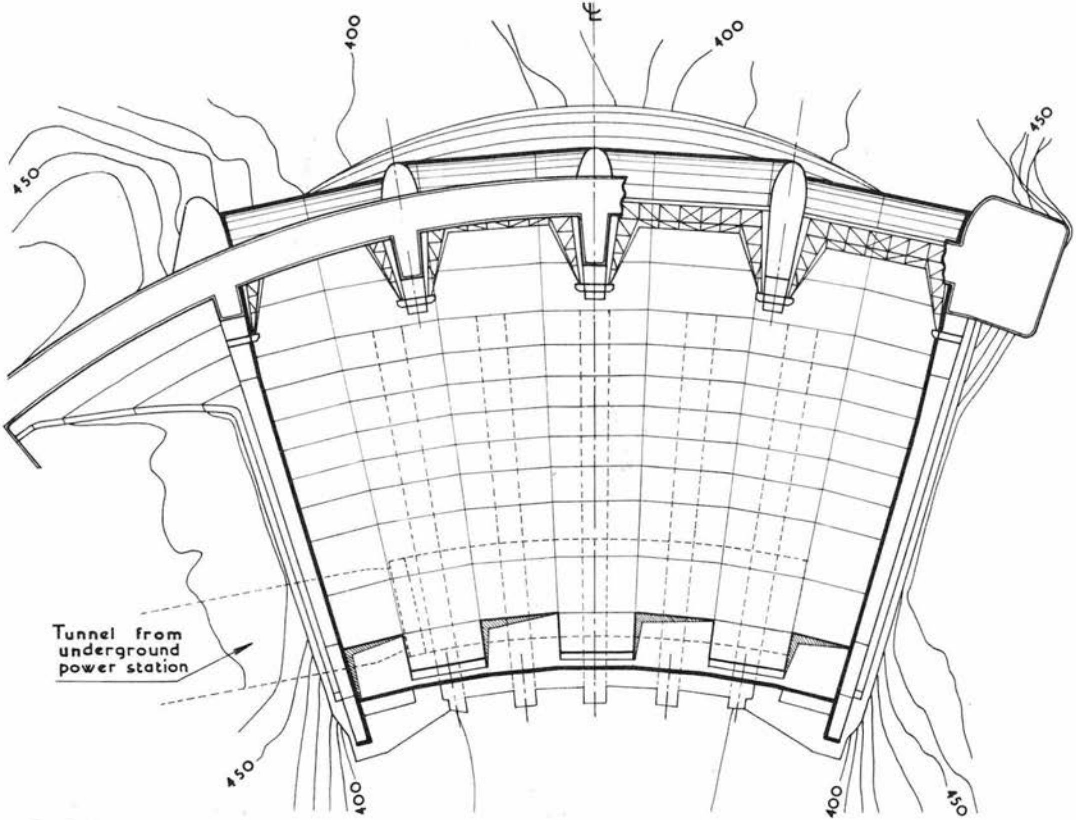

FIG.15 — PICOTE — PLAN OF THE DAM AND SPILLWAY

0 5 10 15 20 25 30(m)

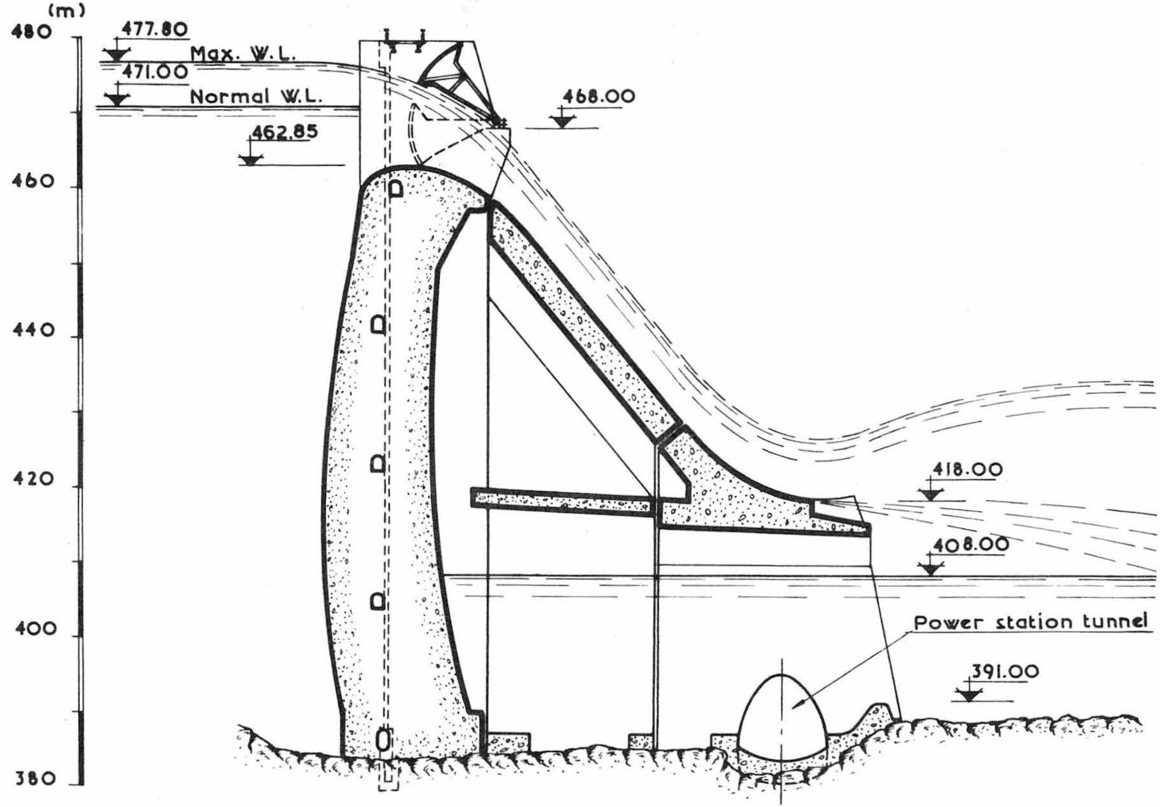

FIG.16 — PICOTE — CENTRAL PROFILE OF THE DAM AND SPILLWAY

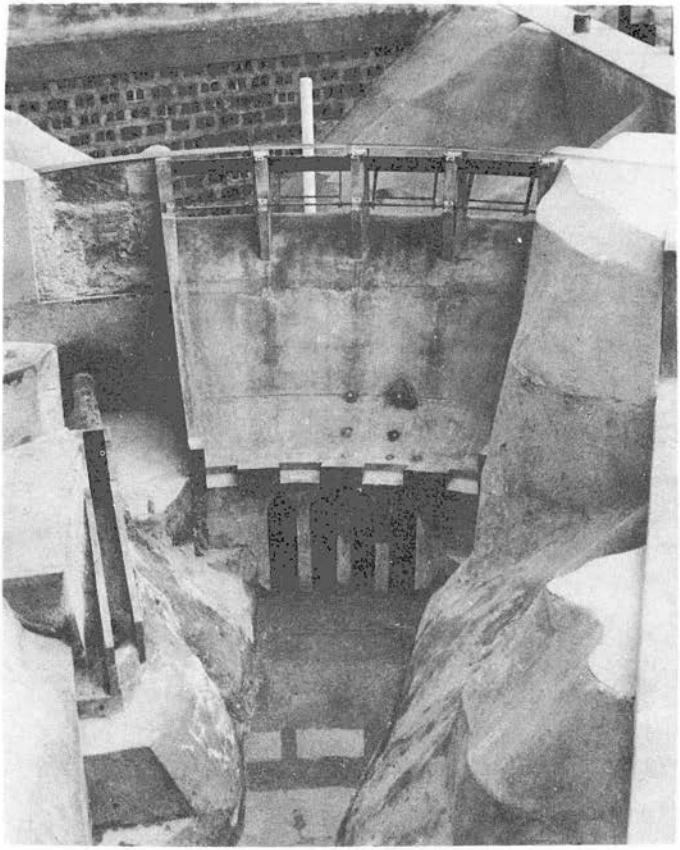

Fig. 17 - PICOTE - Hydraulic model (Hydraulic Laboratory directed and operated by the Civil Engineering Department of HIDRO-ELÉCTRICA DO ZEZERE).

Fig. 18 - PICOTE - Hydraulic model in operation (Hydraulic Laborator directed and operated by the Civil Engineering Department of HIDRO-ELECTRICA DO ZEZERE).

Portuguese engineers to the development of the technique of arch dams is very small when compared with that from other parts in the world, it should be stressed however that the studies which have lately been carried out in Portugal on overflow arch dams have been conducted within the best technical principles of economy and safety.

NOTE:—The works of Cabril, Bouçã and Castelo do Bode constitute the hydroelectric development in the river Zezere of which the Hidro-Eléctrica do Zezere are concessionaires; those of Venda Nova, Salamonde and Caniçada in the rivers Cávado and Rabagão form part of the concession held by the Hidro Eléctrica do Cávado; the dam which is being built at Picote is included in the development of the Douro river scheme awarded to Hidro-Eléctrica do Douro. Castelo do Bode, Venda Nova and Salamonde dams were designed by the French consulting engineer A. Coyne; Caniçada dam was designed by the Civil Engineering Department of the Hidro-Eléctrica do Cávado; Cabril and Bouçã dams as well as Picote were designed by the Civil Engineering Department of the Hidro-Eléctrica do Zezere.

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Journal of the
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ARCH DAMS: THEORY; METHODS, AND DETAILS OF JOINT GROUTING

A. Warren Simonds,[1] M. ASCE

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams held in April, 1939, much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be included in the Symposium. So far, nine papers have been approved: "Arch Dams: Trial Load Studies for Hungry Horse Dam" (Proc. Paper 960) by R. E. Glover and Merlin D. Copen; "Arch Dams: Portuguese Experience with Overflow Arch Dams" (Proc. Paper 990) by A. C. Xerez; "Arch Dams: Theory, Methods, and Details of Joint Grouting" (Proc. Paper 991) by A. Warren Simonds; "Arch Dams: Santa Giustina Single-Curvature Arch Dam" (Proc. Paper 992) by Claudio Marcello; "Arch Dams: Measurements and Studies on Santa Giustina Dam" (Proc. Paper 993) by Claudio Marcello; "Arch Dams: The Reno De Lei Double-Curvature Arch Dam" (Proc. Paper 994) by Claudio Marcello; and "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs" (Proc. Paper 996) by Claudio Marcello.

As other papers are approved, they will be published in the Proceedings. The interested readersshould watch for these papers in following issues of the Journal of the Power Division.

SYNOPSIS

In the construction of arch dams, the present practice of the Bureau of Reclamation is to build the structure in blocks which are bounded by radial joints keyed to resist shear deformations. The purpose of the joints is to permit shrinkage of the concrete, due to dissipation of the setting heat of the cement, to take place without developing irregular cracks throughout the

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structure. In order for arch action to take place, it is essential that no structural discontinuities exist at the time the reservoir load is applied. To make possible the development of arch action during the early stages of reservoir filling and to improve the stress distribution for the "in-service" condition of the dam, a procedure has been developed whereby the contraction joints of arch dams are pressure-grouted, thereby making the structure monolithic. Metal seals capable of expansion and contraction are installed around the peripheries of the joints or areas to be grouted so as to hold the grout in the joint. Special systems of piping with outlets to the faces of the joint are used for conveying the fluid grout into the joints. A technique has been developed whereby abnormal stresses due to grout pressure can be controlled so as not to damage the structure, and an effective job of grouting will result.

INTRODUCTION

An arch dam, as referred to in this paper, is a solid concrete or masonry dam which is curved upstream in plan, and, because of its shape, carries the reservoir waterload partly by its own weight and also by transmitting the remainder of the waterload to the canyon walls by arch action. In order for arch action to develop in the structure, it is essential that no structural discontinuities exist at the time the waterload is applied as the reservoir fills. Sound abutments which are susceptible to only minor deformation are necessary requirements for an arch dam.

An intensive study of the structural action of arch dams was inaugurated in the United States during the early part of this century. It had been realized by designers that contraction joints were valuable for the control of cracking which oftentimes resulted in undesirable leakage. The opening and closing of contraction joints had been observed at the Salmon Creek Dam in Alaska, and an extensive investigation was made of the behavior of that structure due to changes in temperature and waterload conditions.

As the result of these observations, it was concluded that arch action did not develop until after sufficient load had been applied to the structure to close any open contraction joints and the blocks forming the voussoirs of the arch were in bearing. It was concluded that the two main reasons for the phenomenon that an arch dam does not act as an arch before considerable load has been applied to it were: (1) the shortening of the arch rib due to shrinkage of the mass concrete; and (2) the bond of the concrete of the structure to the foundation.

It is obvious that if an arch dam could be constructed in such a way that arch action would begin with the application of reservoir pressure on the upstream face of the dam, a better distribution of stress throughout the structure would result. To obtain this condition a suitable means of bringing the faces of the blocks of the arch into contact before the application of waterload was sought. Two methods were developed: (1) by leaving open slots between the blocks of the arch dam and constructing closing plugs at a time of no reservoir load on the arch and when minimum temperature conditions of the concrete prevailed; and (2) by grouting the open joints under pressure at a time of maximum joint opening before the reservoir waterload was applied.

Typical early examples of these approaches to closures of the arch elements were at Arrowrock Dam where shafts 6 by 6 feet were constructed across the joints and later filled with lean concrete; at Gerber, Waterville, and Ariel Dams where closure plugs were used; and at Bullards Bar, Cushman No. 1, Pocoima, and Lake Spaulding Dams where the contraction joints were grouted. It was during the construction of these latter dams when the use of embedded piping systems installed especially for grouting contraction joints was developed.

History of Joint Grouting Development by Bureau of Reclamation

The first dam in which the contraction joints were grouted to be built by the Bureau of Reclamation was Gibson Dam on the Sun River Project, Montana, which is shown in Figure 1. This dam is a concrete arch having a structural height of 195.5 feet, base width at the crown section 87 feet, crest width 15 feet, crest length 960 feet. Its volume in cubic yards is 167,500. The dam was built during the period of 1926-8, and the contraction joints were grouted in 1930. Because this dam was the forerunner of a number of large dams to be built by the Bureau, an extensive program of research on structural behavior of the arch was undertaken. This included studies of temperature, deflection, and hydrostatic uplift pressure on the base of the structure. Special installations were made at the dam to check various design theories and assumptions. Studies were made of the joint treatment, and elaborate installations for grouting the contraction joints were made.

The grouting of the contraction joints presented a number of problems, and a systematic study of these was inaugurated by the Bureau in 1927. It was realized that when a contraction joint having considerable area was filled with fluid grout, an enormous total pressure could be built up on the faces of the joint by applying even small increments of pressure on the fluid grout. If not carefully controlled, the pressure on the grout could be increased easily until there was danger of damaging the structure. Since an arch dam built with radial contraction joints is designed to be safe when waterload is applied at its upstream face, such a structure may not be stable when excessive pressure is built up in a contraction joint. Excessive pressure in a joint may force the joint open by bending the blocks laterally against neighboring blocks; it may shear a block off along the plane of a horizontal construction joint, or it may force the adjacent blocks to bend upstream, thereby causing tensile cantilever stresses at the downstream face with resultant cracking.

The problems considered in the study of contraction joint grouting procedure included the following:

- Opening of contraction joints
- Unit weight of grout
- Air entrapped between grout outlets
- Tangential deflection of crown cantilever
- Effect of grouting different joints
- Radial deflection and tension areas
- Concrete temperature measurements
- Best time to grout joints

The following is a summary of the results of this investigation. It was concluded that joints opened between 1/16- and 1/8-inch could be grouted readily. The unit weight of the fluid grout was dependent on the water-cement ratio used. Under normal conditions, the fluid grout used weighed about 105 pounds per cubic foot. In the systems of piping and outlets first used, there was a possibility of entrapping air between the grout outlets; however, by injecting

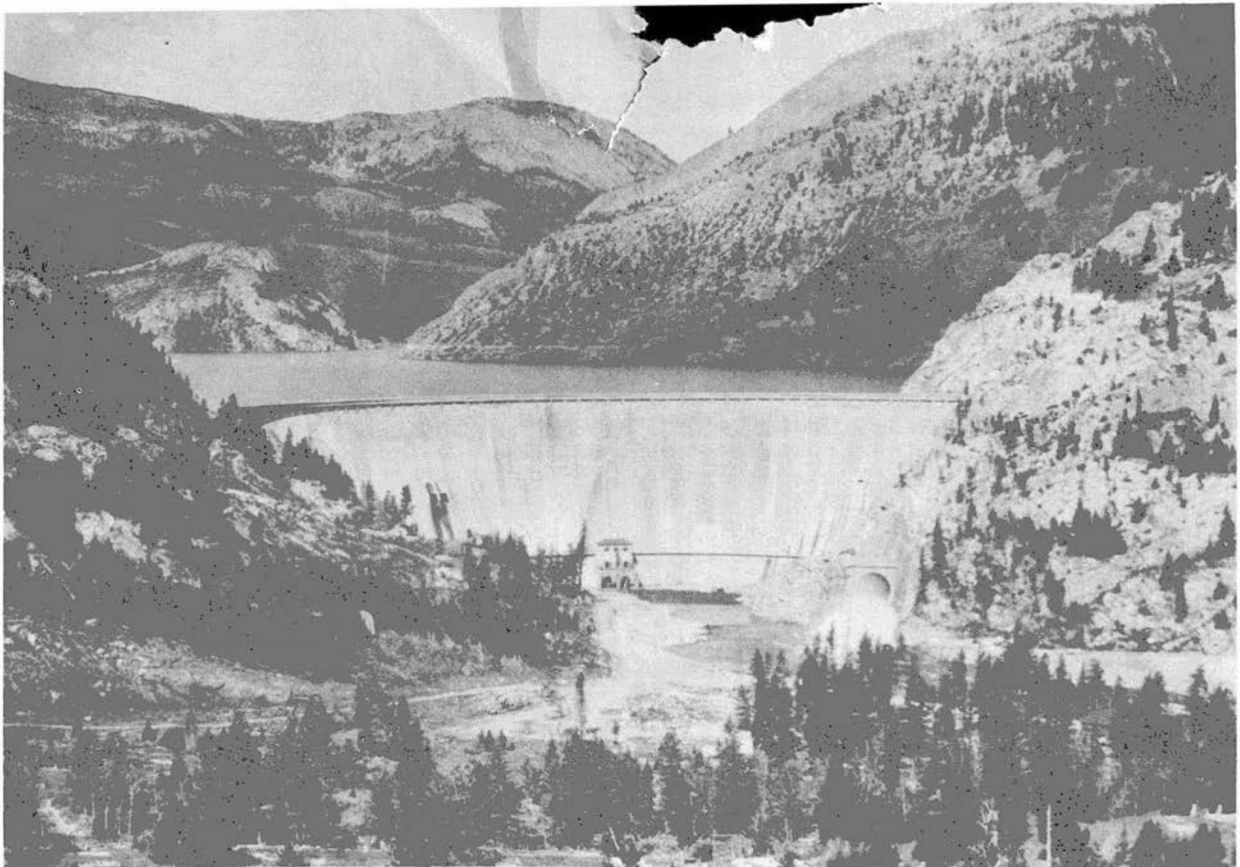


Fig. 1 General View of Gibson Dam, Sun River Project, Montana

the grout slowly, the fluid grout could settle in joints and the air could escape through the outlets in the upper part of the systems. From the study of the deflection of the central blocks in tangential directions when the joint at the cantilever section was filled with fluid grout, it was concluded that the blocks would deflect sufficiently to close the adjacent ungrouted joints. This difficulty could be controlled by filling the adjacent joints with water so as to balance the grout pressure. Studies were made also of grouting the joints along the abutments where the heights of the joints near the ends of the dam became progressively less. In order to control the upstream deflection of the blocks which would be accompanied by the development of tensile cantilever stresses at the downstream face, a system of observations using theodolite reference lines, was evolved. Studies of temperature conditions were made and charts showing the range of temperature variation were plotted so that the time of minimum temperature could be determined. Since there was no provision for artificial cooling of concrete at Gibson Dam, the temperature of the mass concrete was dependent on weather conditions, and the exact minimum could not be determined except by observation. However, it appeared that during the late spring, the temperature would be close enough to the minimum to permit a satisfactory condition for grouting.

Radial Joints

The first designs of contraction joints for arch dams provided for radial joints to be constructed at some specified interval. In constant-radius arch dams, the joints were laid out radial to the axis. In variable-radius and constant-angle types of arches, the joints were designed either curved in plan or with two straight sections at an angle, connected by a curved section so that the upstream part of the joint was radial to either the axis or the upstream face, and the downstream part was radial to the downstream face. In either case, it has proved desirable to construct the joints with keys.

The joint keys developed by the Bureau of Reclamation have now been standardized and are used in the construction of all solid concrete dams. Figure 2 shows the arrangement of keys at the face of one of the blocks of Shasta Dam. Although Shasta Dam is a curved gravity dam, the keys as developed there, are the standard design of the Bureau for arch dams. The keys are dimensioned so as to give equal shearing resistance between the two adjacent joints. This standard key offers the least obstruction to the flow of grout, provides a greater theoretical shear value, eliminates sharp corners which are cracked frequently by form removal, and improves the reentrant angles conducive to crack development resulting from volume change.

As now developed, the fluid grout is injected into the contraction joints through an embedded system of piping and circular outlet units which are spaced uniformly over the face of the joint. The outlet units comprised of two modified electrical conduit boxes, are connected by tees to vertical riser pipes which are attached to a supply header. This system is installed in the high block. The supply header is mounted on metal supports and the outlet units without covers are attached to the concrete forms. After embedment when the forms are stripped, the covers of the units are attached to their companion members before the concrete of the corresponding lift in the adjacent low block is placed.

In the development of grouting systems, a study and some experimenting have been done to determine a satisfactory height for a grouting lift.

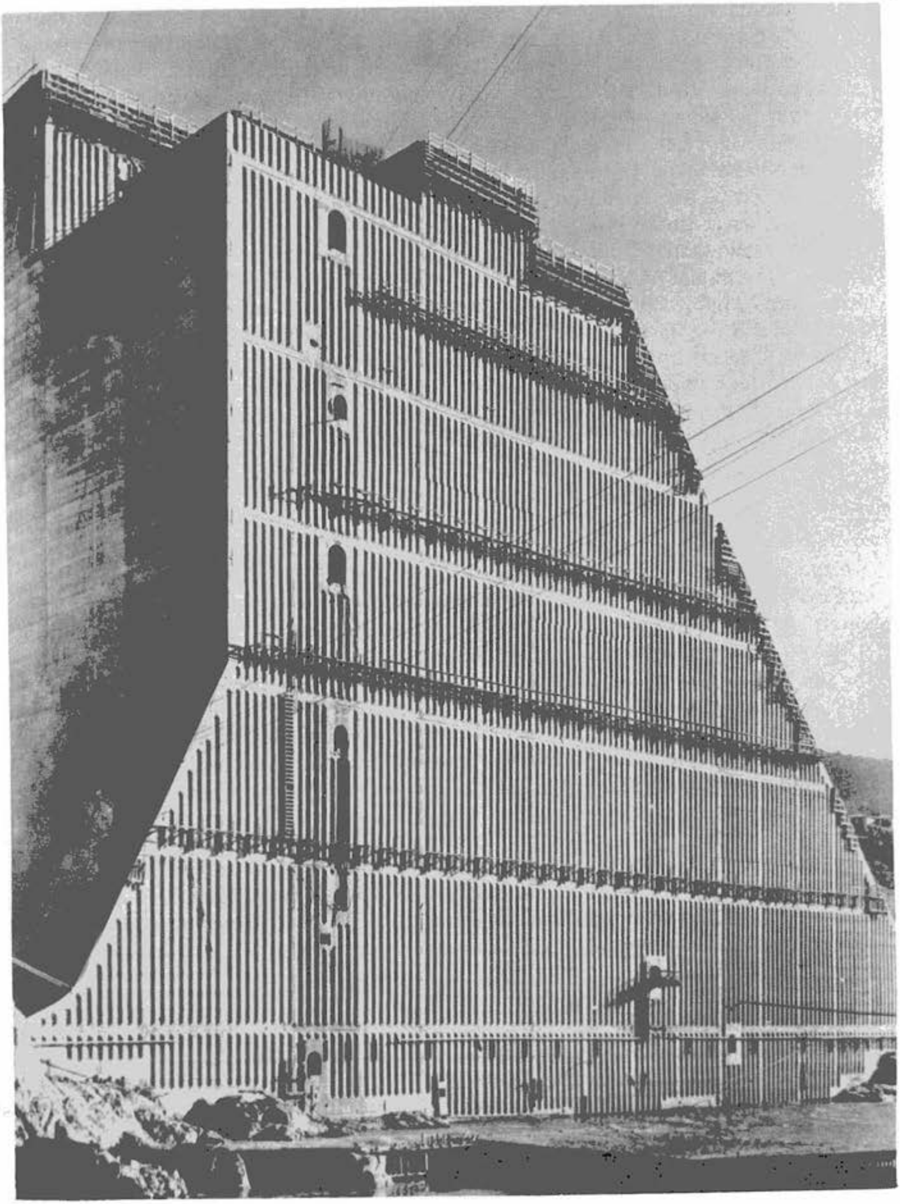


Fig. 2 Face of Joint in Shasta Dam Showing Keys, Central Valley Project, California

Originally the grouting was done from the bedrock to the top of the dam through heights ranging from 25 to 175 feet. Because of the difficulties encountered in grouting through the greater heights, the standard practice now is to provide for grouting through vertical lifts of about 50 feet. A moderate variation from this height does not affect the grouting procedure.

In order to contain the grout in the joint, metal sealing strips around the periphery of the joint are installed. Horizontal sealing strips are used to divide the joints into vertical lifts. The sealing strips are of lightweight metal, preferably either of soft tempered copper, 24-ounce weight per square foot, 0.032-inch thick, or annealed stainless steel No. 20 gage, 0.0375-inch thick. The seals are usually installed in an M- or Z-shape. Connections are made by brazing or welding.

A more recent development consists of installing a vent groove at the top of each grouting lift just below the horizontal seal. This provides for the escape of air and excess water from the grout during the filling of the joint. When difficulties are encountered and it can be anticipated that regrouting at the joint will be necessary, the vent can be used for injecting grout into the joint.

Theory of Grouting Contraction Joints

When arch dams are to be constructed with radial contraction joints which are to be grouted, a stress and stability analysis should be made of the structure before grouting operations are initiated. The blocks of an arch dam are designed to be statically stable under the waterload after the contraction joints are grouted. Before the joints are grouted, severe stress conditions may be set up due to the pressure of fluid grout in the joints.

The forces acting on the radial faces of a block in an arch dam due to fluid grout in a contraction joint vary with the procedure with which the grouting is done. If an individual joint is to be grouted alone, the pressure of the fluid grout in a circumferential direction may be sufficient to force the joint open by bending the blocks over against their adjacent neighbors, thereby squeezing the adjacent joints shut. If a series of joints is to be grouted simultaneously, the excessive bending of the blocks in a circumferential direction may be controlled and the bending stresses held to a minimum. In small arch dams, the usual practice is to grout all joints in one lift from abutment to abutment in one operation. In larger arch dams the joints are grouted in groups; the abutment groups are grouted first and the center groups last.

Where the planned procedure for grouting the contraction joints is to grout all the radial joints from abutment to abutment, different stress conditions are developed. Consider the forces acting on an individual block of arch dam due to grout pressure in the radial contraction joints as shown in Figure 3. The resultant of the forces acting on the faces of an individual block acts in an upstream direction and tends to bend the block as a cantilever upstream. The resultant may be sufficient to cause high tensile cantilever stresses to develop on the downstream side of the structure. These can be controlled to a large extent by a partial filling of the reservoir to balance the resultant of the grout pressure.

Since the pressure of fluid grout in the contraction joints of an arch dam may cause considerable damage if not adequately controlled, an important factor that must be considered in the design of a grouting system for an arch dam is the maximum expected grout pressure. Laboratory investigations of

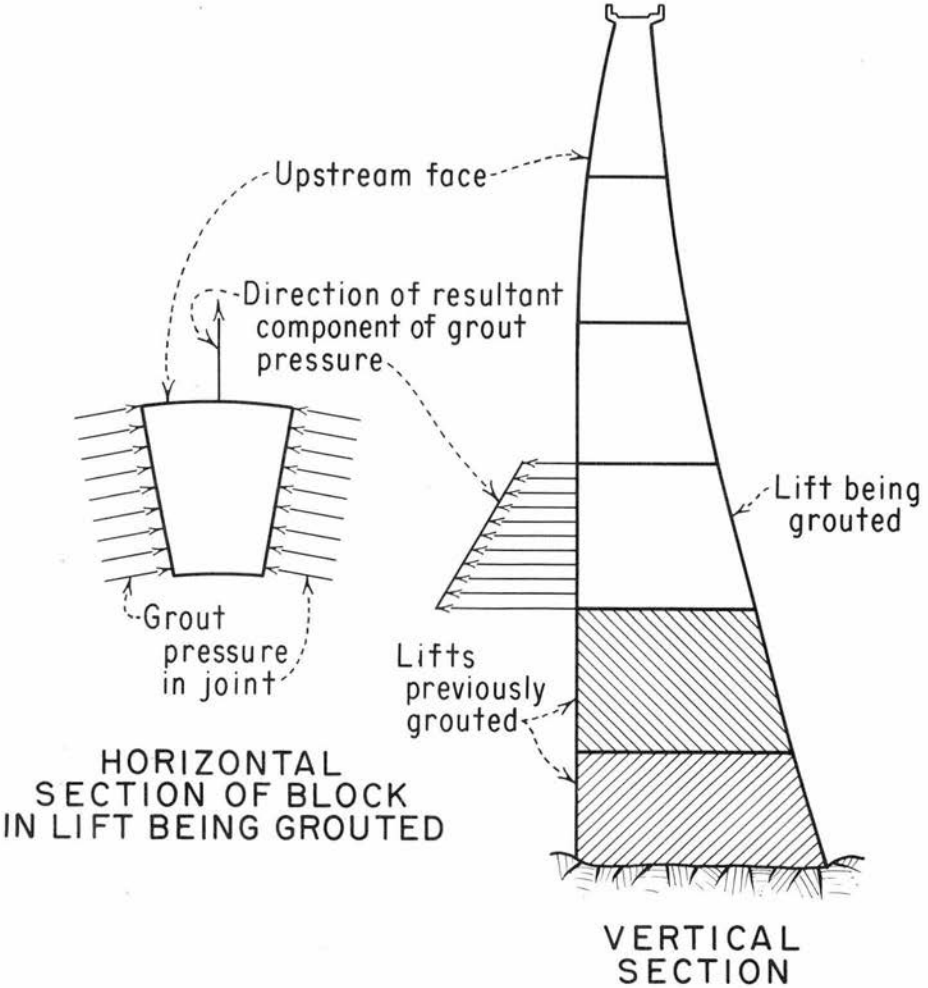


Fig. 3 Forces Acting on a Block of an Arch Dam Due to Grout Pressure

neat cement grouts have indicated that when they are allowed to set under pressures of 30 to 50 psi good films of grout will result. Most contraction joint grouting systems are therefore presently designed so that excessive stresses will not occur when a grouting lift filled with fluid grout has pressures of 30 to 50 psi at the top of the lift. This pressure is usually measured at the vent at the top of the lift being grouted.

The effect of pressures in the contraction joints is to cause the blocks of the arch dam to deflect and, because of this, observations for deflection may be used beneficially for controlling grouting operations. There are two general types of observations: (1) upstream deflection of the dam, and (2) change in width of joint opening. The first type of observation consists in measuring the deflection of some point on the top of the dam at the crown cantilever section with reference to a fixed line normal to the radius of the axis at that point. A theodolite is used in making observations of this type.

The second type of observation consists in measuring the change in width of joint opening. This is done by means of dial gages which register to 0.0001 inch. The gages are mounted usually across the contraction joints at the top of the lift being grouted. An increase in the width of joint opening at the top of a 50-foot-high lift being grouted should be not greater than 0.0200 inch under normal conditions. Occasionally when the grout pressure gets out of balance between joints, one joint will open while an adjacent joint will close. Dial gages will indicate this occurrence and the pressures on the grout can be corrected.

Development of Block Construction

In order to control temperature stresses so as to prevent cracking, it has been found that contraction joints or "formed cracks" must be provided at frequent intervals if cracking of the concrete is to be avoided. Originally, the joints were spaced at intervals of 30 to 60 feet. When dams are of exceedingly large sections such as Hoover, Shasta, and Hungry Horse Dams, it is necessary to provide joints in two directions to control cracking.

In the design of Hoover Dam, limitations on maximum and minimum block sizes influenced the contraction joint design. If the blocks were too large, cracking might occur within the blocks. On the other hand, if the blocks were too small, shrinkage might be so slight that the contraction joints between blocks would not open far enough to take grout. After a study of the conditions under which the grouting would be done, a two-way system of joints extending in radial and circumferential directions was developed. These joints were spaced so that they would be not less than 25 feet apart at the downstream face of the dam and 65 feet apart at the upstream face.

From stress considerations, it was desirable to have continuous radial joints and staggered circumferential joints as shown in Figure 4. All blocks extended vertically from the rock contact at the base of the dam to the intersection with the upstream or downstream face or the top of the dam. The lower part of the large radial joints in the central area of the dam were divided into two sections for convenience in grouting. The circumferential joints were designed so that there were three groups of joints between the abutments in each grouting lift, except at the extreme bottom and top of the dam where there were one and two groups respectively.

In the more recent designs of grouting systems, a number of improvements have been incorporated in the plans based on difficulties which had been

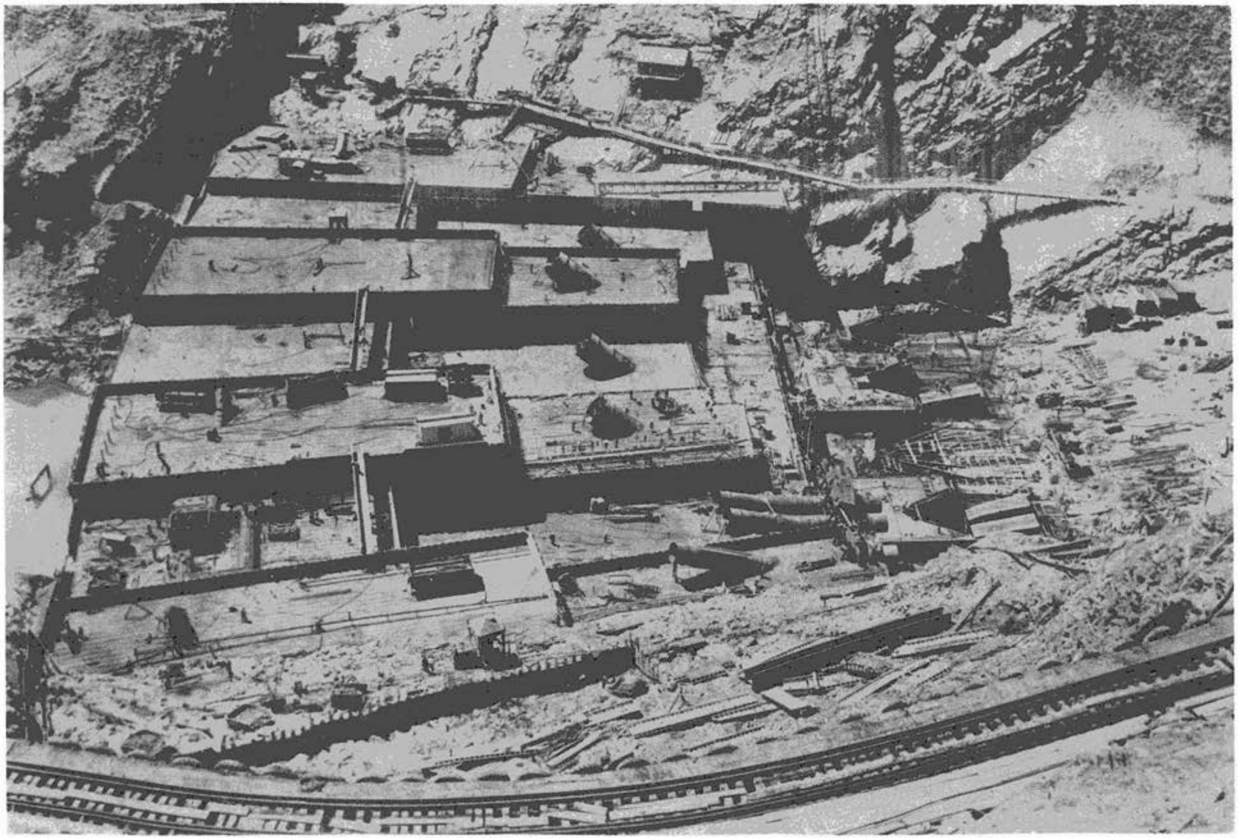


Fig. 4 Block Construction at Hungry Horse Dam, Hungry Horse Project, Montana

experienced previously. These include a more convenient arrangement of the galleries where used, a more effective system of vents at the top of the lift being grouted, and the installation of header drains to be used in cleaning the joints and systems prior to grouting.

A typical plan of a grouting system for a transverse joint is shown in Figure 5. Inasmuch as it is not practicable to provide special galleries in each 50-foot grouting lift in most arch dams for grouting the contraction joints only, the grouting is generally done from temporary cat walks installed on the downstream face. When a suitable gallery is constructed near the foundation of an arch dam for the foundation grouting and drainage, the headers of the piping systems for the contraction joints may be arranged so that the grouting of the lower lifts of the joints can be done from this gallery. In either case, a temporary supply line is constructed from the grout pump to connect to the supply headers of the contraction joints. The grout is injected through the supply headers and the embedded piping systems into the joints. As the grout fills the joints, any water in them is forced ahead of the grout into the vent at the top of the lift. At the downstream end of the vent, a length of pipe about 5 feet long is installed vertically to serve as a temporary standpipe. A pressure gage and valve are installed at the upper end of the standpipe.

The grouting systems for the longitudinal joints are in sections measuring 50 by 50 feet because of the discontinuity that results from the staggered joints in the block pattern. A typical elevation of the grouting system across one block is shown in Figure 6. The grout is injected into the longitudinal system through the supply header and enters each section of the joint through the piping system. Outlets are spaced at intervals along the horizontal keys. The excess water and thin grout are drained from the vent groove at the top of the lift.

If some essential header should become plugged accidentally during construction, and the header could not be utilized, the system is designed so that an alternate header could be used. The plan of the longitudinal headers is shown in Figure 7. Normally the grout would be injected into the supply header; if the supply header could not be used, the joints could be grouted through the 1-1/2-inch return header.

In grouting the longitudinal joints, the critical areas where the pressure must be accurately controlled, are the partial lift joints terminating the downstream face and the joints nearest the upstream face of the dam. In order to provide adequate control for grouting these joints, separate grouting systems should be installed for each set of joints. The grouting of these joints can be done at the same time the regular longitudinal joints in the zone are grouted but individual control of each partial lift is necessary.

The vent system of the longitudinal joints is shown in Figure 8. This system is arranged so that each group 50-by-50-foot sections has a vent return. Riser pipes serve as standpipes to hold hydrostatic pressure on the grout in the joint, similar to the function of the risers on the vents in the transverse system.

Cement Used in Contraction Joint Grouting

The cement used in contraction joint grouting includes normal, modified, and low-heat portland cements, and occasionally special cements such as high-early-strength cement and special grinds of oil-well-grouting cement. The choice of type of cement used depends on field conditions in which

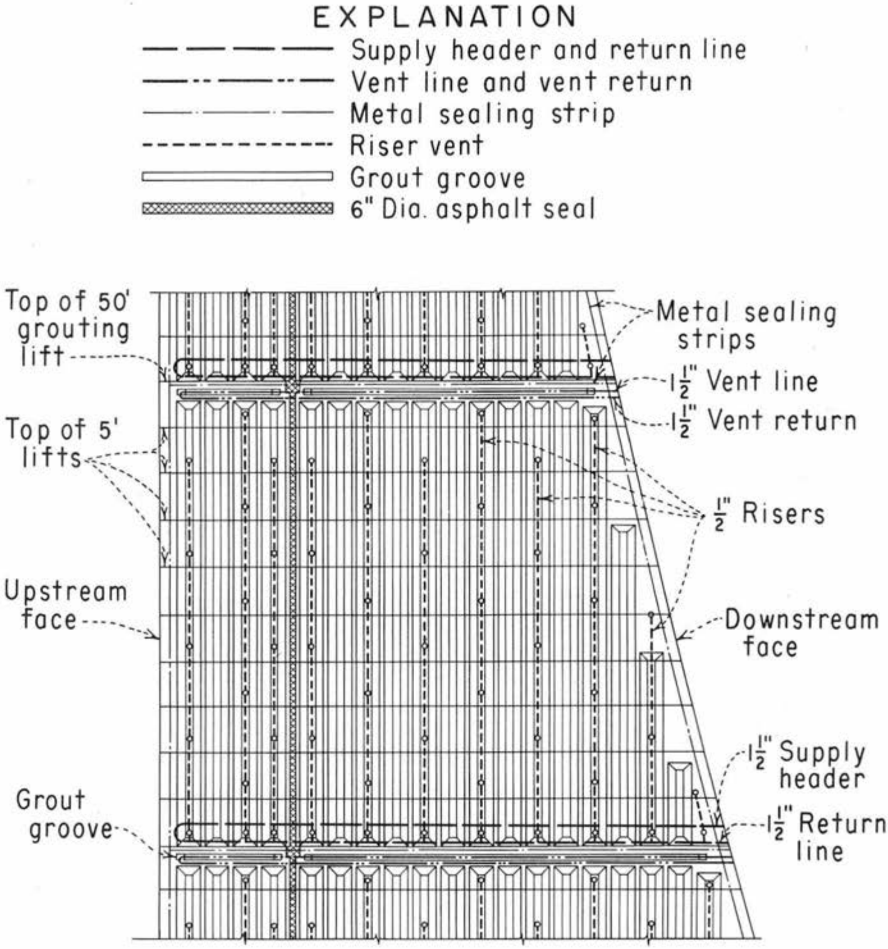
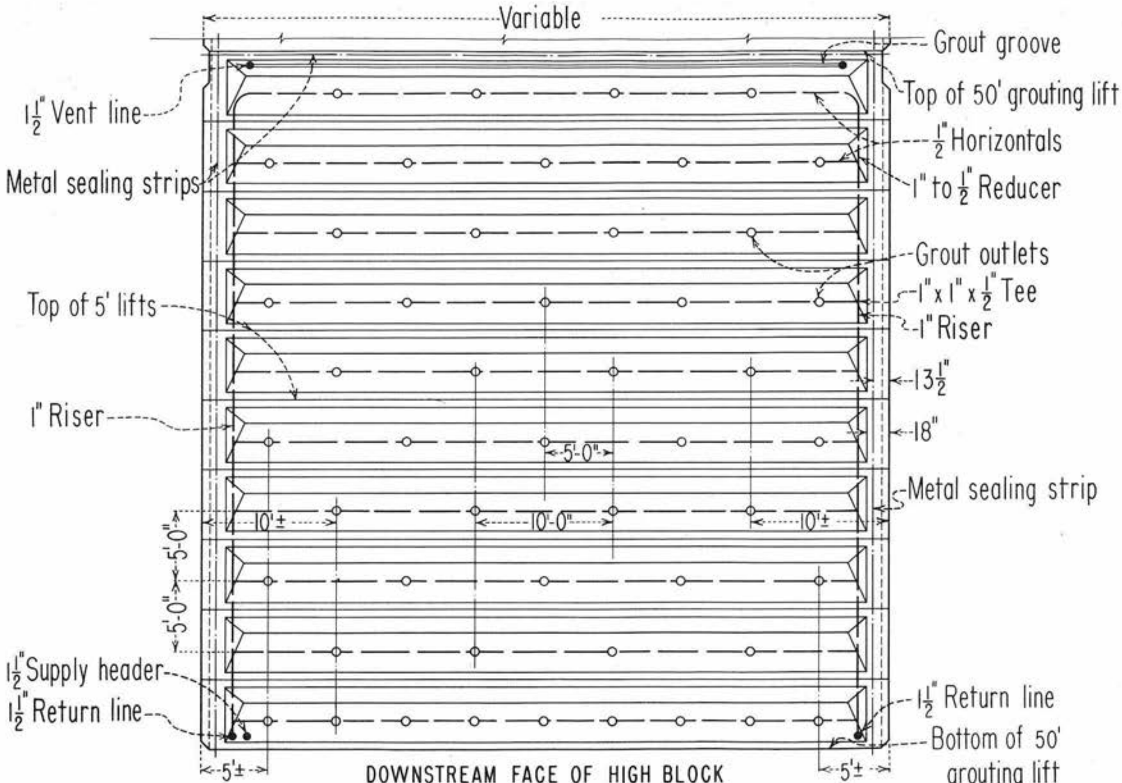


FIG. 5.

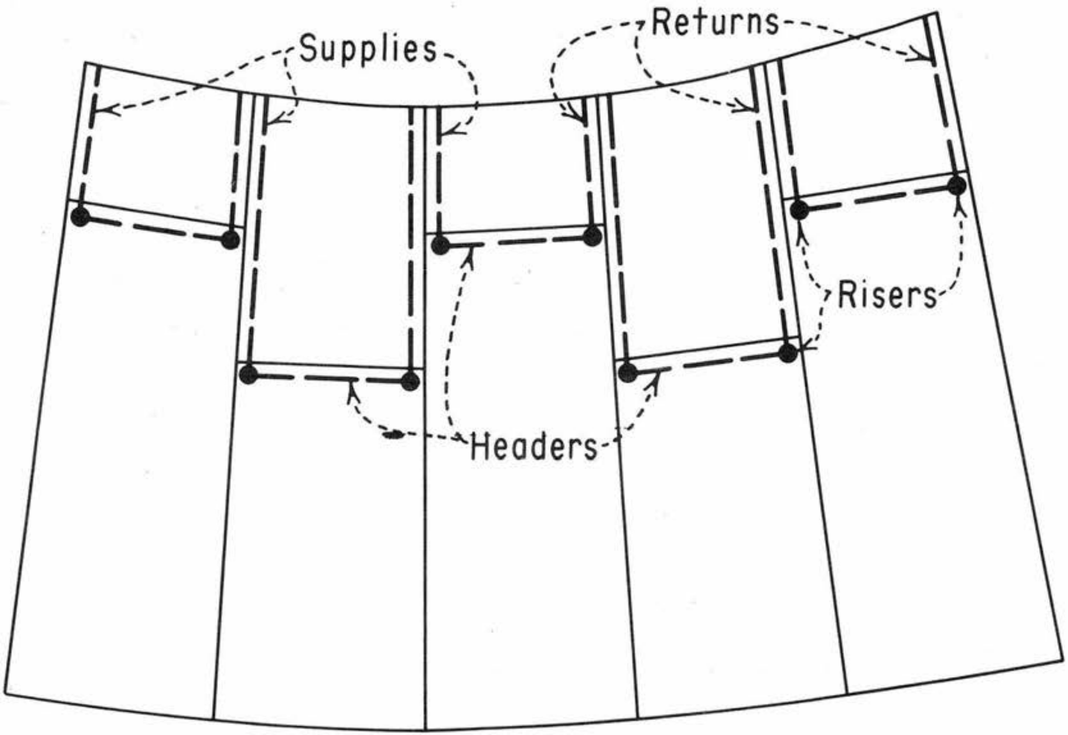
ELEVATION OF
TRANSVERSE CONTRACTION JOINT
SHOWING HEADER AND RISER PIPE LAYOUT



DOWNSTREAM FACE OF HIGH BLOCK
(TRANSVERS HEADERS NOT SHOWN)

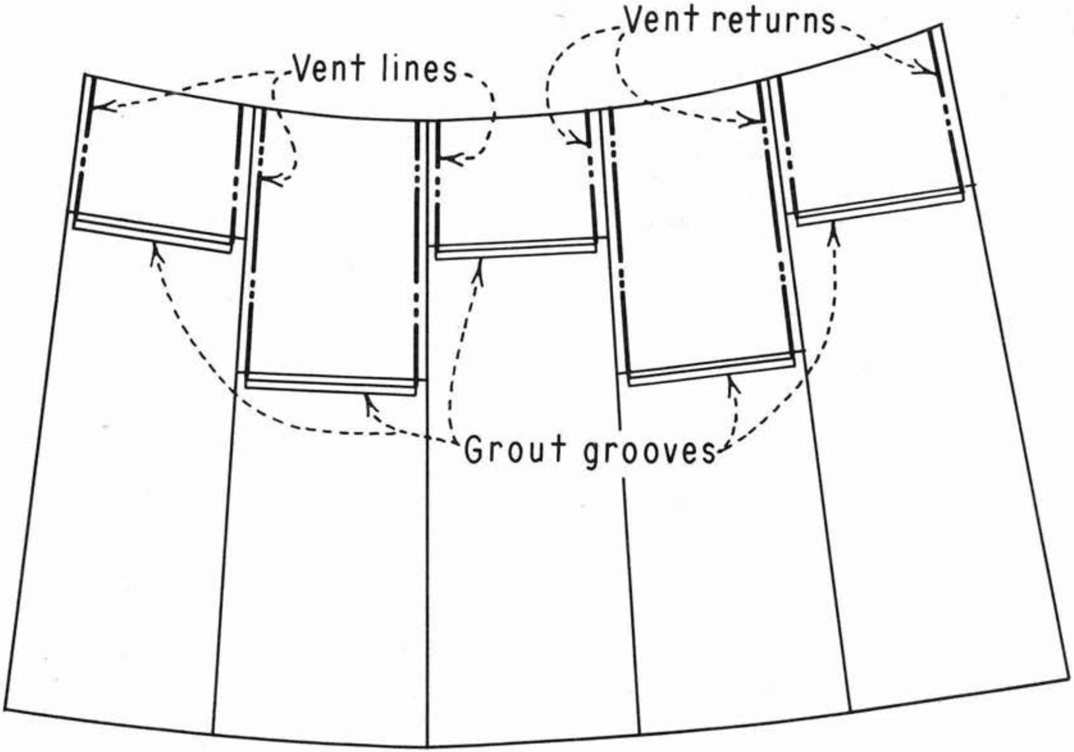
TYPICAL ELEVATION OF
LONGITUDINAL GROUTING SYSTEM

FIG. 6.



TYPICAL PLAN OF GROUT HEADERS
FOR LONGITUDINAL CONTRACTION JOINTS

FIG. 7.



TYPICAL PLAN OF GROUT GROOVES AND VENT LINES FOR LONGITUDINAL CONTRACTION JOINTS

FIG. 8.

transportation, time of set, and ability to seal leaks in inaccessible locations are involved. In general, of these types, modified and low-heat portland cements and oil-well grouting cements have been the most satisfactory. The desirable qualities of a cement suitable for contraction joint grouting are: sufficient fineness for grouting joints where the width of opening is small; freedom from tramp iron, unground clinker, and other foreign matter; and sufficiently slow setting so that the grouting system will not plug before the joints are filled.

In order to obtain a cement suitable for contraction joint grouting, the Bureau of Reclamation originally processed the mill-run cement at the job. This was done by screening it through mechanical vibrating screens or processing it through an air separator. This processing was effective in eliminating tramp iron, unground clinker, lumps due to warehouse set, and foreign matter. The specified fineness of a cement suitable for grouting contraction joints provides that 100 percent of the cement should pass a No. 100 U.S. standard screen and 98 percent should pass a No. 200 U.S. standard screen. In recent years, cements purchased from mills equipped with air separators have met the above specified fineness and have been used with satisfactory results. In the purchase of cement for contraction joint grouting, it has been found that specifying the fineness in terms of screen analysis is more satisfactory than specifying the fineness in terms of specific surface. In addition to fineness, the specifications should provide that the cement be furnished in waterproof bags which will prevent hydration from exposure and lumps due to warehouse set. Furthermore, the cement should not be held in storage longer than 90 days prior to using.

Grouting Procedure

As mentioned previously, the ideal procedure in grouting the contraction joints of an arch dam is to fill all joints from abutment to abutment to the same elevation simultaneously. However, it should be realized that the procedure adopted consists not only of the mechanics of filling the joints with grout but also in balancing the forces due to grout pressure when necessary. Therefore, it is necessary to be prepared to balance the thrust of the grout by holding water under pressure in any adjacent ungrouted joints to prevent excessive deflection.

When ideal conditions exist, the definite quantities of grout are injected into each joint consecutively in order that the joints will fill uniformly through small differentials in height. This process is repeated until the grouting lift of all joints is filled from abutment to abutment. This process eliminates excessive tipping of the blocks with the accompanying high stresses in the concrete. If some joints cannot be grouted because of irregularities of the construction program, or for other reasons, the joints can be prevented from closing during grouting operations by holding water under pressure in them.

When the grout injected into the joints reaches the top of a grouting lift, water is generally admitted to the grouting lift above. Water in the upper grouting lift serves several purposes: first, the water enables the grouting engineer to check for leaks past the horizontal sealing strip into the lift above by flushing water from the supply and return headers; second, the water reduces the unbalanced pressure on the sealing strip caused by the grout below the sealing strip, thereby retarding the development of possible leak; third, the water tends to spring the joint open permitting a more effective spread of the fluid grout in the lower lift.

In normal grouting operations, two sets of temporary piping are necessary: one set to supply grout to the joints, and the second set to supply water to the adjacent joints, when a water curtain is needed to balance the thrust of the grout pressure, and also to supply water to the upper lift of the joints being grouted. In small arch dams where the joints are built in radial directions only, the two piping systems are relatively simple to install. In large dams where the joints are built in transverse and longitudinal directions, the necessary piping systems may reach extensive proportions.

The equipment required for grouting contraction joints depends on the size of the dam and on the accessibility of the headers of the grouting systems. Experience has proven that an equipment unit consisting of one air-driven duplex 10- x 3- x 10-inch grout pump, one 20-cubic-foot-capacity grout mixer and one 20-cubic-foot-capacity agitator sump can meet easily the requirements for grouting 100,000 square feet of joint area in an 8-hour shift, if conditions are normal. In grouting contraction joints, costly delays caused by equipment failure may be minimized by providing a complete standby unit of equipment, which may be placed in service when needed.

The pressure used in contraction joint grouting depends upon the physical dimensions of the blocks of the dam. In general, the higher the pressure used in grouting, the denser is the resulting grout film. The maximum allowable grout pressure is the pressure that the blocks of the dam will withstand before deflecting excessively, or creating undesirable tensile or shearing stresses within the concrete. The allowable pressures at the top of a 50-foot grouting lift for present designs usually vary from 25 to 60 pounds per square inch at the vent header at the top of the lift.

When filling the joints, slow grouting is necessary. In grouting a series of joints, if the grout is injected intermittently, the grout has a better opportunity to settle in the joints, and the excess water and thin grout can be bled from the vent at the top of the lift. When all the joints of the group have been filled and when a flow of thick grout has been obtained at each vent, the valves on the vents are closed and the maximum allowable grout pressure is applied by injecting additional grout into the joint. As soon as the maximum allowable grout pressure is reached, the valve on the supply header is closed and the grout in the joint is held under pressure. As the excess water is slowly squeezed out of the grout into the adjoining concrete, the pressure at the vent drops. When the pressure drop occurs, more grout must be injected into the joints until the maximum allowable grout pressure is again reached. The process is repeated until the joints hold the maximum allowable grout pressure about 30 minutes without showing any appreciable pressure loss.

Grouting operations are governed by careful observation of the deflection of the blocks while grouting is in progress. Dial gages registering to 0.0001 inch are valuable. These are mounted across the contraction joints, to observe the amount of joint spreading. The magnitude of the permissible deflection of the blocks in a dam depends upon the dimensions of the blocks. In general, a contraction joint under pressure should not increase in width more than 0.02 inch at the top of a 50-foot lift. Undesirable tensile or shearing stresses may develop if greater spreading occurs.

Results

At the time Shasta Dam was being designed, the question of relative efficiency of plane versus keyed grouted contraction joints in mass concrete came up, and a laboratory program of testing grouted joints was undertaken.

The specimens consisted of 10- by 25-inch cores drilled from concrete blocks 25 inches square and 48 inches high. These blocks were prepared by bolting two triangular blocks 48 inches high together with a 0.05-inch separation shim between them and grouting this 0.05-inch space. This method produced a grouted joint at an angle of 45^{o} with the axis of the cores. Three cores were drilled from each of three blocks after being grouted at 50, 30, and 10 psi with pressures maintained for 30 minutes. One core from each block was drilled and tested when the grout was 28 days old, and the two remaining cores were drilled and tested when the grout was 90 days and 123 days old. Six cores were also drilled from a control block cast without a joint and tested at the same ages as the grouted cores.

The results of these tests indicated that if the joint between the two blocks of concrete was completely filled, the strength will approach the strength of concrete. While most of the specimens failed along the grouted joint in testing, two cylinders which were tested at the age of 123 days failed in both the joint and the concrete. Figures 9 and 10 show cores before and after testing.

A later series of tests was made using cores drilled from blocks which were made with plane joints, keyed joints, and no joints. The joint width was set at 0.05-inch as before and grouted using pressures of 10, 30, and 50 psi. When these cores were tested, the grouted joints showed surprising strengths. Based on comparison with control cores drilled from blocks without joints, the efficiency of the grouted joints ranged from 81 to 125 percent. Typical results of the tests in this series are shown in Figures 11 and 12.

Subsequent to these studies, some tests were made of cores drilled across the contraction joints at Marshall Ford Dam. Although Marshall Ford Dam is a straight gravity dam, the same methods were used in grouting the contraction joints as for arch dams. After this dam had been completed and placed in service, a few drainage holes, BX-size, were drilled along the longitudinal contraction joint through the mass concrete. These holes were core-drilled and a few of the cores were found to contain excellent examples of grout film. At the time of drilling, no thought was given to making any tests of the grout film, and the cores were discarded by dumping them in a hole eroded by the side of one of the service roads. The cores were exposed to the weather for 2 or 3 years before they were noticed. The cores containing grout films were salvaged along with some check specimens for control and tested. These cores are shown in Figures 13 and 14.

Most of these cores were too short for standard test specimens. Three, however, were prepared for compressive strength tests by sawing into test lengths. Cores having grouted joints were cut so that the diagonal grout seam was entirely within the test length and the ends extended about 2 inches beyond the points where the grout seam intersected the sides of the specimen. This resulted in an average length of 6-1/2 inches for these specimens, thus producing a height-diameter ratio of approximately 3.8. Control cores without grout joints were sawed to the same dimension. The three cores with joints failed along the joint under an average compressive strength of 5,226 psi. Four control specimens failed under an average compressive strength of 6,140 psi.

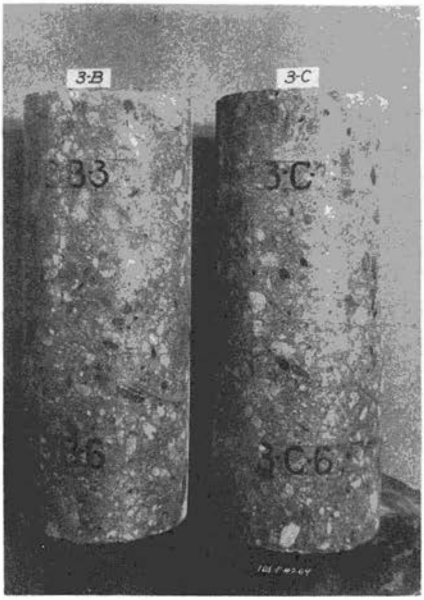


Fig. 9 Grouted Cores From Block No. 3 Before Testing

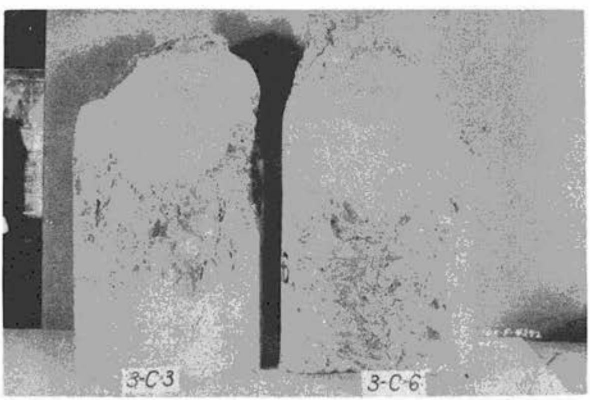


Fig. 10 Grouted Cores From Block No. 3 After Testing

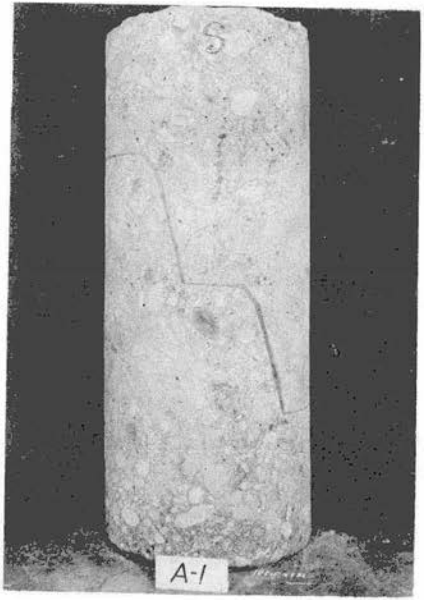


Fig. 11 Core A-1 with Keyed Joint Showing Grout Layer Before Testing. Grouted at 50 psi.

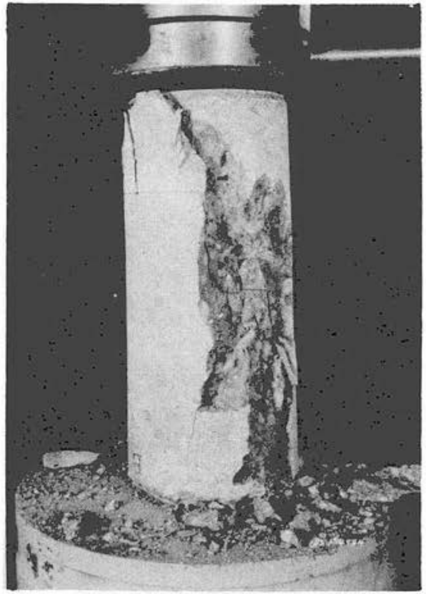


Fig. 12 Core A-1 After Testing Showing Typical Fracture Where Specimen Failed Outside of Joint. Strength 4990.

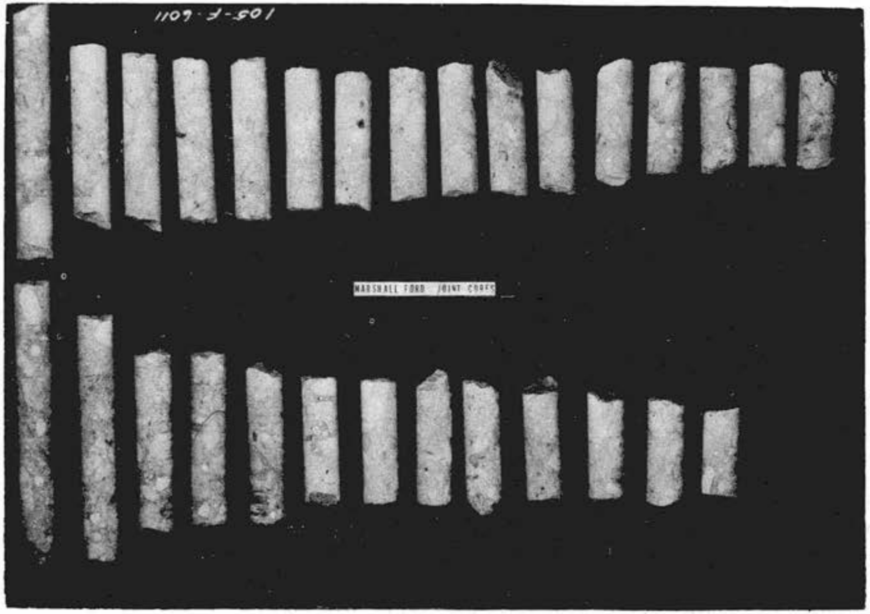


Fig. 13 General View of BX-size Cores Received From Marshall Ford Dam

Fig. 14 BX-size Cores From Marshall Ford Dam
Showing Grout Films

CONCLUSION

The effectiveness of grouting contraction joints of many arch dams has been investigated by drilling cores from across grouted joints. Many cores have been obtained which have excellent films of grout well-bonded to the adjacent concrete. These films have ranged in thickness from 1/50 inch to 3/8 inch. As the result of these investigations, it can be concluded that the system of constructing arch dams with contraction joints which are grouted with neat cement grout at the time the concrete of the dam is at its minimum volume, as now presently used by the Bureau of Reclamation, is an effective means of improving arch action in dams, minimizing cracking and reducing the possibility of leakage through the joints.

Discussion of
ARCH DAMS: THEORY, METHOD, AND DETAILS OF JOINT GROUTING

by A. Warren Simonds
(Proc. Paper 991)

R. E. GLOVER,[1] M. ASCE.—The Author describes grouting procedures used for establishing structural continuity in Arch Dams after the setting heat has been dissipated. It is important that the concrete in the dam should be brought to thermal stability with its surroundings before the grouting is done because a continued shrinkage, due to cooling after grouting has been done, will set up strains which impair the ability of the dam to carry the imposed loads in the most effective manner. In massive arch dams an embedded pipe cooling system may be needed to bring the concrete temperatures to a predetermined level, before grouting is done, because the time required to cool by natural means may be intolerably long. The times required for geometrically similar solids to lose a specified proportion of excess temperature to their surroundings, under similar conditions, is proportional to the squares of similar dimensions.[2] The significance of this rule may be illustrated by comparing the Gibson and Hoover dams. In a dam of the thickness of Gibson about 90 percent of its excess temperature, due to release of setting heat by the cement, will be lost to its surroundings in a year and a half. In the Hoover dam, which is about ten times as thick, a century and a half would be needed to lose the same proportion. This rule explains why some dams need to be artificially cooled and others do not and it also explains why embedded pipe systems are needed in large arch dams.

It is often stated that concrete cooling is used as a crack control measure and in a sense this is true, since a crack is the result of excessive tensile stress. The use of artificial refrigeration to obtain low placing temperatures has been effective as a means of reducing cracks but this type of cooling can not take the place of embeddedpipe cooling in massive arch dams. This is because cooling in arch dams is primarily a <u>stress control</u> measure and the embedded pipe system is the only one which gives the builder a positive control of the concrete temperatures. This system of temperature control has been used successfully on many massive dams. Curves of cooling, as calculated for use at Hoover Dam, permit such systems to be planned effectively in advance of construction. Comparisons of observed and computed performance of embedded pipe cooling systems show that their performance can be calculated with a precision that exceeds the needs for effective planning and use.

All dams are subject to temperature changes due to seasonal changes of

1. Research Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.
2. "Flow of Heat in Dams," by Robert E. Glover. Journal of the American Concrete Institute. Proceedings Vol. 31, Nov-Dec 1934, Page 113.

the external temperatures. In dams thicker than about 35 feet the lag of the mean concrete temperatures behind the seasonal temperatures is one eighth of a period of fluctuation. For the seasonal changes this lag is about a month and a half. If the winter temperatures come to their lowest point at about the middle of January then the lowest mean concrete temperatures may be expected about the first of March. For thinner dams the lag is less.

Discussion of
ARCH DAMS: THEORY, METHODS, AND DETAILS OF JOINT GROUTING

by A. Warren Simonds
(Proc. Paper 991)

JUDSON P. ELSTON,[1] M. ASCE.—I think Mr. Simonds has presented an excellent general coverage whereby in an arch dam it becomes possible to develop arch action prior to any appreciable raising of the reservoir by creating an "arched" monolith. I wonder if in a sense of the word, successful grouting of the joints in an arch dam does not create a "prestressed" arch. Is not the dam made monolithic and does not the equal and even opening of each joint and filling with a dense cement mixture result in a ring in which the blocks are theoretically in tension? Then as the reservoir is filled and load is applied to the ring, would not a slight deflection downstream take place and the end loading be compressive?

The background and history of joint grouting in connection with arch dams is very interesting and informative. Mr. Simonds states on page 991-5 that, "The joint keys developed by the Bureau of Reclamation have now been standarized and are used in the construction of all solid concrete dams." I presume he means only arch dams and not necessarily all gravity dams. I think that within the last five years, considerable study has led some designers to the conclusion that, dependent on the results of studies of topography and shape of the dam site foundation as well as the foundation material itself along with a trial load analysis of the structure, keyways and/or joint grouting may not be necessary or required on gravity type dams. Perhaps Mr. Simonds could enlighten me a little on this subject.

On page 991-7 under the section on "Theory of Grouting Contraction Joints," Mr. Simonds states that high tensile cantilever stresses may develop on the downstream side of the structure, and; "These can be controlled to a large extent by a partial filling of the reservoir to balance the resultant of the grout pressure."

I agree that a partial filling of the reservoir will tend to balance the resultant of pressures but feel that such a procedure is not only difficult to achieve in practice but can become dangerous to the success of the entire grouting operation. Actually, in theory to correctly counteract such pressures, the filling of the reservoir should be taking place at the same time that grouting pressures are being applied to the joints, should it not? This is manifestly almost an impossibility at times during the construction stage for one reason or another. From the practical point of view, might it not be just as satisfactory to cut down on the height of lifts to be grouted? Unless it is possible to have sufficient and exact load acting on the upstream face of the structure to restrain any upstream movement as shown on Figure 3, the joints will be in compression at the downstream face and in tension at the upstream face during grouting. As Mr. Simonds states the adjacent joints

1. Foundation Engr., Uhl, Hall & Rich, Engrs. for the Power Authority of the State of New York on the St. Lawrence River Power Project, Massena, N. Y.

should be full of water under pressure if necessary. Also the enclosing metal grout stops should be correctly designed and carefully embedded in concrete so as not to rupture and leak under pressure. A theodolite should be set up for measurements on the crown cantilever and dial gauge installations made on the up and downstream faces. By cutting the grouting lift heights from 50 feet down to 30 to 35 feet and with extreme care and precaution (close control) during the grouting operation, I would think it possible to secure the required opening distributed horizontally and vertically without raising the reservoir and without developing undue stresses. I do not take exception to Mr. Simonds' statements! I merely wish to point out that in some cases it has not been possible or necessary to raise the reservoir and that, in fact, raising of the reservoir prior to grouting in itself does not insure that unbalanced stresses or strains will not develop.

Discussion of
ARCH DAMS: THEORY, METHODS, AND DETAILS OF JOINT GROUTING

by A. Warren Simonds
(Proc. Paper 991)

A. WARREN SIMONDS,[1] M. ASCE.—The discussion by Mr. Glover contains some interesting observations relating to the thermal conditions of arch dams in regard to thickness of the section. While most arch dams constructed prior to 1930 were allowed to cool naturally, it was always difficult to find a time when the optimum temperature prevailed throughout the structure. The minimum temperature of the mass concrete in a dam occurs in the late winter or early spring season. In the upper part of the structure, where the section is thin, the concrete will reach its minimum temperature while the concrete is still cooling in the thicker part at the lower elevations. It was therefore necessary to select a time for grouting the joints when a more or less "average" temperature prevailed. By installing an embedded system of coils throughout the concrete and cooling the concrete artificially, not only is the setting heat of the concrete removed at an early date without regard to the seasonal variation in temperature, but also better control of the temperature distribution is obtained.

Mr. Elston mentions the possibility of creating an "arched" monolith by grouting the contraction joints. A concrete arch dam should be monolithic for the most desirable distribution of stress. The effect of the grout pressure in the contraction joints may be considered as "prestressing" because the distribution of stress is changed by the grout pressure in the joints. This phenomenon has been indicated by observations on strain meters embedded in the mass concrete and also by precise survey measurements which have shown that the position of structure has been moved as a result of the grout pressure.

Grout pressure in the contraction joints, if properly balanced will place the blocks in the arch ring in compression. The only tension that will develop, as a result of grout pressure, will be in the cantilever elements. When the load comes on the structure, the deflection and the stress distribution will depend on the geometry of the structure, that is the thickness of the arch, radius of curvature, central angle, and also to possible changes in temperature. In a properly designed arch dam, the stresses should be compressive, but if the topography of the site and the configuration of the dam are such that some tension should be inevitable, the tension should be held to a minimum. In most arch dams a downstream deflection will cause a compressive stress to predominate at the abutments. This may be maximum at the downstream face and diminish toward the upstream face or even becoming tensile at the upstream face. For long slender arches, downstream deflection may occur at the crown and an upstream counter flexure at the quarter points nearest the abutments. When this occurs, the stress distribution at the ends of the arch is the reverse from that previously described.*

1. Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.
* Report on Arch Dam Investigation, Engineering Foundation, Vol. I, 1928. Model Tests, Analytical Computation and Observation of an Arch Dam ASCE Separate 696.

The general practice of the Bureau of Reclamation is to use keys in the joints of all solid concrete dams, of both arch and gravity types, regardless of whether the joints are to be grouted or not. In the case of straight gravity dams, where each block or monolith is statically stable and will carry its load independently of the other blocks of the dam, keys are not essential if the joints are not to be grouted.

In regard to the partial filling of the reservoir to balance the resultant of grout pressures in joints, such a condition involves some hazard if the stream is subject to flash floods, in which case, the reservoir water load would be carried by an ungrouted arch. It is not necessary to raise the reservoir water surface to exactly the same level at the same time fluid grout is being injected in the joints. The elevation of the reservoir water surface should be determined within limits which will prevent excessive tensile cantilever stresses from developing in the blocks of the dam. Upstream deflections of small magnitude do not appear to be harmful if they are not accompanied by bending stresses which are likely to cause tensile cracking at the downstream face, or splitting the grout film in the previously grouted lower lifts of the joints.

For ideal conditions, grouting lifts of lesser height than 50 feet, as suggested by Mr. Elston, are desirable from a stress standpoint. From a practical standpoint, the higher grouting lifts which can be used satisfactorily will reduce the total number of lifts in the structure to be grouted and will, therefore, make the grouting operation less costly. Grouting lifts have been used having heights ranging from 25 feet in a few cases to 125 feet; the height of 50 feet has been selected as being practical and economical for most arch dams.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: SANTA GIUSTINA SINGLE-CURVATURE ARCH DAM

Claudio Marcello,[1] M. ASCE
(Proc. Paper 992)

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams held in April, 1939, much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be included in the Symposium. So far, nine papers have been approved: "Arch Dams: Their Philosophy" (Proc. Paper 959) by Andre Coyne; "Arch Dams: Trial Load Studies for Hungry Horse Dam" (Proc. Paper 960) by R. E. Glover and Merlin D. Copen; "Arch Dams: Portuguese Experience with Overflow Arch Dams" (Proc. Paper 990) by A. C. Xerez; "Arch Dams: Theory, Methods, and Details of Joint Grouting" (Proc. Paper 991) by A. Warren Simonds: "Arch Dams: Santa Giustina Single-Curvature Arch Dam" (Proc. Paper 992) by Claudio Marcello; "Arch Dams: Measurements and Studies on Santa Gustina Dam" (Proc. Paper 993) by Claudio Marcello; "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam" (Proc. Paper 994) by Claudio Marcello; "Arch Dams: Isolato Double-Curvature Arch Dam (Proc. Paper 995) by Claudio Marcello; and "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs" (Proc. Paper 996) by Claudio Marcello.

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

SYNOPSIS

This paper outlines the design and construction of an Italian arch dam 152 meters high and located in an extremely narrow gorge. Briefly discussed are

Note: Discussion open until November 1, 1956. Paper 992 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 3, June, 1956.

1. Cons. Engr. and Technical Director, Societa Edison, Milan, Italy.

the structural analysis of the dam—confirmed by model, construction plant, and the extensive foundation consolidation and cut-off grouting.

The dam was built during the years 1946-1950 to provide a large seasonal reservoir with storage capacity of 183 million cubic metres in the Valley of Noce River (Trento) (Fig. 1).

The reservoir is intended for annual regulation of the Taio power station, which is directly fed from the reservoir, as well as for regulation of the plant of Mezzocorona, situated farther downstream, both plants being included in the Noce hydroelectric scheme and owned by Soc. Edison.

The first plan of the structure is to be attributed to the late Ing. Angelo Omodeo.

The final design was completed in 1943, although the construction work cound be actually started only in 1946. The time required for construction was 4 years, because of some difficulties connected with post-war conditions.

The dam has a maximum height of 152.50 metres above foundations, a length at crest of 124.20 metres, and a thickness at the crown varying from 3.50 m at the top to 16.50 m at the bottom. Maximum storage level is at El. 530 metres above sea level.

The structure is a single-curvature arch dam, in accordance with the shape of the gorge which has practically vertical walls: the arches are symmetrical, with thickness increasing from crown to abutments (Fig. 2).

The extrados of the elemental arches is circular with single center, the intrados is also circular, but with five centers and three radii, one of which corresponds to the middle center and the other two correspond to the two symmetrical pairs of center on either side (Fig. 3).

The dam is abutted directly on the rock up to the height of 102.50 m above foundations, on a lightly reinforced shoulder from the height of 102.50 to 142.50 m, and on solid gravity thrust blocks from this height to the crest.

Angle amplitudes are variable from $77^{o}50'$ at the bottom to 108^{o} at the top, the mean curvature radius increasing from 25.50 to 44.50 m.

Both dam facings are of bare concrete, and steel forms were used. No special plaster was applied for the lower portion of the upstream face, where a mesh-rendering is provided up to El. 446.

A light reinforcement was provided behind the facings for stress spreading purposes.

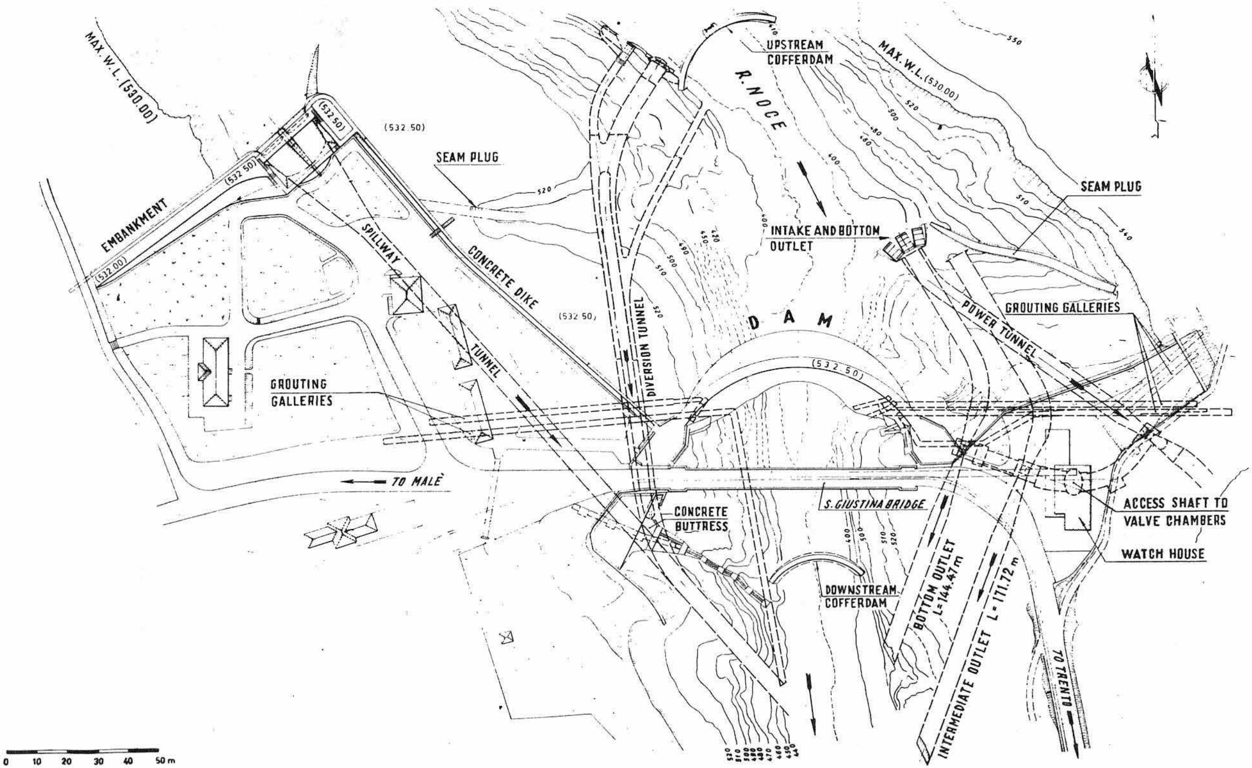
During construction the dam was divided into 7 blocks by 6 vertical temporary joints spaced from 12.50 to 15.50 m (measured on the upstream facing). The four joints in the central part divide the whole height of the dam, while the two lateral joints were provided only for the portion above El. 460 (Fig. 4).

After shrinkage was almost completed the joints were filled with concrete and subsequently cement grouted. Grouting was made by sections 15 m high, through a system of pipes embedded into the concrete, under low-temperature conditions (Fig. 6).

Grout leakage was avoided by the use of copper strips embedded by one half into the concrete block and one half into the joint fill.

No special drainage system was provided inside the structure, whereas the foundations are drained by means of porous concrete pipes. These lead to a vertical shaft driven to the deepest point of a longitudinal scar in the bedrock

GENERAL LAYOUT

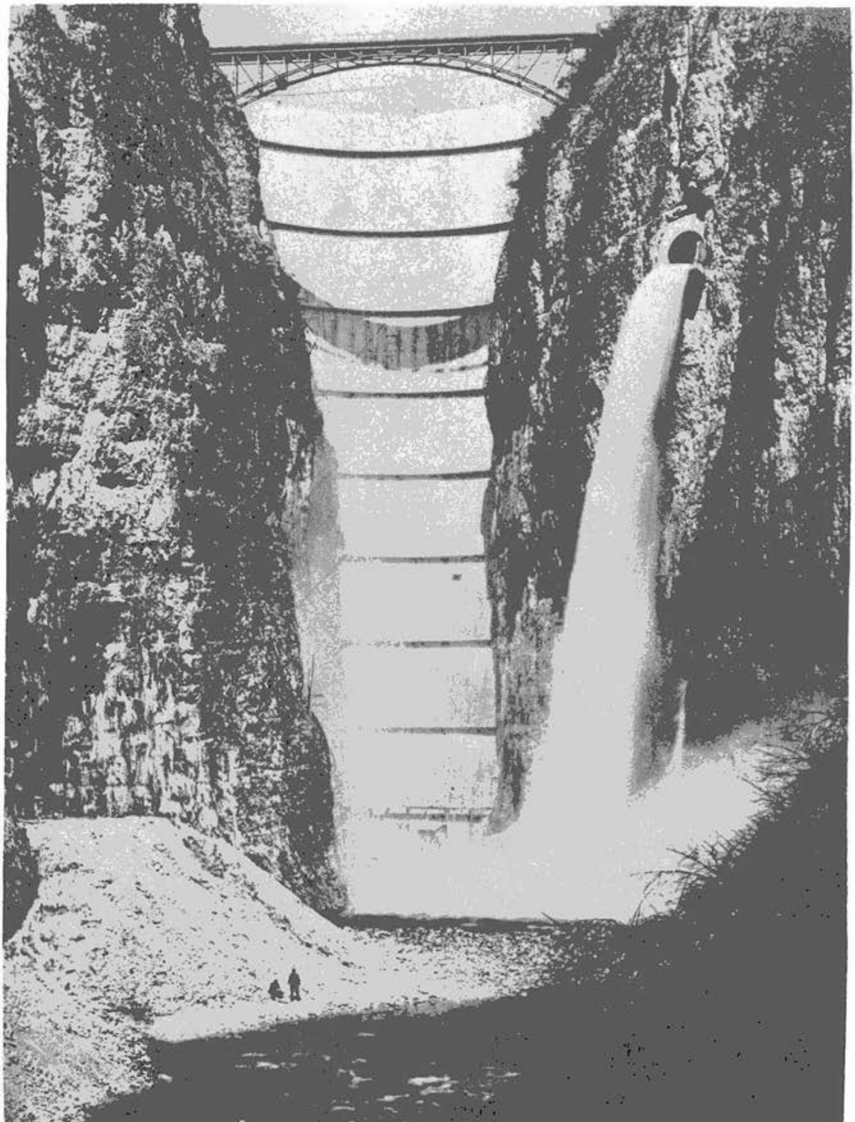


MARCELLO

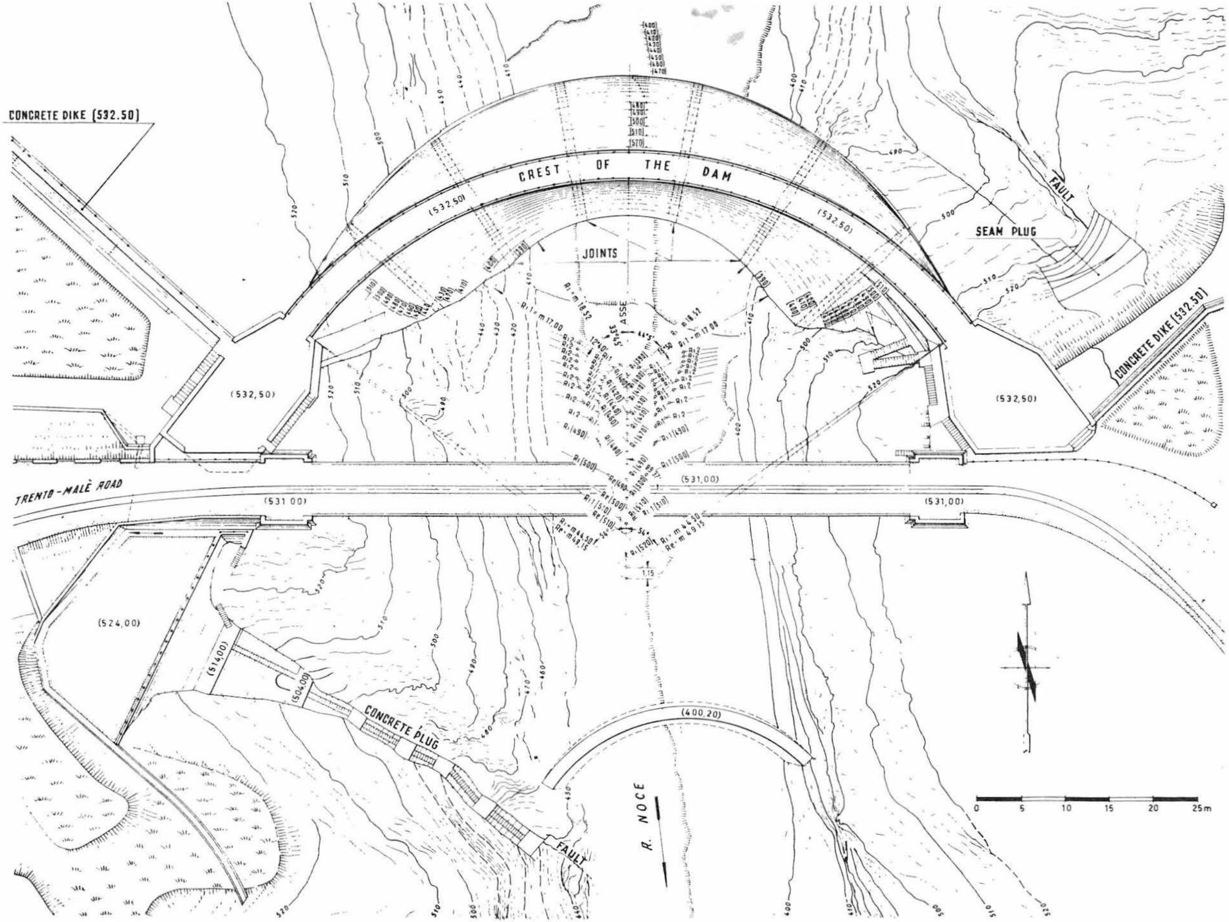
FIG. 1

SANTA GIUSTINA DAM

FIG.2



View of the downstream face from the bottom of the gorge. The dam has a maximum height of 152.50 m, a crest length of 124.20 m and a thickness at crown variable from 3.50 m at the crest to 16.50 at the bottom. See on the right the intermediate outlet under operation.

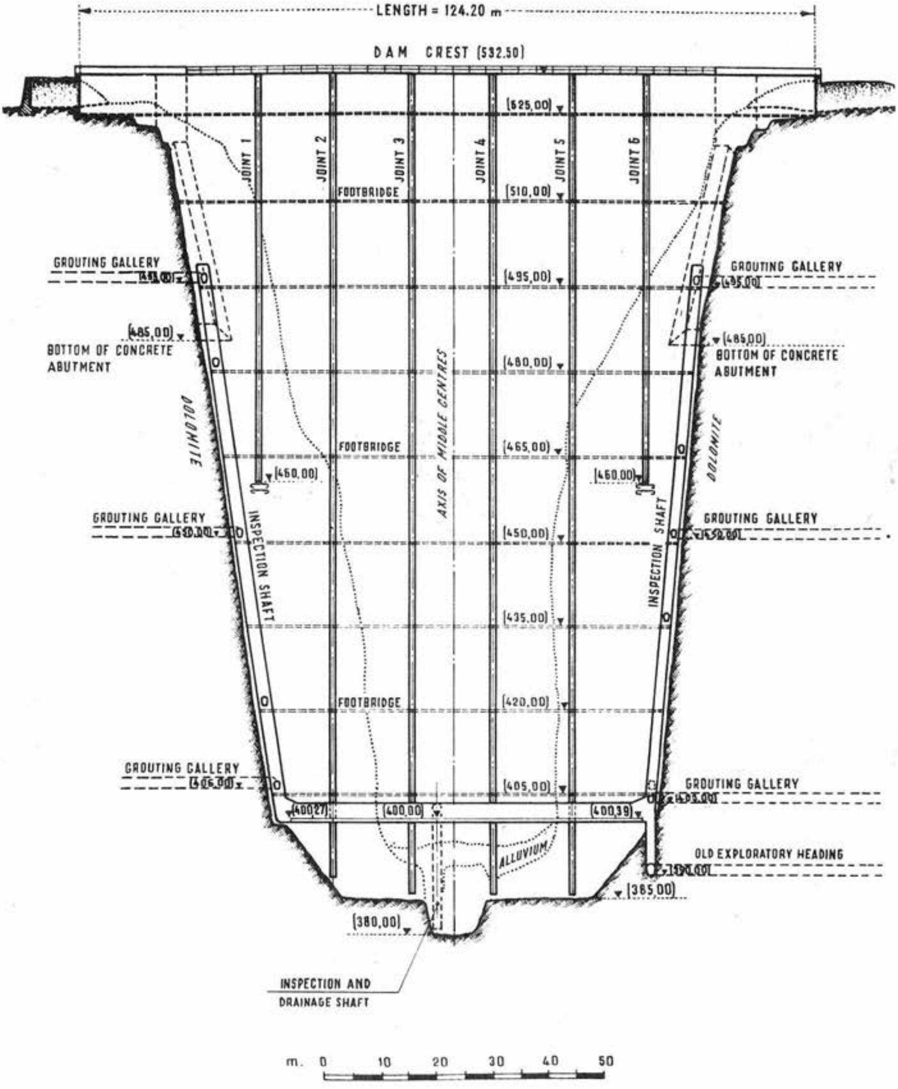


PLAN OF THE DAM

FIG. 3

FIG. 4

SECTIONAL ELEVATION
DOWNSTREAM VIEW



and connected at its upper end with a horizontal inspection gallery about 20 m above the foundation plane. Access to this gallery is provided through two sloping adits running along the abutments.

Nine horizontal inspection footbridges spaced 15 m are arranged along the downstream face (Fig. 5).

Statical Analysis

For purposes of design, the dam was considered as consisting of a series of horizontal independent arches keyed at the abutments.

The arches below El. 460, for which the ratio radius/thickness is smaller than or equal to 3, have been analized by the simplified method of "rigid arches;" the modulus of elasticity of both rock and concrete was assumed as equal to and the pressure transmitted to the rock was assumed as uniform over the whole thickness of the arch.

The arches above El. 460 have been considered as elastic, and subject to hydrostatic pressure along the extrados, to temperature changes and to shrinkage.

The statical analysis of these arches has been performed by the method of elasticity ellipse with the following assumptions:

- Specific gravity of water	1.00 ton/m^3
- Shrinkage made equal to a temperature decrease of	3°C
- Maximum temperature difference between intrados and extrados	3°C
- Maximum yearly temperature variation	20°C

The maximum compression stresses with full reservoir amounted to 43.7 Kg/cm^2 and the tensile stresses to 3.9 Kg/cm^2, in the assumption of sole action of hydrostatic pressure at its maximum value (corresponding to maximum water level).

Under simultaneous action of maximum hydrostatic pressure, expected temperature fluctuations, and shrinkage, the maximum principal stresses reach 46.6 Kg/cm^2 for compression and 7.8 Kg/cm^2 for tension.

With empty reservoir the maximum principal stresses due to temperature fluctuations and shrinkage were found to reach 9.1 Kg/cm^2 for compression and 9.7 Kg/cm^2 for tension.

Model Testing

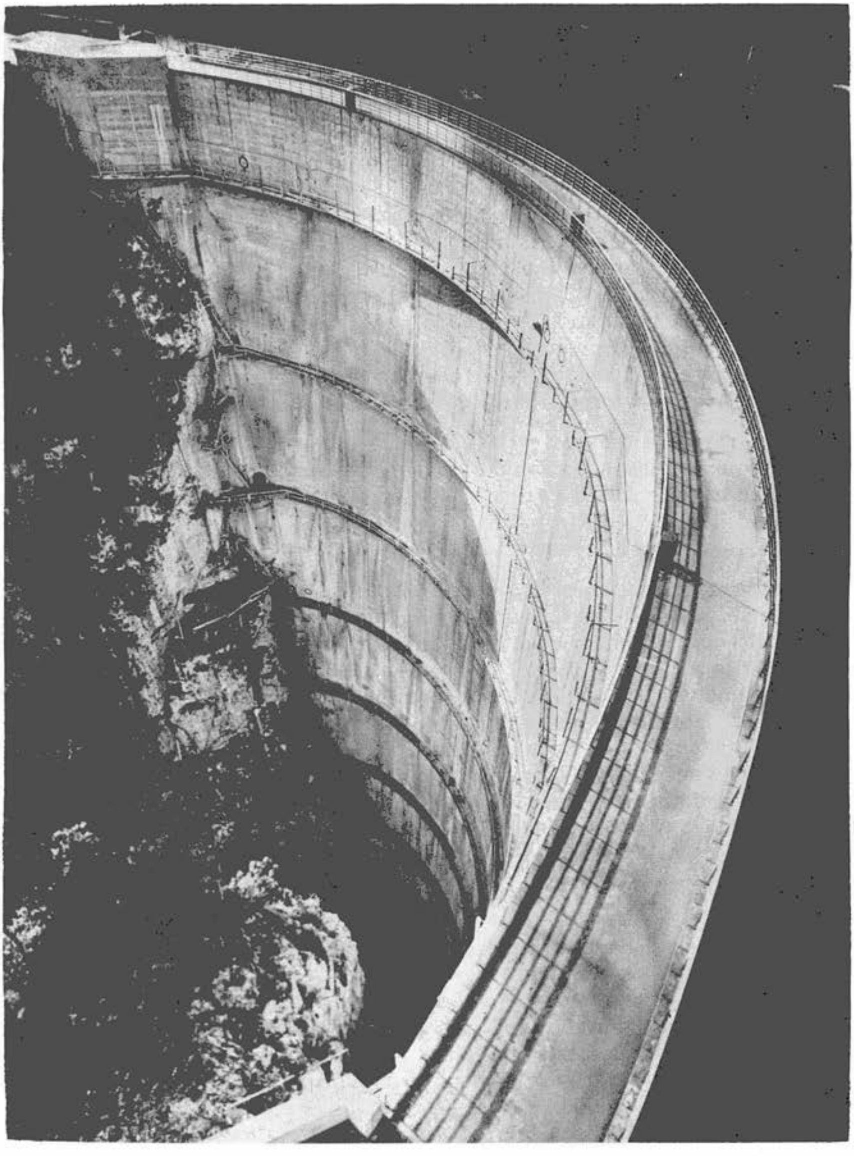
All calculations have been checked by means of model tests carried out at the Politechnic School of Milan.

A first series of bi-dimensional tests were performed on models of isolated arches to ascertain the statical behaviour of the thicker parts, for which the theoretical analysis is always rather uncertain. For the same purpose, tri-dimensional tests were carried out on a chalk and celite model of the part of the dam below El. 440.00.

Load tests have also been carried out on a 1:60 scale chalk and celite model of the whole dam, taking into consideration only the hydrostatic pressure at maximum water level; also ultimate strength tests were made.

According to these tests, no tension has been found in the horizontal planes (arches) and negligible ones in the vertical planes (cantilevers):

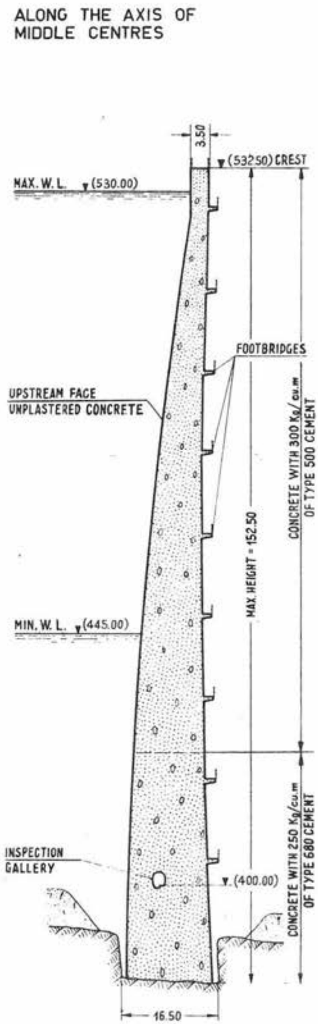
SANTA GIUSTINA DAM FIG.5

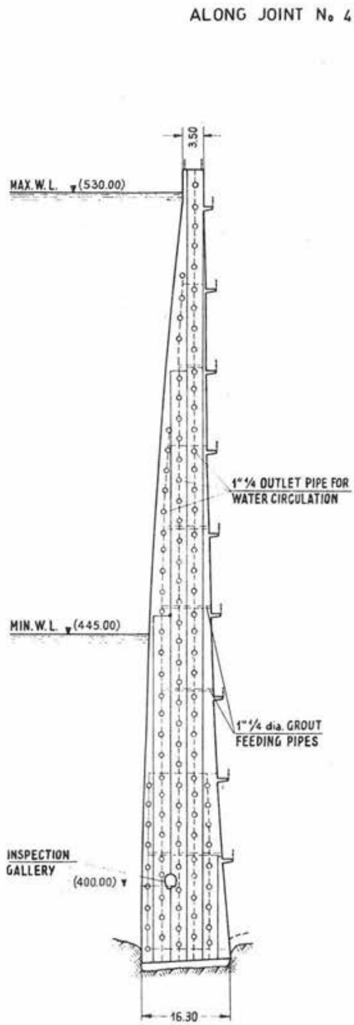


View from the left bank; see on the downstream face the horizontal inspection footbridges spaced 15 m.

FIG. 6

CROSS SECTIONS





compression is generally less than calculated, especially for the lower arches, except for a few regions where compressive stresses were found to be up to 10% higher than calculated.

Construction

Construction work was actually started in 1946; up to that time the only work performed concerned excavation, and preparation of quarrying, crushing and screening installations for aggregates.

Aggregates for the dam were obtained from a quarry of dolomite rock opened on the left bank, about 200 m upstream from the dam.

The rock was transported by cableway from the quarry to the crushing plant on the right bank, where it was crushed and sorted in five classes, namely: very fine, below 0.1 mm; from 0.1 to 4 mm; from 4 to 12 mm; from 12 to 30 mm; and from 30 to 70 mm.

For the dam portion below El. 425.00, concrete contained 250 Kg per m^3 of cement having a standard mortar average strength of 680 Kg/cm^2 after 28 days. Above El. 425.00 concrete contained 300 Kg/m^3 of cement having a standard mortar strength of 500 Kg/cm^2.

Aggregate grading was, in weight:

- with 680 Kg/cm^2 cement

very fine elements, from zero to 0.1 mm	12%
elements from 0.1 to 4 mm	22%
" from 4 to 12 mm	15%
" from 12 to 30 mm	21%
" from 30 to 70 mm	30%
	100%

- with 500 Kg/cm^2 cement:

very fine elements, from zero to 0.1 mm	12%
elements from 0.1 to 4 mm	22%
" from 4 to 12 mm	15%
" from 12 to 30 mm	25%
" from 30 to 70 mm	26%
	100%

From the mixing plant, located near the crushing and screening plant, concrete was distributed by means of two Blondins and for the lower portion also by means of a proper chute and derrick.

Concrete was poured in steel forms by 50 cm lifts and compacted by means of electric vibrators (Figs. 8 and 9).

The progress of pours was as follows:

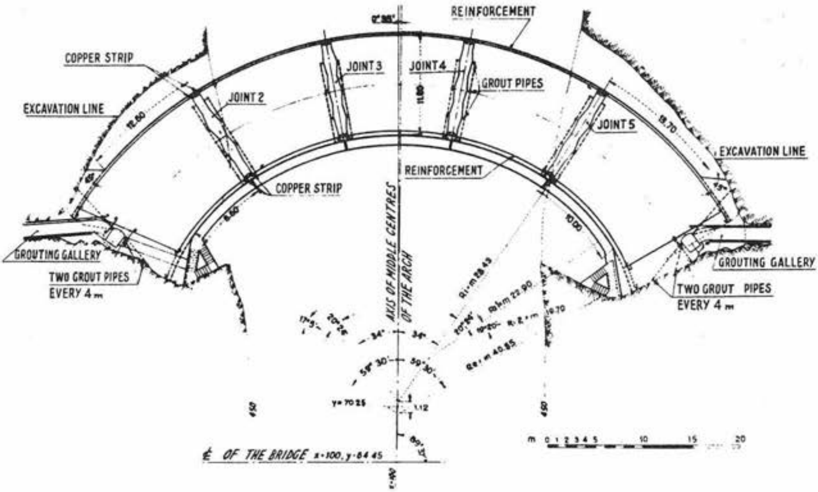
1946 -	6,400 cu.m	- up to elevation	396
1947 -	24,000 "	- " " "	422
1948 -	46,200 "	- " " "	479
1949 -	28,400 "	- " " "	524
1950 -	7,000 "	- " " "	532.50
Total	112,000		

When resuming the work after winter suspensions, the whole surface of the blocks as well as the bottom of points were prepared for the new pours by adequate cleaning and chipping to an average depth of 5 cm.

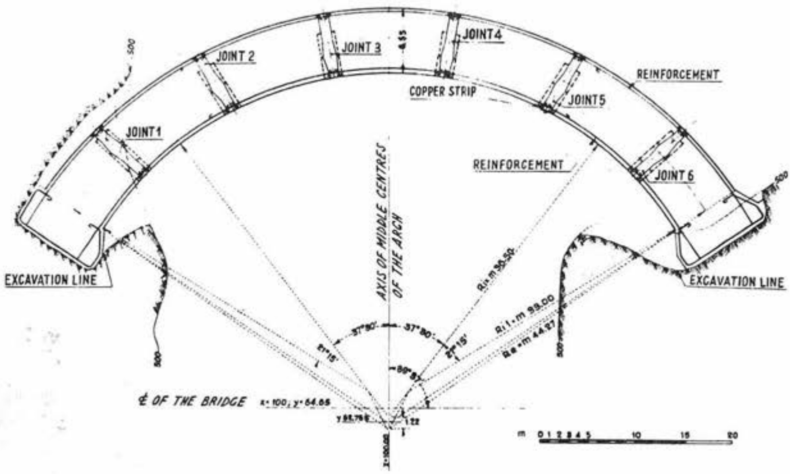
The filling of the joints left open during the preceding working season was

FIG. 7

HORIZONTAL SECTION AT EL. 450.00

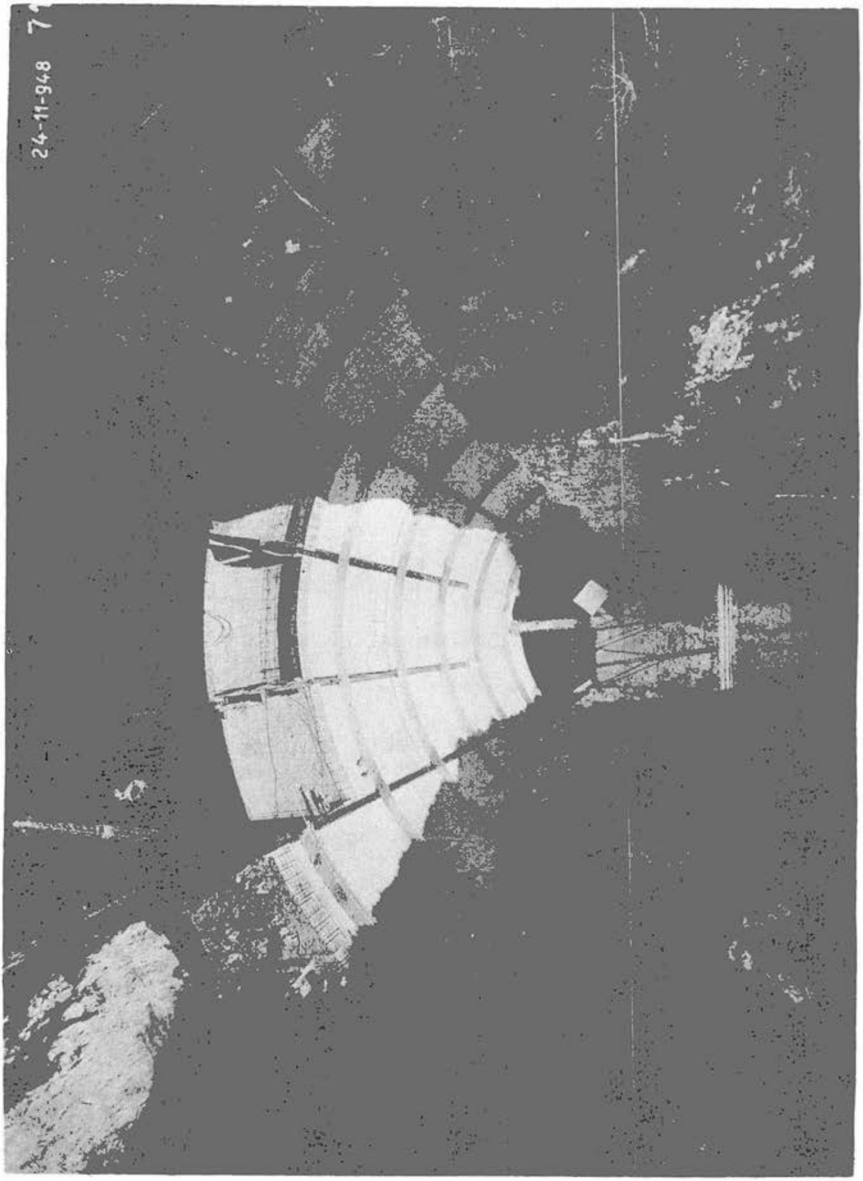


HORIZONTAL SECTION AT EL. 500.00



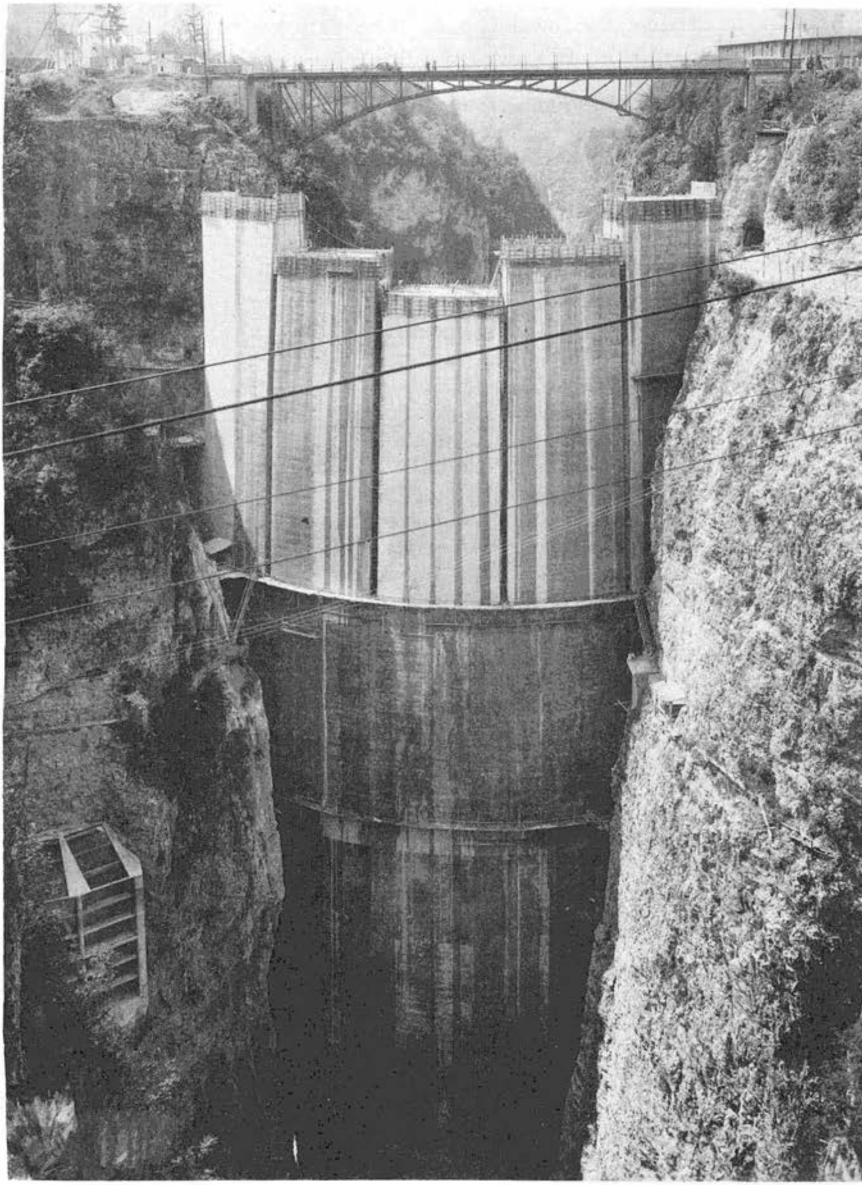
SANTA GIUSTINA DAM

FIG.8



View of the dam under construction. The concrete pours were made by blocks divided by provisional construction joints to be filled with concrete and then to be grouted after shrinkage.

SANTA GIUSTINA DAM FIG.9



View of the upstream face when the construction was practically ultimated. The provisional joints spaced 12.50 ÷ 15.50 m are already closed and sealed in the lower part of the dam. See at the top the steel forms for the pours.

performed at the beginning of the subsequent one and concrete was used which had the same composition as the concrete of the adjacent blocks, so far as aggregate grading, and water and cement proportioning were concerned.

Joints were subsequently grouted along the opposite joint faces (joint to fill); at the same time, the lower portion of the dam upstream face (below El. 446.00) was coated by hand-applied mesh-rendering after chipping of the concrete.

Laboratory tests showed an average compressive strength of 420 Kg/cm^2 after 28 days for the concrete mixed with 250 Kg/m^3 of 680 Kg/cm^2 cement and 380 Kg/cm^2 for the concrete mixed with 300 Kg/m^3 of 500 Kg/cm^2 cement.

Average strength after 1 year was for both types higher than 470 Kg/cm^2 with peaks of 700 Kg/cm^2.

The concrete was found to be practically impervious.

Waterproofing

Waterproofing work had a considerable importance because the dolomite rock on which the dam is founded showed horizontal stratifications with faults and a number of small jointing (Fig. 10).

A deep grout curtain had therefore to be provided, with holes driven down to 50-60 m below foundation plane.

Three tunnels were driven into the gorge walls at Els. 406 - 450 and 495, reaching about 100 m into the rock. Vertical grout holes were drilled downwards from these tunnels, spaced 2.50 m centers to a depth of over 50 m, so as to cover the whole underlying rock section down to the next tunnel.

Sloping grout holes were also drilled from the abutments blocks, from the above said grouting tunnels and from the outlet tunnels, in order to obtain a good connection between the vertical groutings. The holes, driven by rotary drill, were grouted by sections at a pressure of 80 atmospheres by means of compressed air driven pumps and hydraulic leakage tests were made at pressures of 10 to 12 atm.

The water-cement ratio of the mix used for grouting varied between 8 to 1 and 6 to 1. A total of over 65,000 m of holes were drilled: average absorption was little more than 100 Kg. of cement per metre of bore.

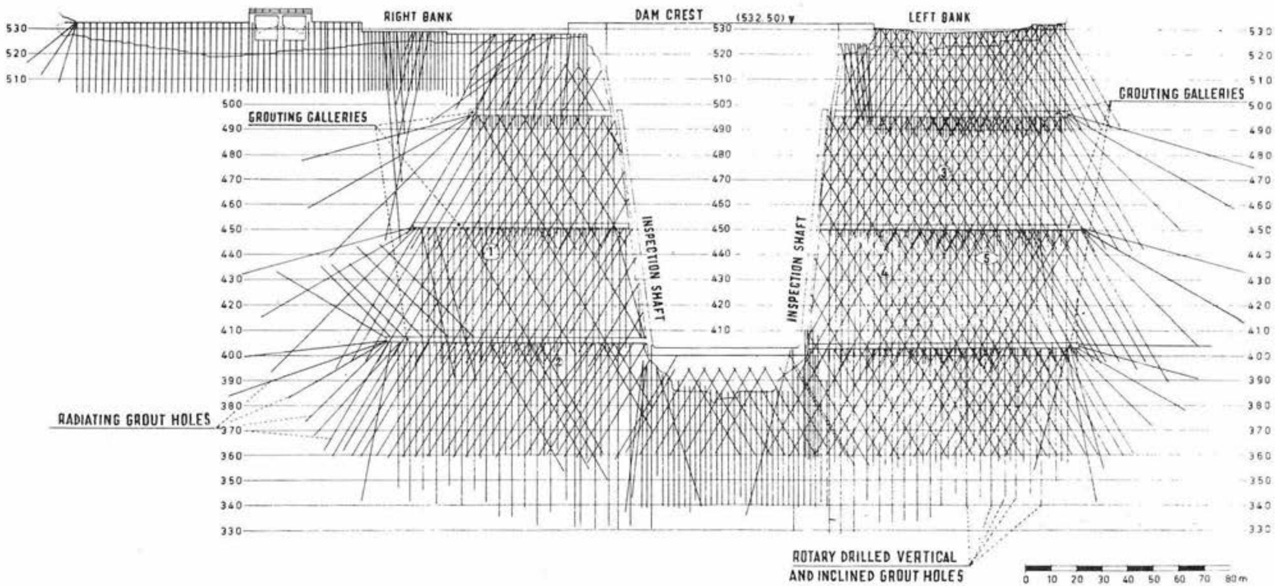
The faults and seams which endangered watertightness were filled with concrete plugs. A seam crossing the gorge upstream of the dam and extending on both banks was plugged with concrete and grouted after accurate cleaning and washing.

Many precautions had to be adopted for the right bank, where a fault could have affected the stability of the structure; this fault was subjected to a thorough washing by means of high-pressure jets (up to 30 atm) driven through a series of holes drilled so as to cross it, and was subsequently grouted. Some small cavities in the rock were filled with concrete and grouted.

Consolidation grouting at a pressure of over 70 atms. was carried out at the abutments, through 424 holes, and so distributed as to cover a mass of

GROUT CURTAIN

DOWNSTREAM VIEW

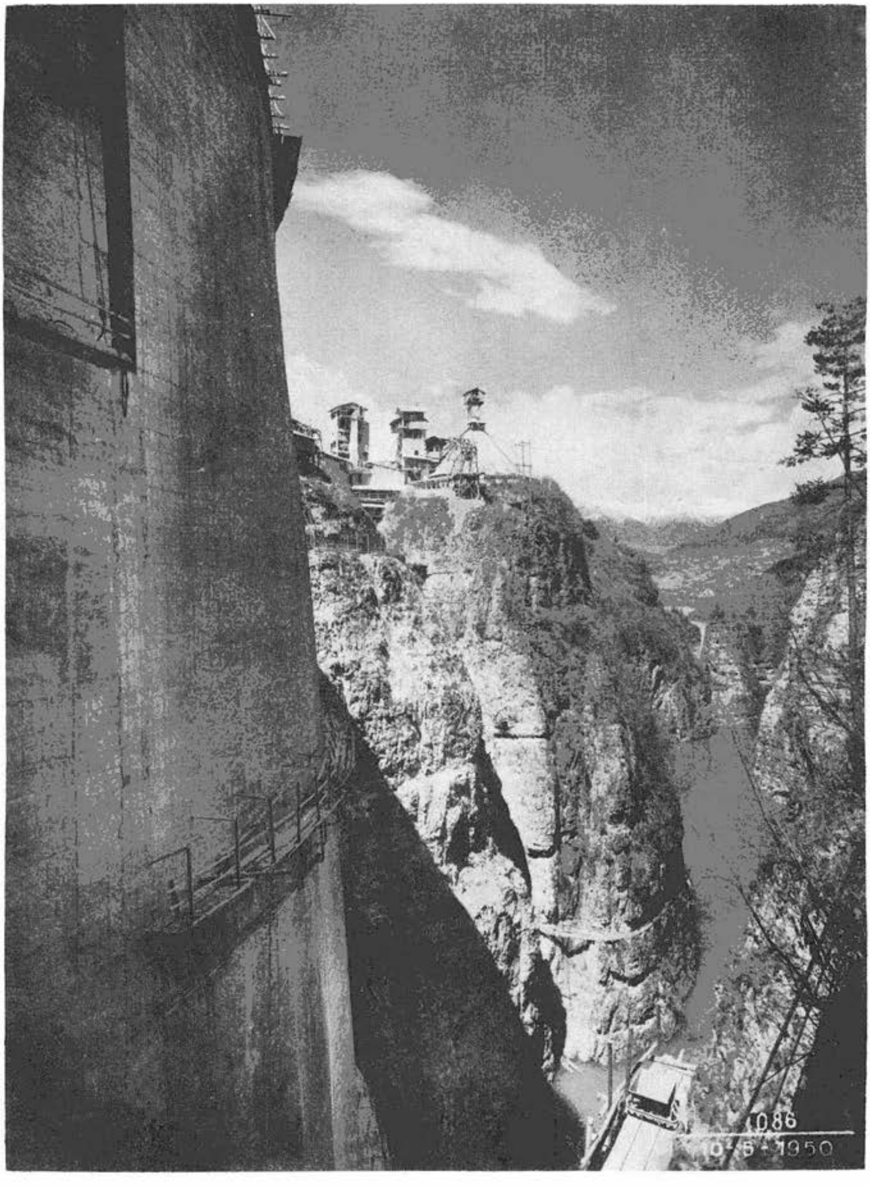


1 SPILLWAY TUNNEL
2 DIVERSION TUNNEL
3 INTERMEDIATE OUTLET
4 BOTTOM OUTLET
5 POWER TUNNEL

TOTAL LENGTH OF GROUT HOLES 65,000 m
GROUTING AT MAX PRESSURE OF 80 Kg/sq. cm ;
CEMENT/WATER RATIO OF GROUT VARYING FROM 8:1 TO 6:1
INJECTED CEMENT : 0,1 METRIC TONS PER METRE OF GROUT HOLE

FIG. 10

SANTA GIUSTINA DAM FIG. 11



Detail of the upstream face. In the background the field installations.

rock sufficient to support by itself the thrust of the dam. The total length of the holes grouted is 18.424 m with absorption of 1,020. 5 tons of cement.

The dam was designed and built by the Hydroelectric Plant Construction Department of Soc. Edison of Milan, under the direction of the writer.

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POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: MEASUREMENTS AND STUDIES ON SANTA GIUSTINA DAM

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(Proc. Paper 993)

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams held in April, 1939, much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be included in the Symposium. So far, nine papers have been approved: "Arch Dams: Their Philosophy" (Proc. Paper 959) by Andre Coyne; "Arch Dams: Trial Load Studies for Hungry Horse Dam" (Proc. Paper 960) by R. E. Glover and Merlin D. Copen; "Arch Dams: Portuguese Experience with Overflow Arch Dams" (Proc. Paper 990) by A. C. Xerez; "Arch Dams: Theory, Methods, and Details of Joint Grouting" (Proc. Paper 991) by A. Warren Simonds; "Arch Dams: Santa Giustina Single-Curvature Arch Dam" (Proc. Paper 992) by Claudio Marcello; "Arch Dams: Measurements and Studies on Santa Giustina Dam" (Proc. Paper 993) by Claudio Marcello; "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam" (Proc. Paper 994) by Claudio Marcello; "Arch Dams: Isolato Double-Curvature Arch Dam" (Proc. Paper 995) by Claudio Marcello; and "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs" (Proc. Paper 996) by Claudio Marcello.

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

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SYNOPSIS

Owing to the unusual proportions of this structure, special plans were made to study its behavior resulting from variations of water level and temperature. This paper is a preliminary report and presents data obtained from four years of observation and compares these results with those obtained by analysis.

This paper presents some preliminary results of an investigation concerning Santa Giustina Dam, a dam built in Northern Italy after World War II.

The dam spans a very deep gorge with extremely steep walls, on the course of Noce River, and forms a reservoir with a storage capacity of 183,000,000 m^3. The dam is a practical symmetrical, 152.50 m high, thin-arched structure. The thickness in the crown section varies from 3.50 m at the crest (el. 532.50 m a.s.l.) to 16.50 m at the foot (el. 380 m a.s.l.); the chord at the crest is 75 m approx.; the upstream arches have curvature radii variable from 49.15 to 39.35 m and span angles varying from 106° to 74° (see Figs. 1 and 2).*

Owing to the uncommon dimensions of this structure, it was considered particularly interesting to investigate its behaviour in respect to the variations of water level and temperature by direct measurement, and to compare these results with those obtained by various methods used for the analysis of arch dams.

To obtain the required experimental data, the dam was provided with a system of thermometers and a system of extensometers. The periodic reading of these instruments (daily temperature readings and weekly strain readings) was supplemented by systematic geodetical measurements. These latter consisted mainly of the measurement, once every three months approx., of a triangulation intended to give the planimetric coordinates of a series of points of the downstream face of the dam. Geometrical levellings were also carried out, to give the elevations of a series of points of the dam as well as of the gorge walls.

This first report takes into consideration the radial deflections of points of the central cantilever, as measured in the first 20 triangulations made during as long as four and a half years, and the strains and stress increases found by extensometrical measurements carried out at the same time as the geodetical measurements.

As far as theoretical analysis is concerned, the paper gives the first results obtained by two methods intended to calculate the deflections and the stress variations in the dam in function of temperature and water level variations occurring between the first geodetical measurement and those following. These results are then compared with the data already found by experience.

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* A detailed description of the dam is the subject of another paper entitled "Santa Giustina Single-Curvature Arch Dam," illustrated in this same Symposium.

SANTA GIUSTINA DAM

FIGURE 1- CROSS SECTION OF THE DAM AT CROWN CANTILEVER — SCALE 1:1000

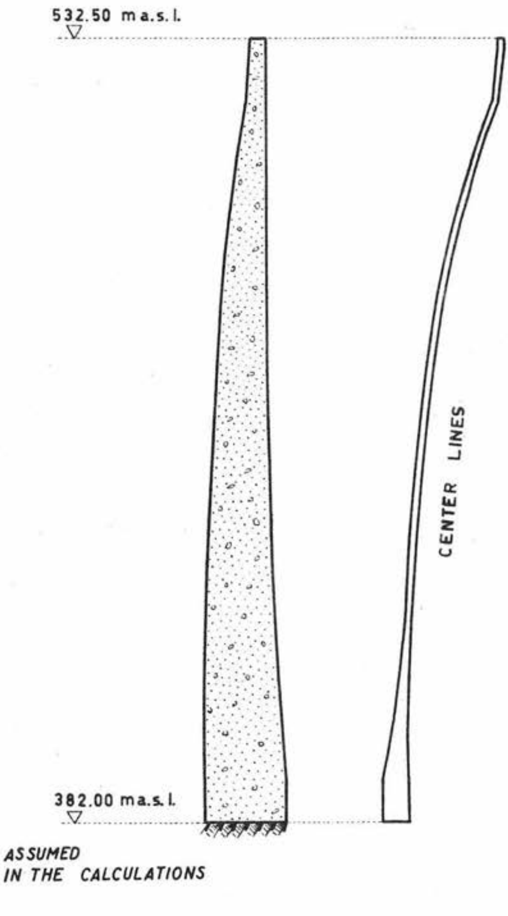
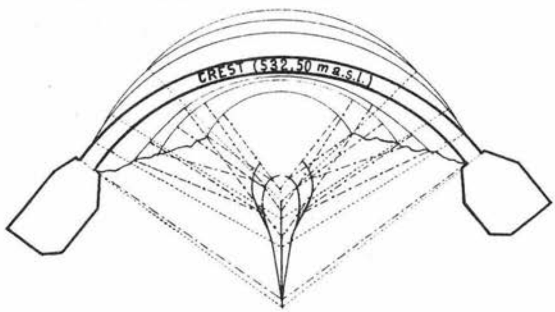


FIGURE 2- PLAN OF THE DAM - SCALE 1:1000



Geodetical Measurements—Deflections of the Central Cantilever

Out of all the results of the geodetical operations carried out to determine the movements of the Santa Giustina dam, only the deflections of the central cantilever measured during four and a half years of investigations are reported in this first paper. This period extends from early October 1950, when for the first time the reservoir water level reached elevation 435 m a.s.l. (equal to 53 m above the dam foot) and February 1955, when the setting heat had already been almost completely disposed of and the dam had undergone three practically periodic annual cycles of variation in water level and temperature.

The 20 triangulation measurements, the results of which are here reported, were carried out at the dates shown in Table 1, when the water level was as indicated in the same table.

Fig. 3 illustrates all the targets placed on the downstream face of the dam; a filled-in triangle marks the targets placed at the crown of the arches, i.e., the ones considered in the present report. The radial displacements of these targets are shown in Fig. 4. The first two graphs show variations in water level and in the average of the temperatures measured by the thermometers embedded in the dam body during the time between the first and the last of the 20 triangulations under consideration. At the abscissa corresponding to the date of each triangulation, a segment parallel to the axis of ordinate shows the serial number of the measurement. The temperature graph is interrupted in the first half of 1954, between triangulations 15 and 16, owing to a break in the electrical system controlling the thermometers.

The other graphs of Fig. 4 show the radial displacements of the targets. A broken line shows, for each target, the radial displacements resulting from the various triangulations and referring to the position the target had at the time of the first triangulation. This way of plotting the results makes it clear that, after a first settling down, the dam has begun to oscillate across a mean position of equilibrium which is fairly exactly determined. It is to be remarked that, as far as the upper part (approximately the upper third) of the dam is concerned, this mean position can already be identified after the fourth triangulation, i.e., when only nine months of observations had been made and the free surface of the reservoir had not yet reached El. 500 m a.s.l. The three lower targets had instead settled down more than a year later, i.e., after the tenth triangulation.

The distance of these mean positions of the targets of the central cantilever from the positions determined at the time of the first triangulation is shown in Fig. 5. A broken line joins the representative points in order to show the approximately deflection line of the cantilever. It is to be noted that the upper half of the structure is deflected upstream, whereas the central part of the dam shows a smaller downstream deflection.

The graphs of Fig. 4 also show the fact that the amplitude of the oscillations of each target across the corresponding mean position tends to decrease with time.

The correlation between the displacements and the variations of temperature and water level is evident; thermal variations have greater influence than water level variations.

Table 1

Triangulation Measurements

sure nts	Date of measurement	Mean water level [m a.s.l.]
st	from 3rd to 10th October 1950	435.49
nd	from 15th to 25th November 1950	474.20
rd	from 20th March to 3rd April 1951	500.22
.th	from 8th to 21st July 1951	497.62
th	from 30th September to 11th October 1951	522.99
th	from 15th to 25th November 1951	529.96
'th	from 26th February to 5th March 1952	492.28
3th	from 6th to 20 April 1952	484.11
)th	from 1st to 11th July 1952	525.43
)th	from 28th September to 7th October 1952	528.82
.th	from 9th to 19th January 1953	505.81
2th	from 27th March to 12th April 1953	475.65
3th	from 17th to 23rd July 1953	517.74
4th	from 18th to 25th November 1953	529.00
5th	from 25th to 28th February 1954	497.59
6th	from 29th April to 1st May 1954	473.27
7th	from 8th to 10th July 1954	527.80
8th	from 31st August to 2nd September 1954	527.09
9th	from 16th to 18th November 1954	509.67
0th	from 31st January to 2nd February 1955	501.74

SANTA GIUSTINA DAM

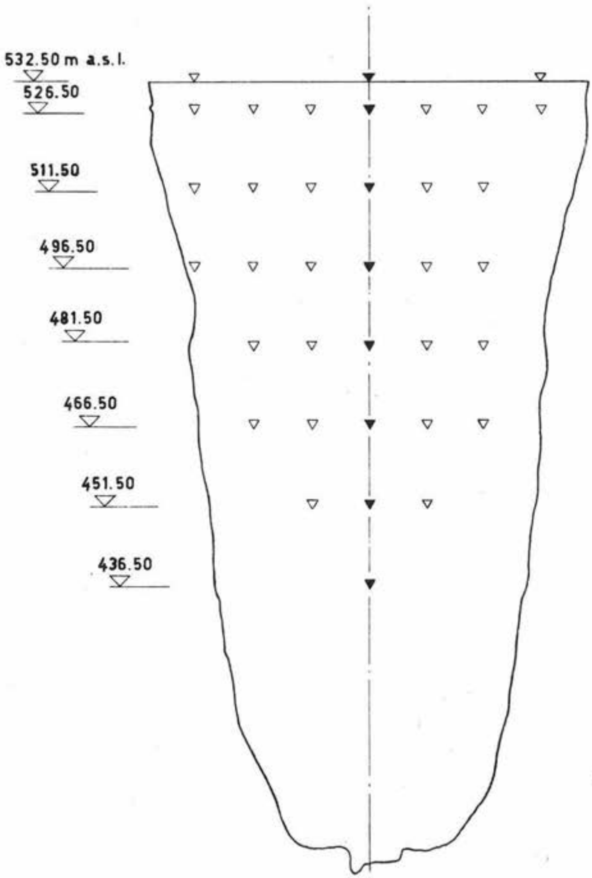


FIGURE 3 - TRIANGULATION TARGETS AT DOWNSTREAM FACE

Measurements with the Extensometers—Strains and Stresses

The Santa Giustina dam acts essentially as an arch dam: for this reason it was provided with a system of extensometers designed to control only one cantilever, the central one, and four arches, at Els. 406, 451, 481 and 526 m a.s.l. respectively.

The purpose of this installation was to measure the strains at the boundary of the sections at the crown and at the abutments.

To this end, twelve sets of attachments for removable extensometers were provided on the downstream face of the dam and they were specifically: one set at the crown and two sets at the abutments of each of the four arches in question. Each set of attachments allows for four positions of the extensometer: one vertical, one horizontal and two at 45^0 to the others. In the upstream side of the dam, at a distance of approx. 25 cm from the face, one horizontal electroacoustical extensometer was embedded in the concrete near to both abutments; one horizontal and one vertical extensometers were similarly embedded in the crown section.

Immediately to the left of the crown section of the four arches in question, two more sets of attachments for removable isolated extensometers were provided on the downstream face (these attachments allow for only horizontal and vertical positions); one isolated electroacoustical extensometer was similarly embedded vertically in the concrete near to the upstream facing. Each downstream set of attachments was placed in a region isolated from the rest of the structure by means of a peripheral groove; this groove was cut to a sufficient depth to prevent the transmission of stresses. The strains measured in these conditions are due only to local temperature variations, to shrinkage or swelling of concrete or to other causes, excluding however the stresses.

Strains due only to stresses are obtained by subtracting the strains measured by isolated extensometers from the strains measured by the other ones. The stresses may then be found by means of well known formulae of the Mechanics of Continua.

Obviously, the extensometers only allow the evaluation of the stress variations occurring between two subsequent measurements: the evaluation of the actual stresses existing when the second measurement is taken requires a knowledge of the stresses existing during the first measurement.

* * *

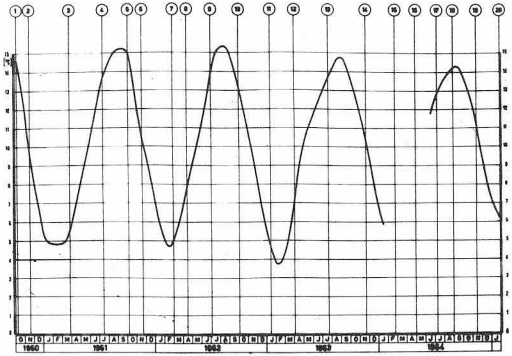
The strain values now available are only those obtained from the removable extensometers in the periods beginning shortly before the third triangulation, for the arches at Els. 406 and 461, and shortly before the fourth triangulation, for the arches at Els. 481 and 526.

These removable extensometers are mounted on a 800 mm base and have an amplification ratio equal to 250.

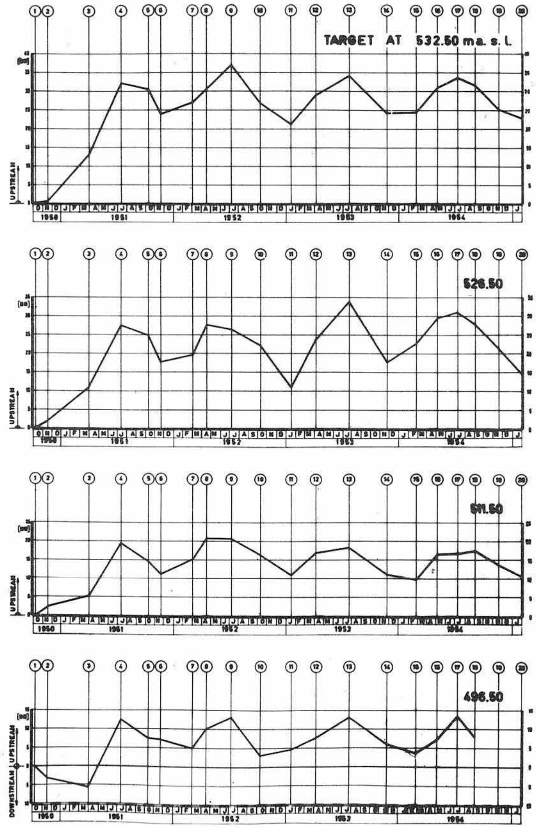
The results of the strain measurements have been elaborated with the purpose of determining the unit strains and the stress variations occurring between the first triangulation, for which extensometrical measurements are available, and subsequent ones. The following method was adopted.

As already said, readings on the extensometers are usually taken once a

AVERAGE TEMPERATURE OF THE DAM



RADIAL DISPLACEMENTS OF THE TARGETS PLACED AT THE CROWN OF THE ARCHES



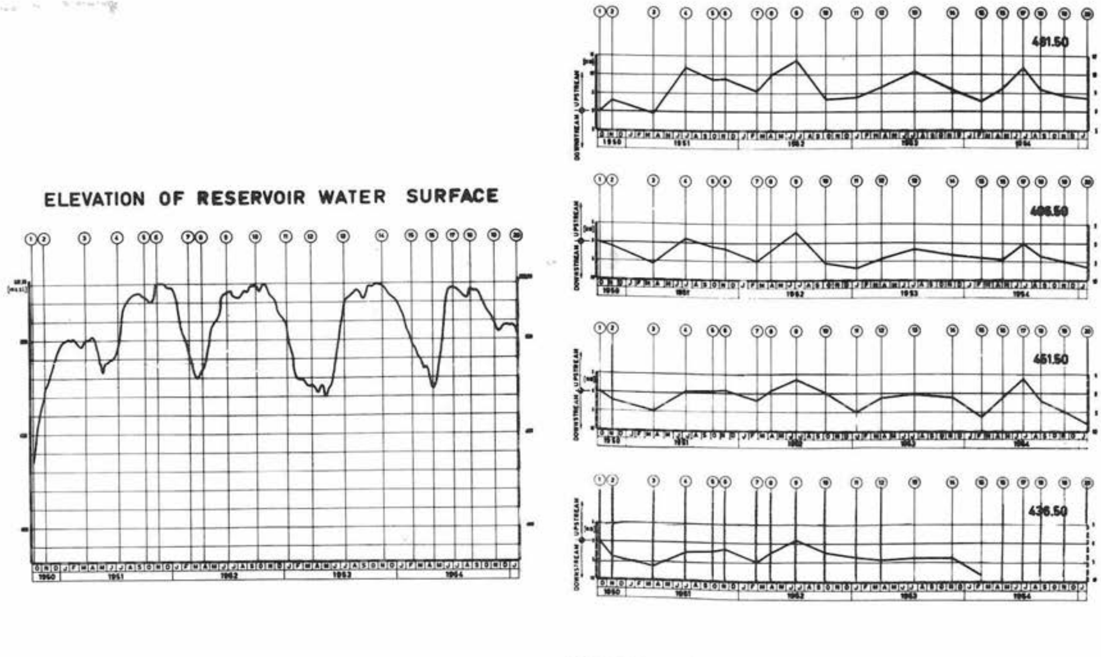


FIG. 4 - TEMPERATURES; WATER SURFACE LEVEL; RADIAL DISPLACEMENTS OF THE TARGETS PLACED AT THE CROWN OF THE ARCHES.

Fig. 4. Temperatures; water surface level; radial displacements of the targets placed at the crown of the arches.

SANTA GIUSTINA DAM

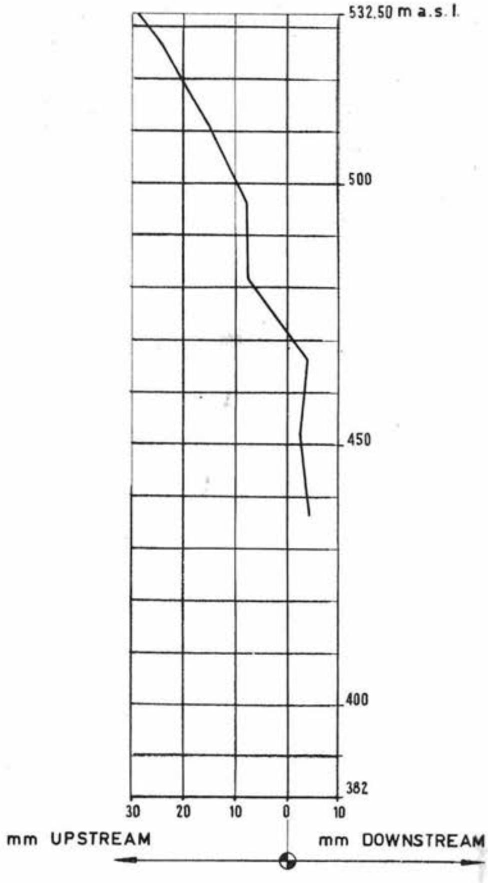


FIGURE 5 - AVERAGE RADIAL DISPLACEMENTS OF THE TARGETS PLACE AT THE CROWN OF THE ARCHES, WITH RESPECT TO THE POSITIONS RECORDED DURING THE FIRST MEASUREMENT

SANTA GIUSTINA DAM

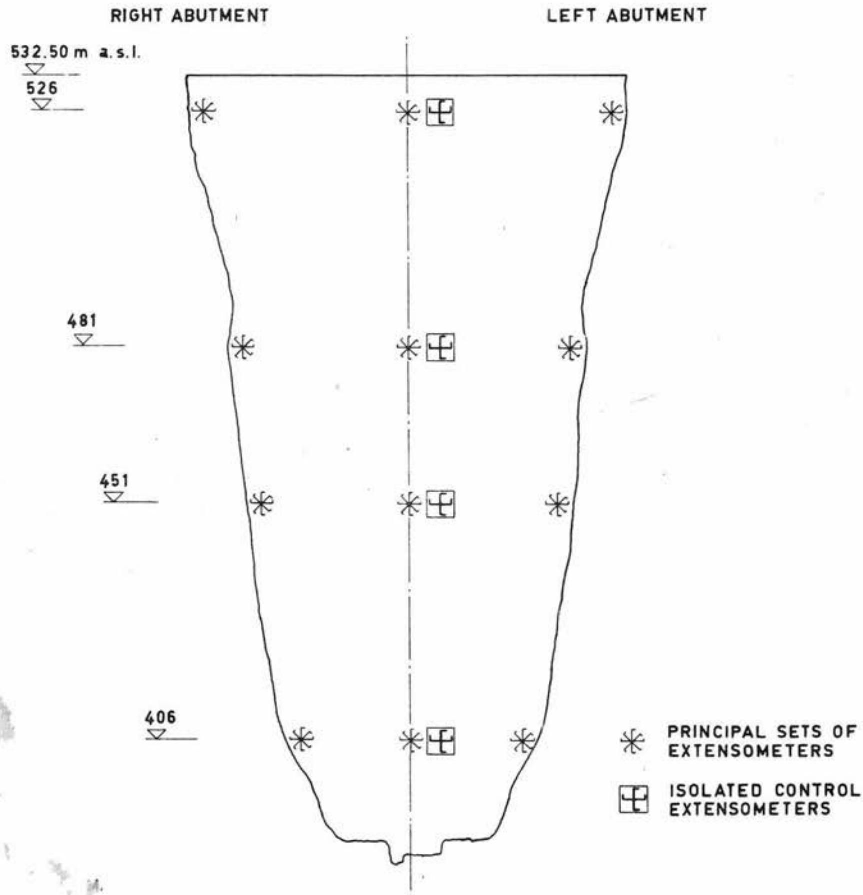


FIGURE 6 - REMOVABLE EXTENSOMETERS: MEASUREMENT POINTS

week. When, during one triangulation, more than one reading was taken, then their average was introduced into the calculations. The total strains detected by the extensometers were calculated as the difference between the values corresponding to each triangulation and the ones corresponding to the reference triangulation (triangulation No. 3 for the arches at El. 406 and El. 451, and triangulation No. 4 for the upper two arches). The average of the vertical and horizontal strains shown by the isolated extensometers placed in the same arch was subtracted from these total strains.

The strain values thus calculated were further revised to allow for measurement errors; in this we took advantage of the fact that, in a plane strain state, the sum of the unit strains along two perpendicular directions is the same whatever the two directions may be: this means that it must be:

$$\epsilon_h + \epsilon_v = \epsilon_a + \epsilon_b$$

where ϵ_h and ϵ_v are the horizontal and vertical strains at a certain point of the facing, and ϵ_a and ϵ_b are the analogous strains at 45^0 to the former.

The measured values did not usually agree perfectly with this condition: the difference between the strain sums, if any, was then distributed among the single strains, in proportion to their absolute values. The strains due to stresses are represented by the values corrected in this way.

These strains are shown graphically in Fig. 7. For each of the twelve measurement points four broken lines have been plotted, corresponding to the strains measured at the time of each triangulation along the four directions already mentioned.

These graphs clearly show the almost periodic character of the strain variations, which are closely correlated with the yearly temperature cycle. On the whole, strains are more regular at the crown of the arches than at the abutments.

A less discontinuous and still more clearly periodic character than that appearing in Figs. 7, 8 and 11 is likely to be shown by the strains and consequently by the stresses which will be obtained from elaborating the other extensometer readings, besides the ones taken at the time of traingulations. This elaboration is now being done.

* * *

For the calculation of the specific stresses, or better of their variations, in the arches (horizontal stresses) and in the cantilevers (vertical stresses), the following formulae have been used:

$$\sigma_a = E \frac{\epsilon_h + \nu\epsilon_v}{1 - \nu^2}$$

$$\sigma_c = E \frac{\epsilon_v + \nu\epsilon_h}{1 - \nu^2}$$

$$\tau = \frac{E}{2(1 + \nu)} (2\epsilon_a - \epsilon_h - \epsilon_v),$$

where Young's modulus of elasticity of concrete E was assumed to be equal to 400,000 kg/cm^2, based on the results of measurements carried out by means of a soniscope, and Poisson's ratio to be equal to 1/6.

The results are plotted in Fig. 8: this shows the progressive stress variation indicated by the extensometers, which is referred to the unknown stress values at the time of the third triangulation, for the two lower arches, and at the time of the fourth one, for the two arches at Els. 481 and 526 m.

Observations similar to those made on Fig. 7 for the strains can also be made on Fig. 8, and they are specifically:

- the stress variation shows an annual periodicity and is mainly influenced by the temperature cycle;
- stresses are more regular at the crown;
- the continuity and the periodicity of stress variation will be clearer when the elaboration of all the data is completed.

The explanation of some results may then be easier, in particular the explanation of the high vertical strain values found at the left abutment at Els. 526 and 481 and eventually of the high stress values calculated from the former strains.

It can also be noted that the principal directions are not far from being horizontal and vertical respectively.

Temperature Measurements

The measurement of temperature was limited to the central cantilever of the dam, on account of the orientation of the axis of the gorge being in a North-South direction and because of its U-shape. Eleven electrical thermometers were embedded in this central cantilever and in particular in the arches at Els. 527, 480, 455.50 and 407.50. In each arch, two thermometers were placed at a distance of 35 cm from the two faces and one at an equal distance from them: this latter was not provided in the arch at El. 506, where only the two thermometers at a 35 cm distance from the faces were placed.

The arrangement of the electrical thermometers is shown in Fig. 9, which also includes thermometer No. 12, placed on the downstream face at El. 480 to measure the air temperature, and thermometer No. 13, placed on the upstream face at El. 430 to measure the water temperature. This latter was continuously immersed in water from the time of the first filling of the reservoir. Readings are taken from all these thermometers every day.

* * *

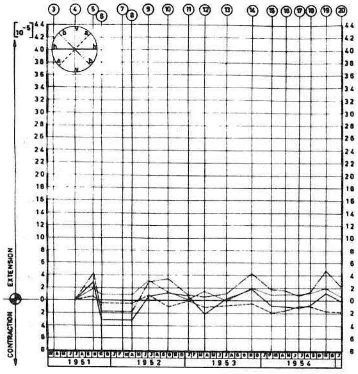
The results of temperature measurements were first worked out to find the time at which the setting heat may be considered as practically disposed of, i.e., the time after which the initial aperiodic thermal transient is over, the temperature variations in the dam consequently showing a periodical character, which is controlled by the seasonal cycle of temperature changes acting on the two faces of the dam.

The yearly moving averages of the temperature values measured by each of the thermometers embedded in the dam have been calculated with this purpose: this means that, month by month, the averages of the temperatures recorded by the same thermometer in the previous twelve months have been

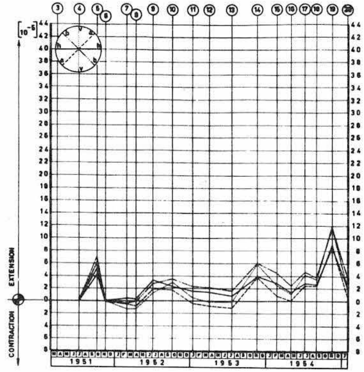
SANTA GIUSTINA DAM

ARCH AT ELEVATION 526 m a.s.l.

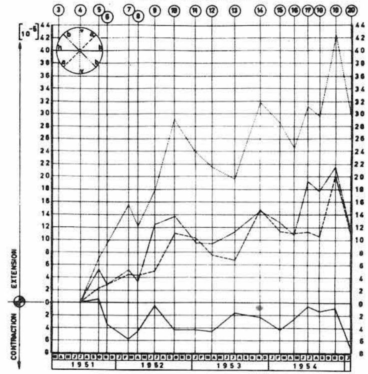
RIGHT ABUTMENT



CROWN

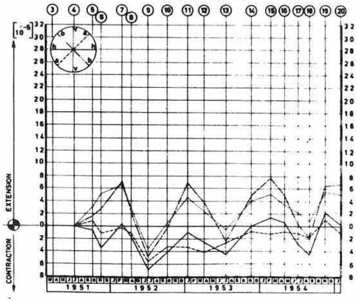


LEFT ABUTMENT

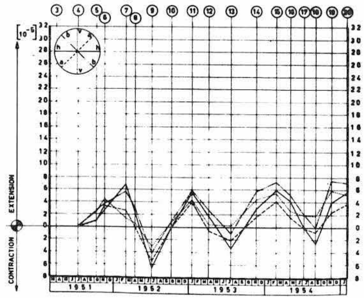


ARCH AT ELEVATION 481 m a.s.l.

RIGHT ABUTMENT



CROWN



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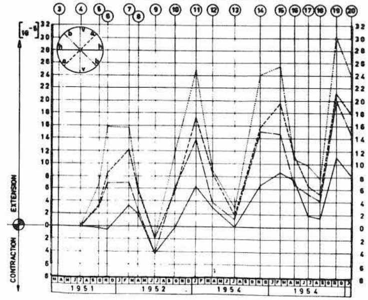


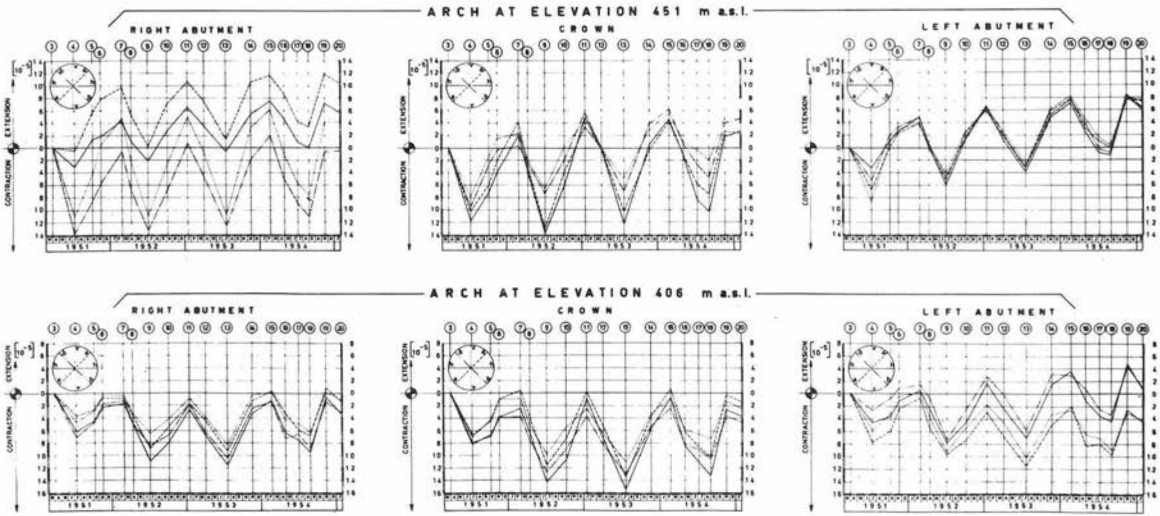
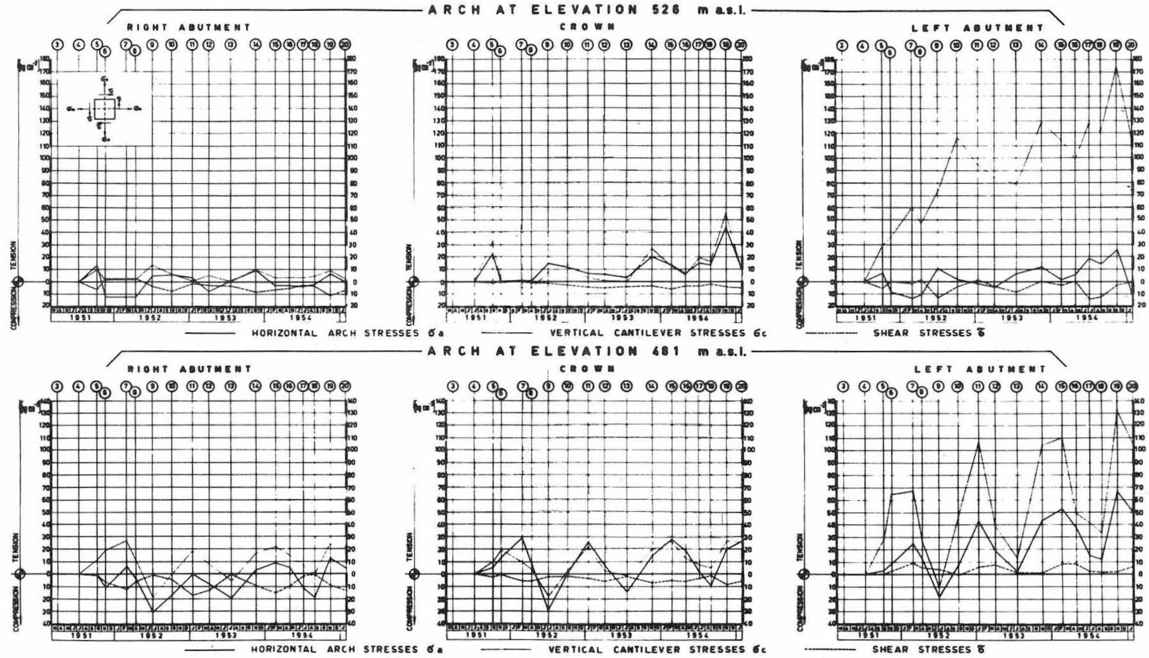


FIGURE 7 - OBSERVED STRAINS WITH RESPECT TO THE STATUS RECORDED DURING THE THIRD OR FOURTH TRIANGULATION

SANTA GIUSTINA DAM



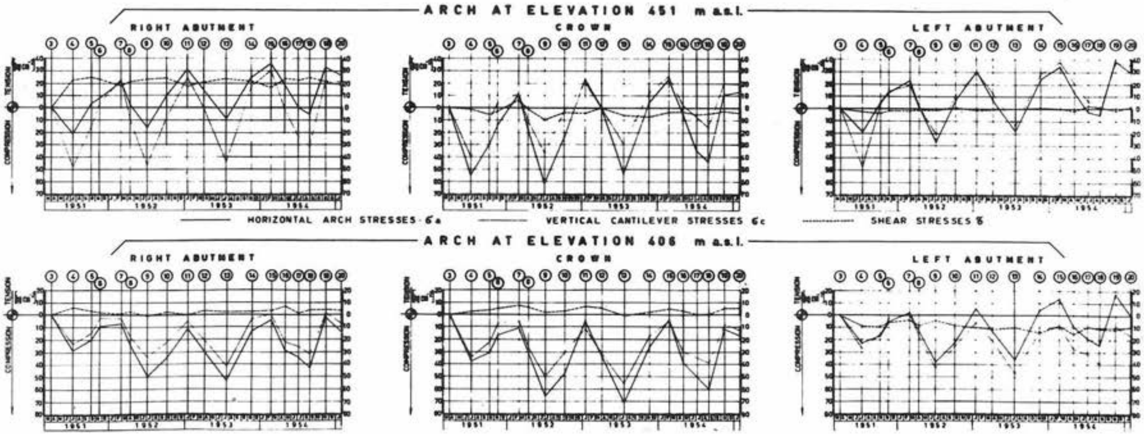


FIGURE 8 - OBSERVED STRESSES WITH RESPECT TO THE STATUS RECORDED DURING THE THIRD OR FOURTH TRIANGULATION

computed, obviously starting from one year after the beginning of observations. These averages are plotted in Fig. 9.

When the temperature measured by a thermometer is influenced only by the seasonal cycle of temperature changes, the average for twelve consecutive months is practically independent of the group of months to which it applies. On the contrary, during the first period of the dam's existence, the dispersal of the setting heat makes this yearly moving average decrease with time. It can therefore be concluded that from the time at which this moving average becomes constant, the aperiodic component can no longer be superimposed on the periodic component of temperature, i.e., the setting heat has no more influence.

The graphs of Fig. 9 show in particular that:

- in the upper 70 m of the dam, i.e., above the elevation of thermometers nos. 4, 5 and 6, the presence of setting heat had already been limited to the inner part of the dam, since early autumn 1951 (i.e., 2-3 years from construction) and had faded away in the next few months;
- in the lower 30 m above the foundations, the setting heat was still remarkable in the outer portions too, where it lasted for another year, i.e., for five years after construction. Unfortunately, a break in thermometer No. 2 made it impossible to carry out the same investigation for the inner core of this lower portion of the dam.

* * *

The temperature measurements were elaborated further as far as they were necessary for the calculation of the deflections and stresses, which were to be compared with those actually observed.

First of all, the thermal conditions of the dam during each triangulation were found. The isothermal lines in the central cross section were then drawn, taking into account the average temperature recorded by each thermometer during the triangulation in question.

From these isothermal lines it was possible to construct diagrams showing the temperature distribution between the two faces of the dam at 15 different elevations.

Each of these diagrams was then replaced by an equivalent trapezoidal diagram having its gravity center on the same vertical line; this was done because the subsequent calculations could only take into account a linear distribution of temperature across the dam thickness.

These diagrams were used to calculate the differences D between the upstream and downstream face temperature, besides the average temperature t at the 15 elevations mentioned above.

The thermal changes undergone by the dam between two triangulations were assumed to be represented by the corresponding variations of t and D.

Statical Analysis of the Dam According to the Tölke and Ritter Methods

So far calculations have been carried out, to find the deflections which might be expected at the arch crowns and the stresses at the strain measurement points at the time of each triangulation.

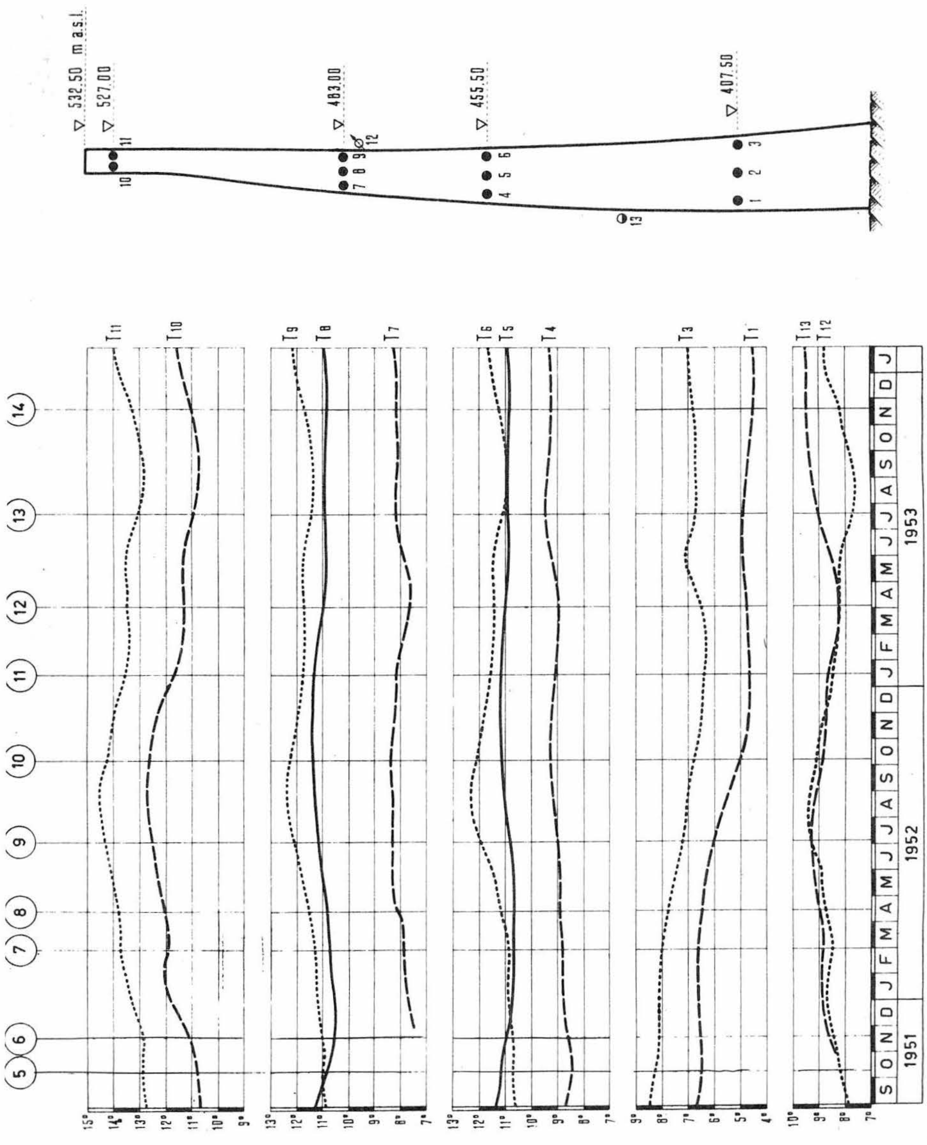
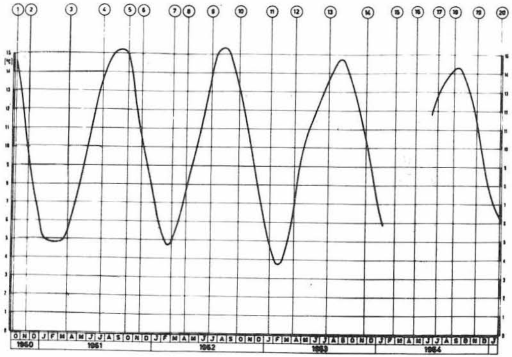


FIGURE 9

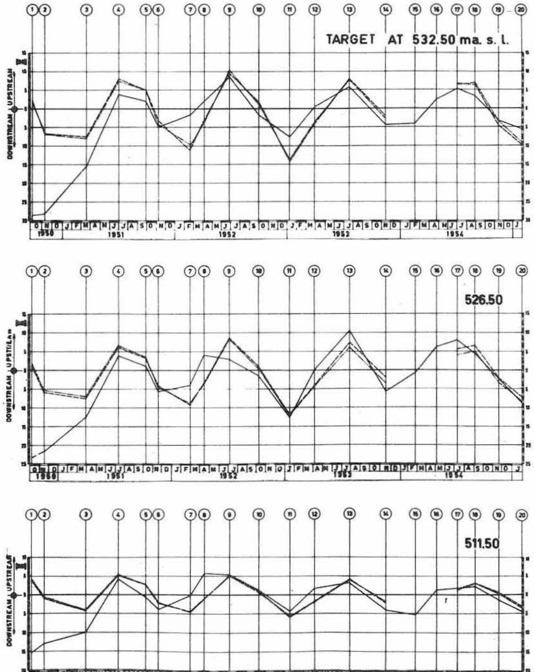
ARRANGEMENT OF THE ELECTRICAL THERMOMETERS; YEARLY MOVING AVERAGE OF RECORDED TEMPERATURES

AVERAGE TEMPERATURE OF THE DAM



RADIAL DISPLACEMENTS OF THE TARGETS PLACED AT THE CROWN OF THE ARCHES

———— OBSERVED VALUES
– – – – CALCULATED VALUES ACCORDING TO TÖLKE'S METHOD
–·–·–· CALCULATED VALUES ACCORDING TO RITTER'S METHOD



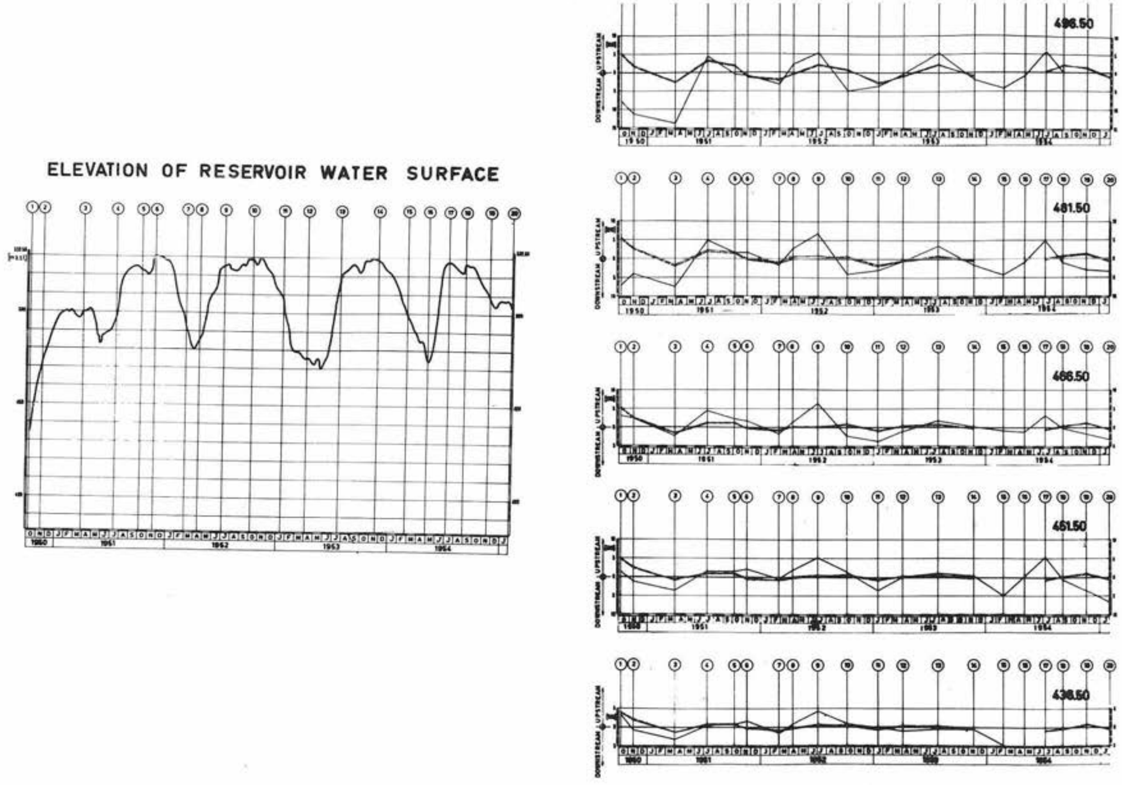
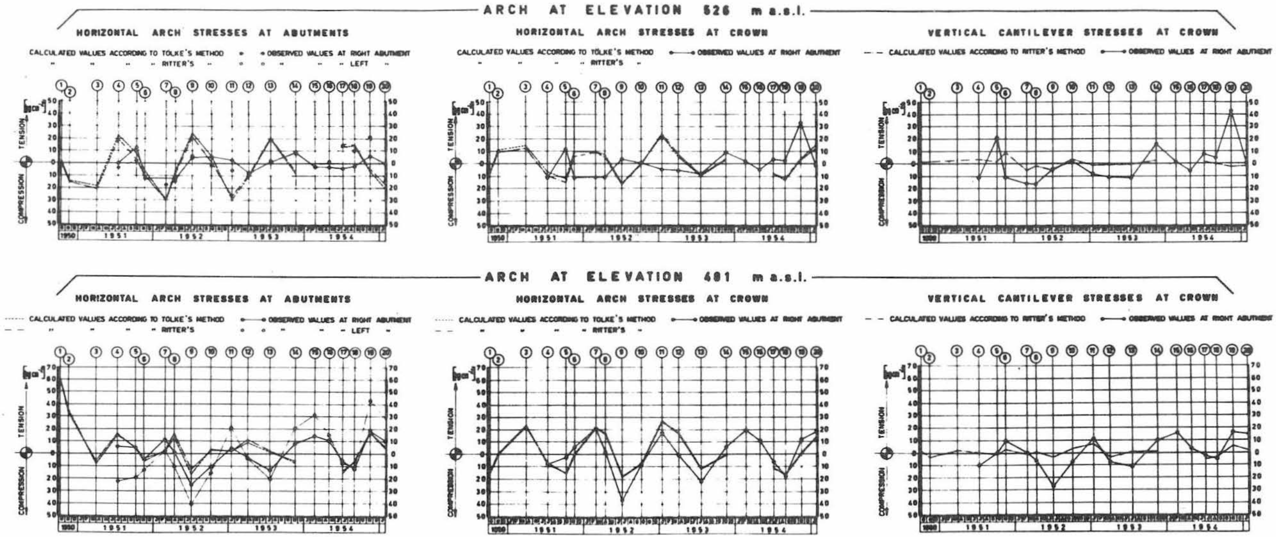


FIG.10 - TEMPERATURES; WATER SURFACE LEVEL; CALCULATED AND OBSERVED RADIAL DISPLACEMENTS OF THE TARGETS PLACED AT THE CROWN OF THE ARCHES, WITH RESPECT TO THEIR AVERAGE POSITIONS

Fig. 10. Temperatures; water surface level; calculated and observed **radial displacements** of the targets placed at the crown of the arches, with respect to their average positions.

SANTA GIUSTINA DAM



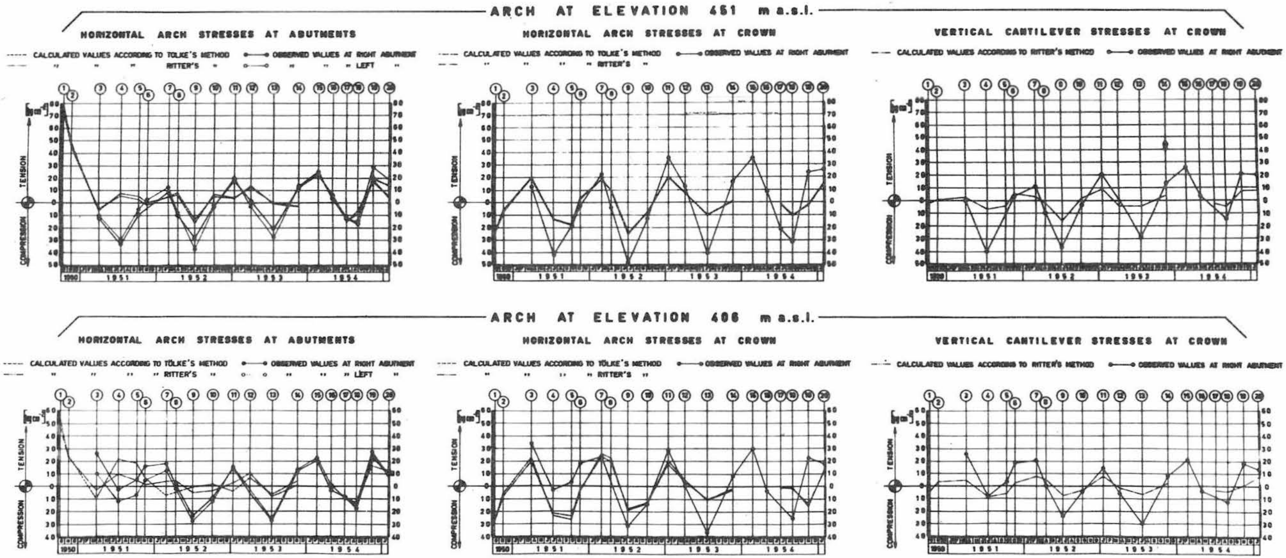


FIGURE 11 - CALCULATED AND OBSERVED VARIATIONS OF STRESSES AT DOWNSTREAM FACE, WITH RESPECT TO THEIR AVERAGE VALUES

In these calculations, the behaviour of the dam under water level and temperature variation was studied taking into account its tridimensional shape according to four different methods; only part of the results of two of these methods are stated in this preliminary report.

The first method is Tölke's; it is used in the form suggested by this author in the case of cantilever thickness and thermal variations depending on elevation according to any law.

The differential equation of the fourth order to which this method leads, cannot be exactly solved. The finite differences method was therefore used for its integration, the central cantilever being divided into 16 sections. In this way, systems of 21 linear algebraical equations with 21 unknowns were obtained. The number of systems was equal to the number of triangulations taken into consideration in the analysis of the dam. The subsequent analysis of the arches, which were assumed to be independent and subject to the actions pertaining to each of them according to the method mentioned above, was carried out in accordance with Tölke's theory for arches with a circular centerline and thickness slightly increasing from crown to abutments.

The second method considered here is the "arch-cantilever method." On account of the U-shape of the gorge, the radial deflections of the arches were made to agree with the deflections of the central cantilever only, according to this method's original form as conceived by Ritter. It was however generalized to take into account the temperature changes acting on the dam, besides the water pressure.

The analysis was carried out on 16 arches, using systems of 16 linear algebraical equations with 16 unknowns.

In these calculations the dam bottom was assumed to be at El. 382 m a.s.l.

Yielding of the foundation rock was not taken into consideration in either method; this will be done only in a second stage, in order to separate the influence of rock yielding from other influences.

* * *

Calculations have been carried out for 18 of the 20 triangulations which are mentioned in this first report; triangulations Nos. 15 and 16 could not in fact be analyzed since the break in the thermometer station made it impossible to record the thermal conditions of the dam at that time.

The results are summarized in Figs. 10 and 11. The first shows a comparison between the deflections of the arch crowns calculated by the two methods, and the deflections actually recorded at time of the triangulations; all deflections are measured from the mean position. The results of the two methods are in quite good agreement; their agreement with the experimental results must be considered good, especially if the complexity of the problem is taken into account. Agreement is particularly good for the upper arches, except in the observations of the first half year, which was affected by the previously mentioned settling down of the dam; this will be investigated further.

Fig. 11 shows a comparison between the stresses (stress variations) at the measurement points of the removable extensometers, as calculated by the two methods, and the stresses actually measured. In this case too, agreement between the two methods is very good. As was to be expected, the agreement of the results of these methods with the experimental data is not very good, as far as the abutments are concerned; whereas it is fairly good for σ_a at the crown, at least in the three lower arches. Furthermore, the diagrams on the right of the figure prove that the cantilever is actually more heavily loaded than both methods show.

Stresses too are much more influenced by temperature changes than by water level variation.

* * *

As was stated above, this paper gives only a brief account of some of the results of a much more considerable analysis which is still being carried out; more definite conclusions are expected after this analysis is completed.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: THE RENO DI LEI DOUBLE-CURVATURE ARCH DAM

Claudio Marcello,[1] M. ASCE
(Proc. Paper 994)

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams held in April, 1939, much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be included in the Symposium. So far, nine papers have been approved: "Arch Dams: Their Philosophy" (Proc. Paper 959) by Andre Coyne; "Arch Dams: Trial Load Studies for Hungry Horse Dam" (Proc. Paper 960) by R. E. Glover and Merlin D. Copen; "Arch Dams: Portuguese Experience with Overflow Arch Dams" (Proc. Paper 990) by A. C. Xerez; "Arch Dams: Theory, Methods, and Details of Joint Grouting" (Proc. Paper 991) by A. Warren Simonds; "Arch Dams: Santa Giustina Single-Curvature Arch Dam" (Proc. Paper 992) by Claudio Marcello; "Arch Dams: Measurements and Studies on Santa Giustina Dam" (Proc. Paper 993) by Claudio Marcello; "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam" (Proc. Paper 994) by Claudio Marcello; "Arch Dams: Isolato Double-Curvature Arch Dam" (Proc. Paper 995) by Claudio Marcello; and "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs" (Proc. Paper 996) by Claudio Marcello.

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

Note: Discussion open until November 1, 1956. Paper 994 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 3, June, 1956.

1. Cons. Engr. and Technical Director, Societa Edison, Milan, Italy.

SYNOPSIS

This paper outlines the design by the Trial-Load Method of an unusual Italian arch dam. The centerlines of the horizontal arches are shaped to approximately coincide with the funicular curves of the arch loads. Design is not yet complete and will be verified by model studies.

The purpose of the Reno di Lei dam is to create in the Lei Valley (Retic Alps), at an elevation of over 1900 m a.s.l., a seasonal reservoir with a storage capacity of 200 million cubic metres, for regulation of the output of a system of 3 hydroelectric plants in series, with power station at Innerferrera, Andeer and Sils.

These plants belong to the Kraftwerke Hinterrhein A.G., a Swiss-Italian Company in which Soc. Edison of Milan also partakes.

The design of the dam was developed in 1954-1955. A small-scale model of the dam is now under construction to provide a mean for checking and completing design calculations and to suggest the possible adjustment of the structure design capable of improving its statical behaviour.

The main geometrical data of the designed arch dam are:

— maximum crest height above the deepest point of foundations	138 m
— thickness at crest	15 m*
— thickness at base	28 m
— length of crest arch	635 m
— chord of crest arch	520 m
— rise of crest arch	160 m
— volume of concrete	800,000 m^3

The crown section of the upstream face projects by 28 m at the crest level and by about 5 m at the base (Fig. 3).

The horizontal sections of the dam are arches of uniform thickness from crown to abutments. Their centerline coincides approximately with the funicular curve of the load supported by the arches, (see "Statical analysis"). The crest is at El. 1932, which is the maximum flood water level; a 2 meter high concrete wall will provide adequate protection against possible waves.

On the left bank, from a point about 13 m below the crest elevation, the dam will be placed against a concrete thrust block.

For construction purposes the dam body will be divided into blocks by temporary joints to minimize the tensile stresses due to shrinkage.

These joints will be about 15 m apart, and, after shrinkage, will be filled with concrete and grouted.

Possible pore pressure will be eliminated, or at least minimized, by means of a system of vertical drain pipes arranged behind the upstream face; drainage collectors and inspection galleries are also provided in the dam body (Fig. 4).

---

* This value of thickness was fixed by the Swiss military authorities, since the basin laying below the reservoir is in Swiss territory.

Statical Analysis

The stability analysis was made taking into consideration the reciprocal contribution to stability of the horizontal sections (arches) and of the vertical sections (cantilevers) (Fig. 6).

The dam was divided into 8 equally spaced horizontal arches of one unit height and into 15 vertical cantilevers. The dam is perfectly symmetrical with respect to the vertical plane at the crown of the arches and therefore it has been possible to limit the calculations to the cantilevers of only one half of the dam (8).

The arches are parabolic, of even thickness, and the equation of their centerline is of the type $y^2 = Wx$, where the parameter W is variable in function of the elevation.

The cantilevers are vertical dam sections of one unit width: their centerline is the intersection of the surface containing the centerlines of the arches with a vertical plane approximately normal to the dam faces. At each elevation, the thickness of the cantilever is equal to the thickness of the corresponding arch. Each cantilever has its base at an elevation which is the arithmetical average of the elevations of two consecutive arches.

The analysis has in general been carried out on the basis of the trial-load method, revised to fit parabolic arches and suitably simplified; in particular, deformations due to shear and twist actions have not been considered in the analysis of both the arches and the cantilevers, whereas the elastic movements of abutments and foundations have been considered, taking into account the tilting induced by the bending moment and shear, as well as the displacements induced by normal stress, shear, and bending moment.

At the foot of each cantilever the concrete was assumed not to be able to withstand tension as if the cantilever were cracked; consequently the active section at the base, in respect to the capacity of being deformed of the foundations, is only the area which will ultimately result in being subjected to compression, and was therefore determined by a series of approximations.

For the calculation of the factors of influence of the arches and cantilevers, the load assumptions were: unit forces at the intersection points, for cantilevers, and funicular loads with unitary mean value, for arches. Funicular loads are loads having a funicular curve coincident with the arch centerline.

The subdivision of the hydrostatic load between arches and cantilevers was obtained by bringing into agreement the horizontal cantilever deflections and the projection of the arch deflections onto the center plane of cantilevers, and then checking the results obtained against the assumptions made as regards both the reacting section at the foot of the cantilevers and the load distribution on the arches.

Only the deflections due to the radial components of the parts of hydrostatic load pertaining to the arches and to the cantilevers were considered.

A 5% allowance was accepted in the agreement of deflections in order to make the load surface more uniform since, owing to the simplifications assumed at the beginning, the perfect identity of deflections would have produced unacceptable irregularities on this surface. Anyway, a more accurate calculation will be carried out by means of an electronic computer, to obtain the complete agreement of arch and cantilever deflections.

The stability analysis of the arches was then made by the theory of elasticity, taking into account the parts of hydrostatic load and the effects of the elastic yielding of abutments, and assuming a sinusoidal variation of the temperature between $\pm$ 10°C.

The analysis of the lower arches, these being the thicker ones, was carried out with the methods suitable for thick arches and taking into consideration only their part of hydrostatic load.

No allowance was made for shrinkage effects because provision of joints was made. These will be sealed only after complete concrete curing and under temperature conditions close to the average of the expected temperature fluctuations.

The analysis of the cantilevers was made taking into account the part of hydrostatic load, the dead weight of the cantilevers and the uplift pressure, assumed as linearly variable through the thickness.

The maximum stresses are recorded in the following tables. For the arches, the peak values were 74 Kg/cm^2 for compressive stress and 6 Kg/cm^2 for tensile stress in the assumption of simultaneous hydrostatic pressure and thermal effect at the maximum assumed values. With an empty reservoir the maximum value of thermal stress is very small ($\pm$ 2 Kg/cm^2).

For the cantilevers, the maximum values were 60 Kg/cm^2 for compressive stress and 18 Kg/cm^2 for tensile stress respectively.

Model

As mentioned above, a small-scale model of the designed dam is now being constructed. It will be made of a suitably processed concrete.

This model will undergo the statical tests necessary to determine the deflections, stresses and layout of isostatical lines of both full and empty reservoirs.

Ultimate strength tests will also be carried out as far as breaking point.

Waterproofing

The dam is to be built on a site consisting of paragneiss and mica-schist rock banks with strata parallel to the slope on the western bank and normal to the slope on the eastern bank.

The rock, as far as stability and watertightness are concerned, is generally sound.

A layout of waterproofing and consolidation groutings is being studied, based on a careful geological survey, and on the outcomes of a set of core drillings purposefully carried out.

The design of the dam was entrusted to Soc. Edison and performed by this Company's Hydroelectric Plant Construction Department which is directed by the writer.

It is in the intention of the writer to go farther into the object of this paper as soon as the design of the dam is finally completed.

RENO DI LEI
Parabolic arch dam -

TABLE OF STRESSES ON THE ARCHES

Kg/cm^2

a) Elastic arches

Arch No.	stresses	Pressure on the arch		Pressure on the arch and temperature changes				Temperature changes with empty reservoir	
		Crown	Abutment	Crown Summer	Crown Winter	Abutment Summer	Abutment Winter	Crown	Abutment
8	e	33.10	54.72 (17.73)	28,78	37.42	51.63 (14.19)	57.81 (21.27)	∓ 0.46	± 0.72
	i	48.77	27.07 (64.00).	53,13	44.41	30.19 (67 57)	23.95 (60.43)	± 0.47	∓ 0.69
7	e	39.65	24.52	35,36	43.94	21.67	27.37	∓ 0.55	± 0.88
	i	42.13	57.61	46,48	37.78	60.50	54.72	± 0.61	∓ 0.92
6	e	45.41	16.91	41,24	49.58	14.42	19.40	∓ 0.62	± 1.06
	i	35.54	64.57	39,80	31.28	67.12	62.02	± 0.71	∓ 1.00
5	e	46.69	8.36	42,57	50.81	6.30	10.42	∓ 0.73	± 1.30
	i	28.37	67.26	32,63	24.11	69.42	65.10	± 0.88	∓ 1.20
4	e	50.73	-4.57	46,62	54.84	-5.93	-3.21	∓ 0.89	± 1.68
	i	15.93	72.70	20,36	11.50	74.29	71.11	± 1.18	∓ 1.47

Note: the bracketed figures are referred to the section where the curve of pressures shows the maximum of excentricity.

b) Rigid arches

Arch No.	Pressure		Stresses (Kg/cm^2)				
	Hydrostatic t/m^2	on the arch t/m^2	After Mariotte: for hydrostatic pressure	After Lamé: for pressure hydrostatic	After Lamé: for pressure on the arch	After Résal: for pressure on the arch, Crown	After Résal: for pressure on the arch, Abutment
3	94.875	51.819	71.10	76.18	41.29	51.52	68.69
2	112.125	80.477	52.50	58.78	42.19	47.83	63.77
1	129.375	196.375	30.18	38.42	58.32	20.10	26.80

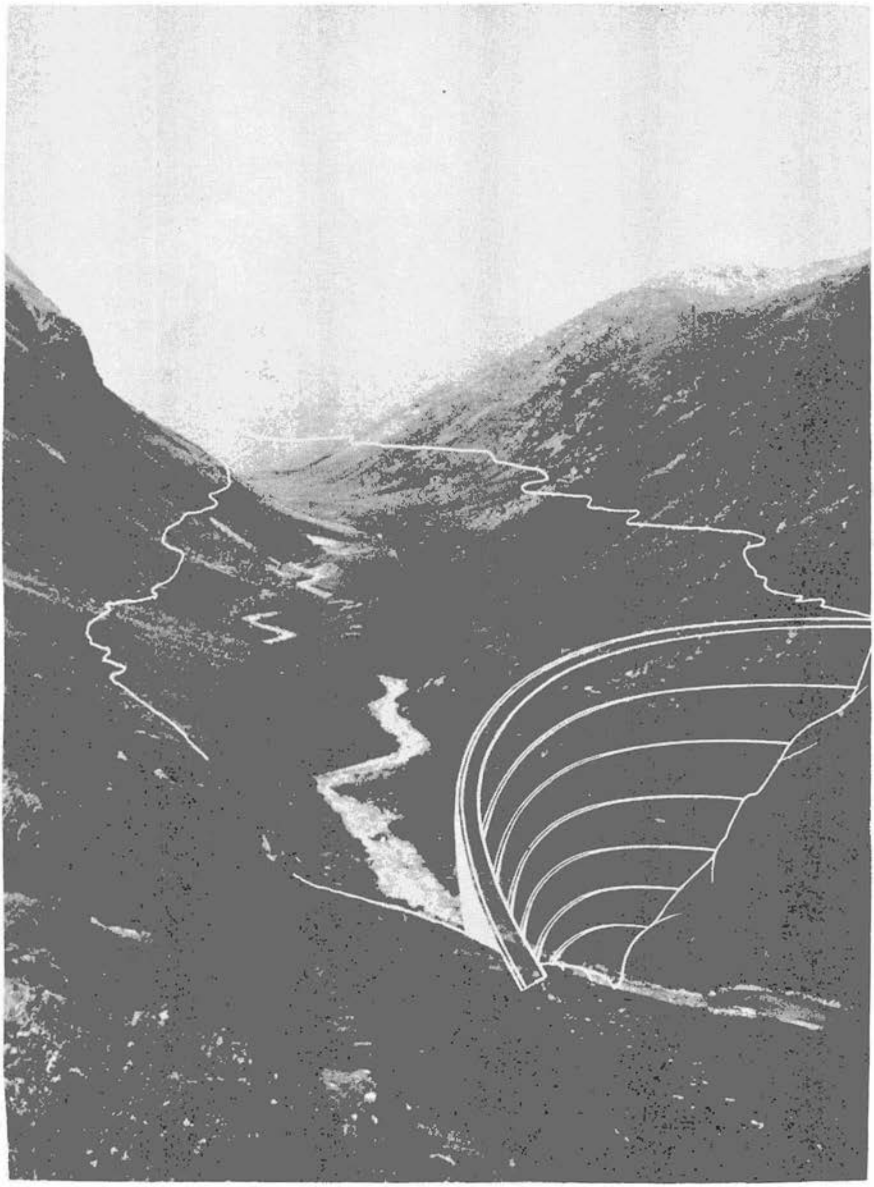
RENO DI LEI
Parabolic arch dam -

TABLE OF STRESSES ON THE CANTILEVERS

Kg/cm^2

Arches	Cantilevers															
	1		2		3		4		5		6		7		8	
	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream
8	5.03	2.22	5.47	2.15	5.58	1.55	5.43	1.65	3.92	2.95	1.74	4.48	0.32	6.12	-	7.20
7	I2.24	I.72	I4.4I	-0.6I	I4.26	-1.26	I2.43	0.46	9.33	2.82	-I.5I	I3.93	-	2I.39		
6	25.29	-3.26	22.94	-4.60	20.75	-4.06	I6.06	-I.07	4.62	I2.07	-	40.03				
5	I9.50	2.08	26.40	-6.30	I8.52	-0.49	2.56	I6.7I	-	37.29						
4	I4.73	5.88	I6.47	4.76	2.2I	I9.54	-	60.98								
3	-5.49	29.76	-0.72	24.00	-	56.58										
2	-I8.24	43.84	-	45.9I												
I	-	50.98														

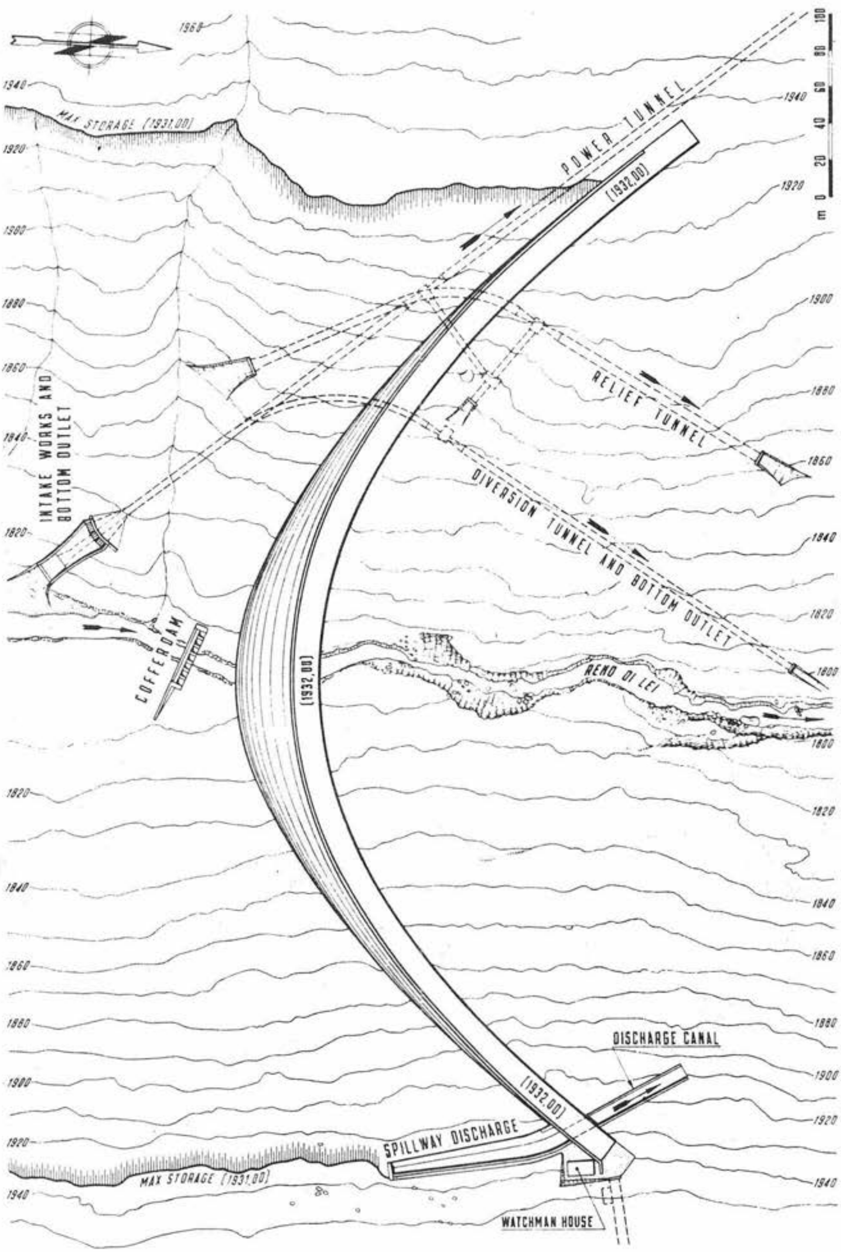
FIG. 1



View of the Reno di Lei Valley. The dam will have a maximum height of 138 m, a length at crest of 635 m, a chord of 520 m and thickness of arches varying from 15 m at crest to 28 m at base. The volume of concrete required for the construction of the dam is 800,000 m^3.

FIG. 2

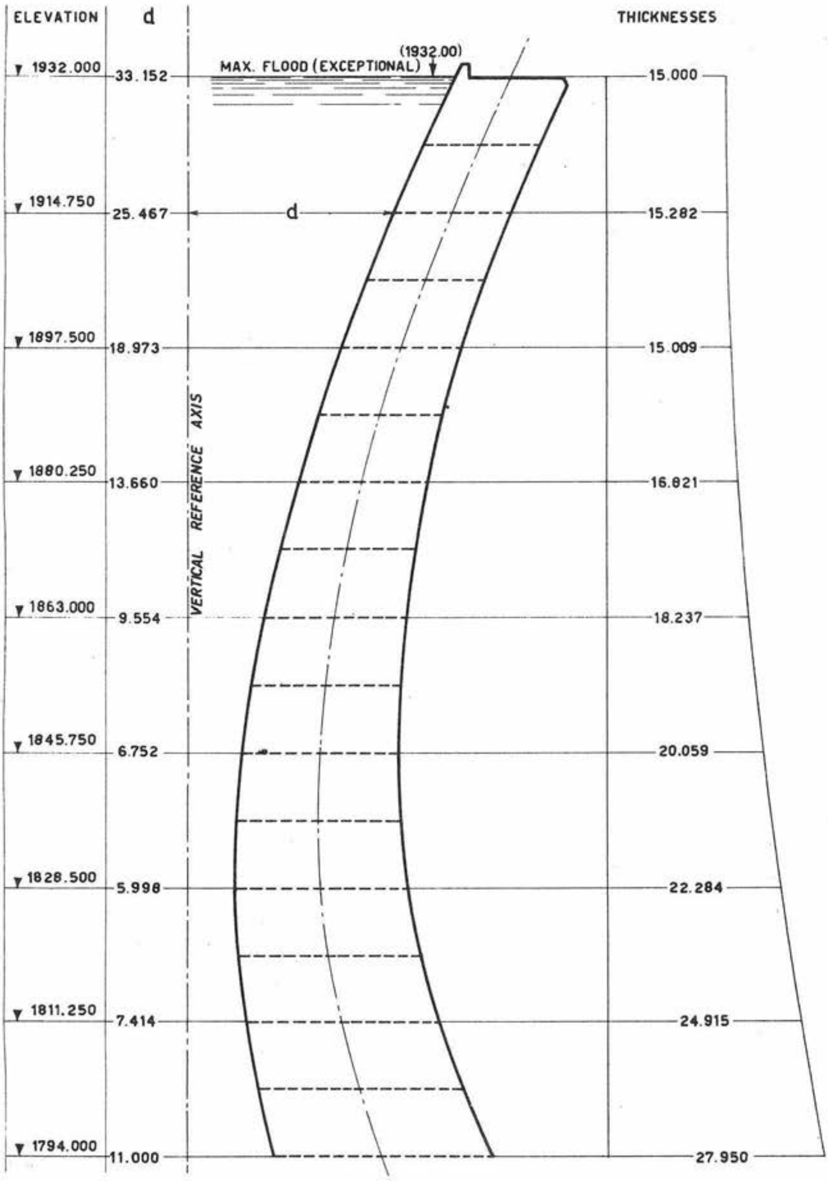
GENERAL LAYOUT



RENO DI LEI DAM

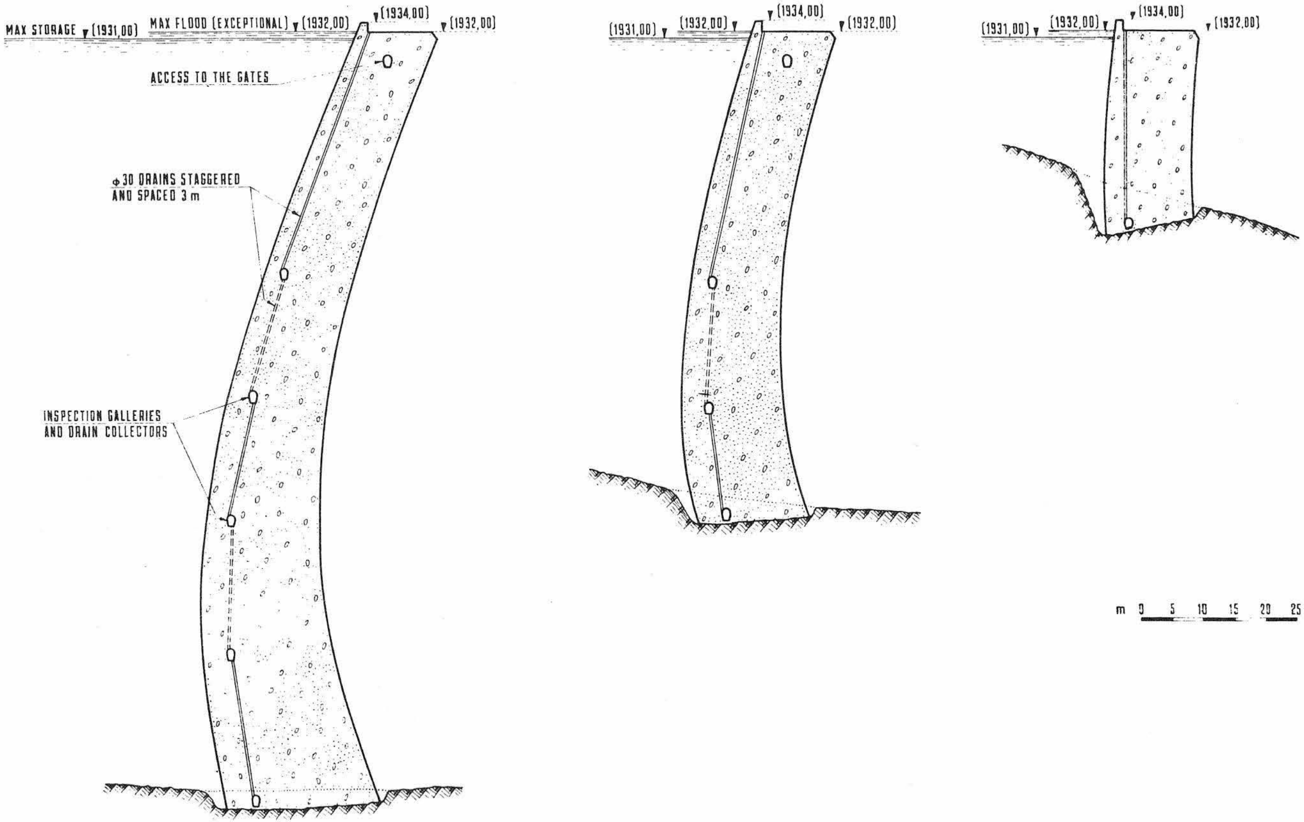
FIG. 3

SET-OUT DATA OF SECTION AT CROWN



RENO DI LEI DAM

CROSS SECTIONS



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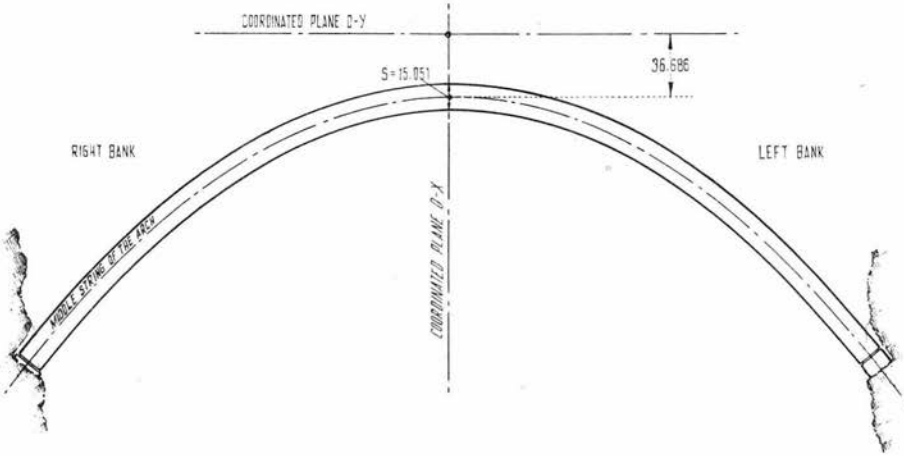
June, 1956

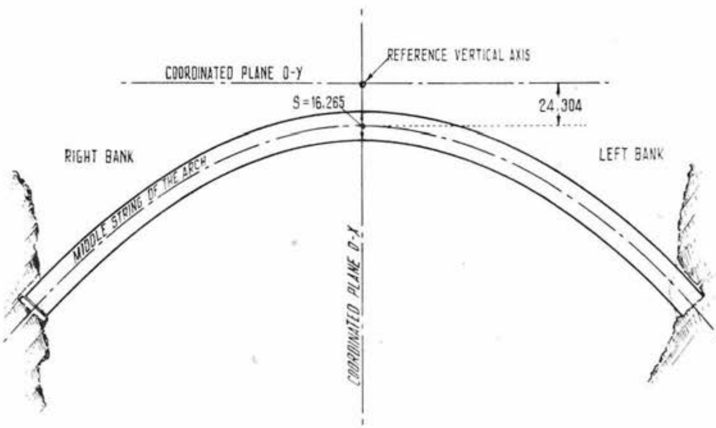
FIG. 4

FIG. 5

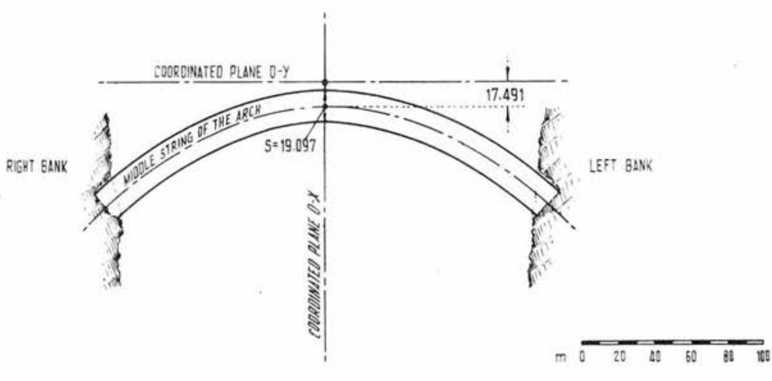
HORIZONTAL SECTION AT EL. 1923.375



HORIZONTAL SECTION AT EL. 1888.875



HORIZONTAL SECTION AT EL. 1854.375



SCHEME OF CALCULATION

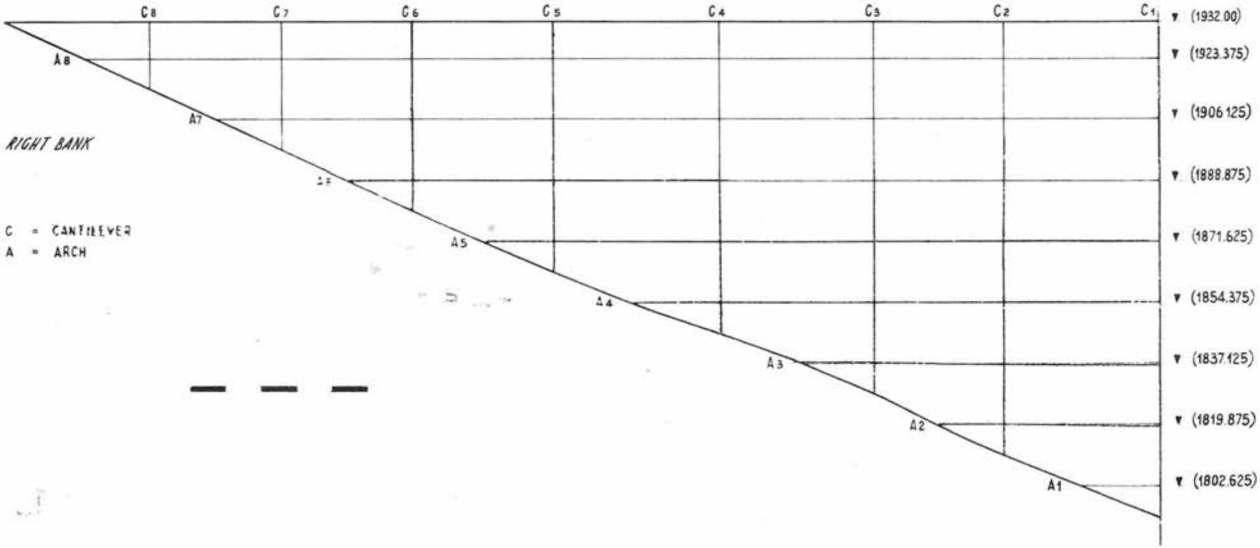


FIG. 6

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ARCH DAMS: ISOLATO DOUBLE-CURVATURE ARCH DAM

Claudio Marcello,[1] M. ASCE
(Proc. Paper 995)

FOREWORD

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1. Cons. Engr. and Technical Director, Societa Edison, Milan, Italy.

SYNOPSIS

This paper outlines the design and construction of an Italian arch dam 152 meters high and located in an extremely narrow gorge. Briefly discussed are the structural analysis of the dam—confirmed by model, construction plant, and the extensive foundation consolidation and cut-off grouting.

General

The Isolato double-curvature arch dam was constructed in the summers of the years 1952 and 1953 in order to provide a reservoir for the monthly regulation of the inflow of the basin harnessed by the Liro hydroelectric plant No. 3 in the Spluga Valley (Lepontine Alps) owned by Soc. Edison.

The dam was designed in 1950 and the corresponding model was tested from the statical point of view in the Laboratory of the Politechnic School of Milan.

The dam, shown on Fig. No. 2, has a maximum height of 132 ft, 40.50 m above foundation level and a thickness of 3.93 to 2.06 m at the crown. The upstream face has a single center, with radii variable from 18.88 to 37.46 m and center angles variable from 66.264 to 148.165 centesimal degrees, while the downstream face has three centers below El. 1243 and a single center above this elevation.

Contraction joints divide the dam into three blocks below El. 1233 and five blocks above this elevation (see Fig. No. 3). These joints were grouted in the winter following the concrete pouring and were made watertight by means of U-shaped 2 mm copper sheets embedded in the concrete.

Figs. No. 1 and 4 show the dam as seen from downstream and from upstream respectively.

Statical Analysis

The statical analysis was carried out on individual arches keyed into the abutments, normal to the centerline of the crown cross section, subject to hydrostatic pressure, dead weight, temperature changes and shrinkage of concrete.

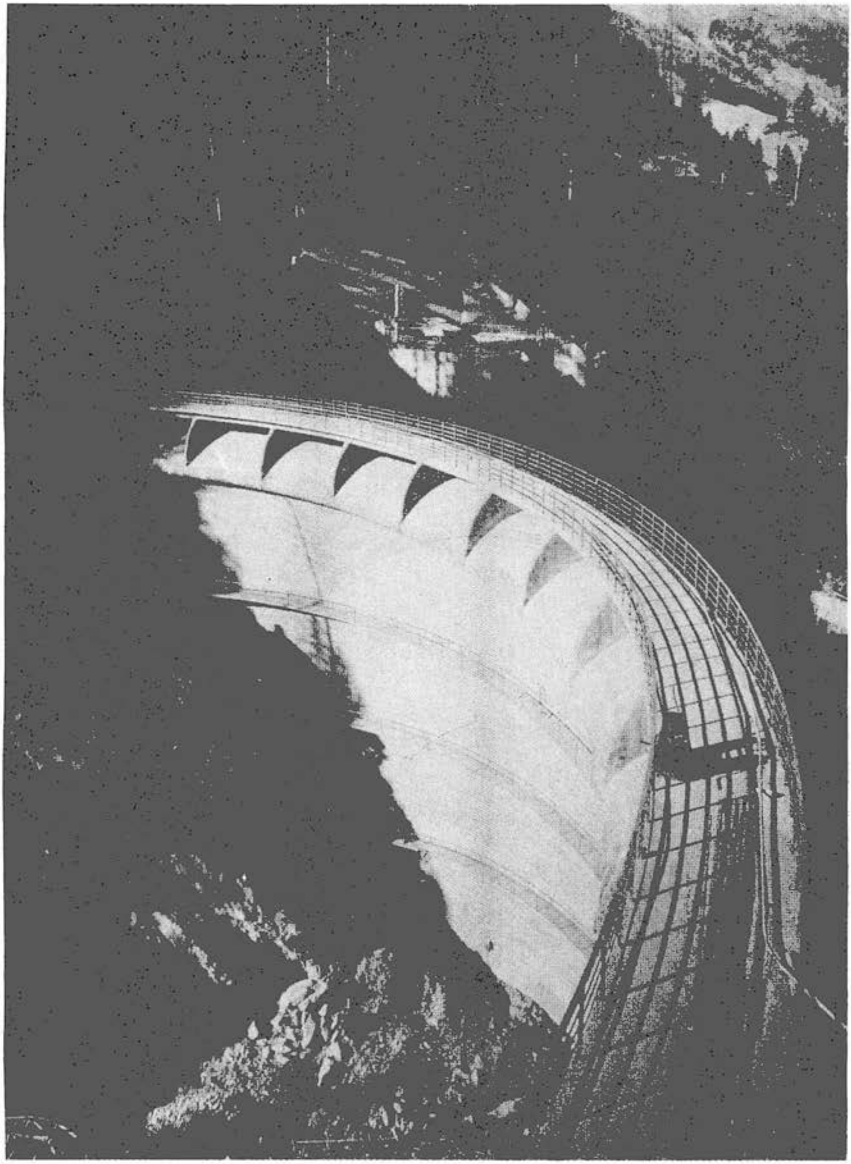
This analysis, which makes no allowance for any vertical solidarity of the dam, is justified by the comparatively small size of the gorge and consequently also of the thicknesses of the structure.

Fig. No. 5, of the cross section of the dam, shows the arches which have been checked.

The following factors have been considered:

— specific gravity of water	1.00 ton/cu.m
— specific gravity of concrete	2.50 ton/cu.m
— annual variation in temperature	$\pm$ 10° C
— daily variation in temperature	$\pm$ 6° C
— concrete shrinkage during setting made equal to a uniform decrease in temperature of	3° C

Fig. 1



View from downstream. The dam has a maximum height of 40.50 m and a thickness at crown variable from 3.93 m to 2.06 m. In the background, the spill way with automatic sector gate.

FIG. 2

GENERAL LAYOUT

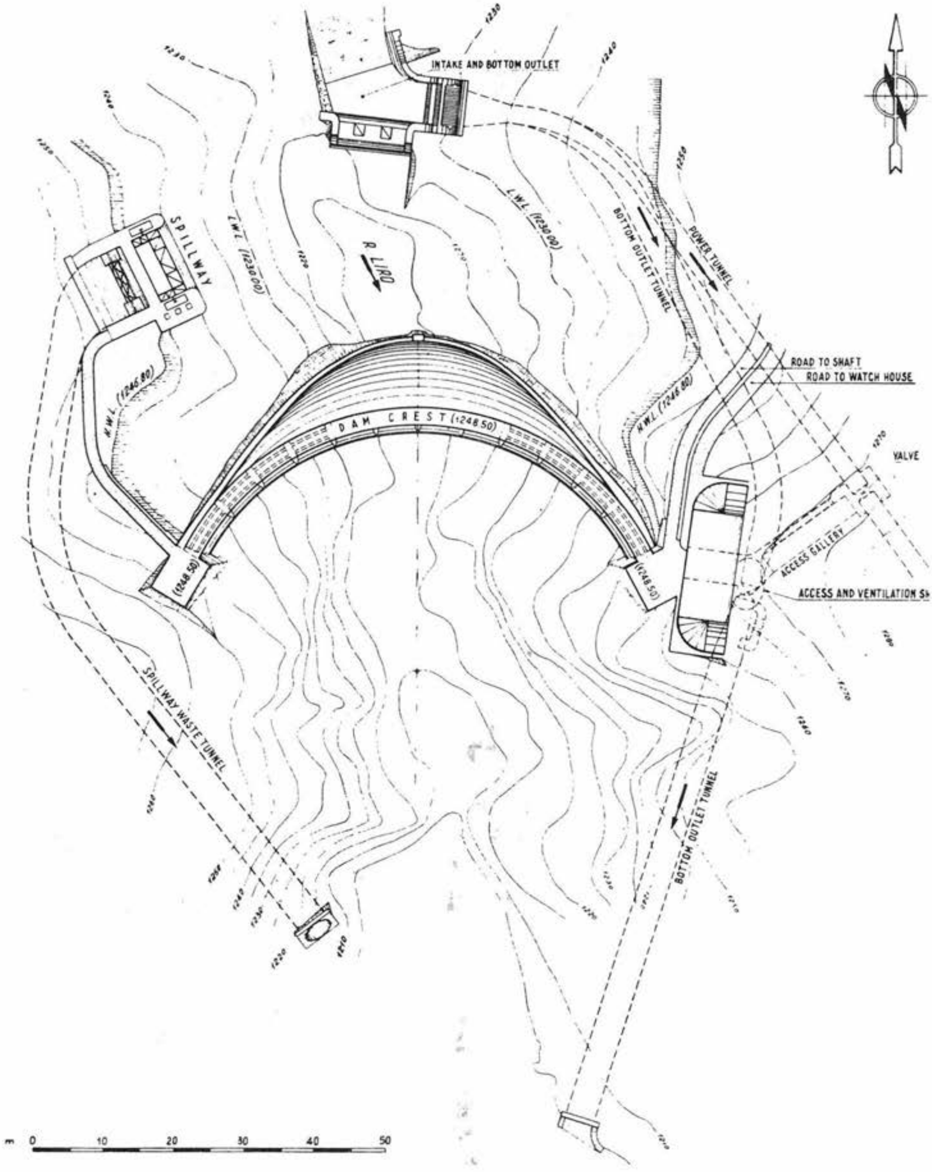
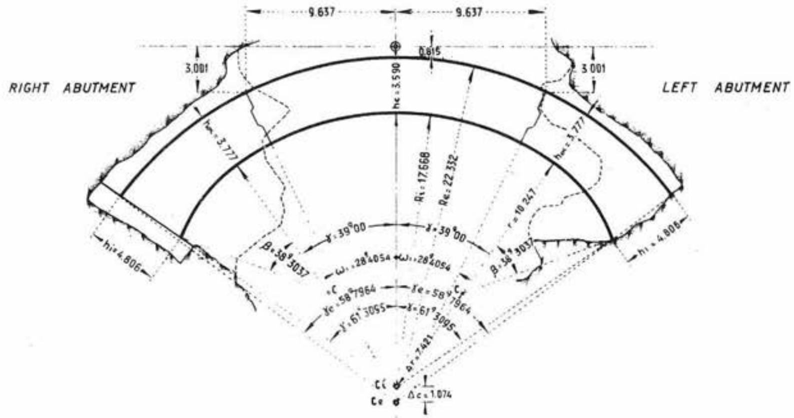


FIG. 3

HORIZONTAL SECTION AT EL. 1218.00



HORIZONTAL SECTION AT EL. 1233.00

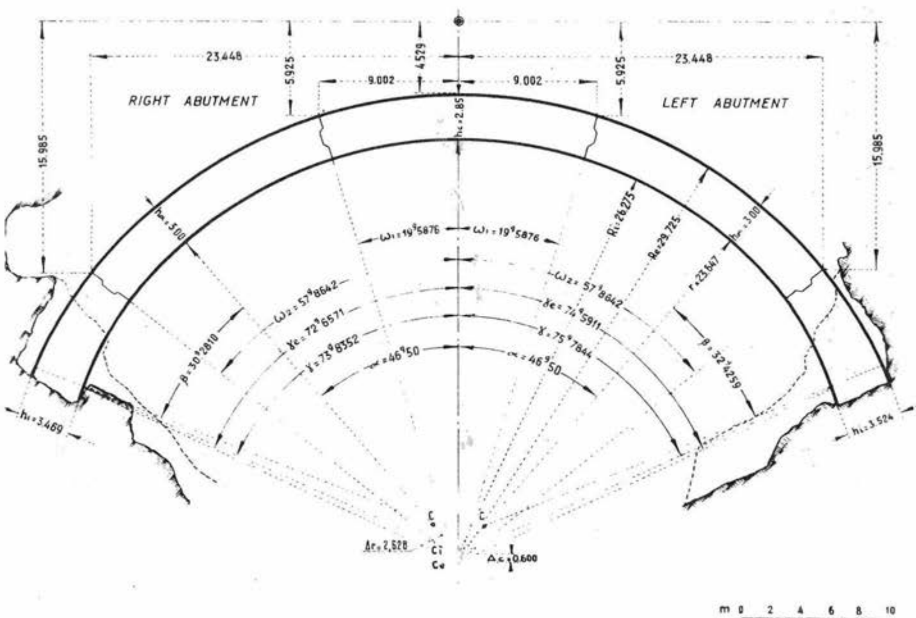
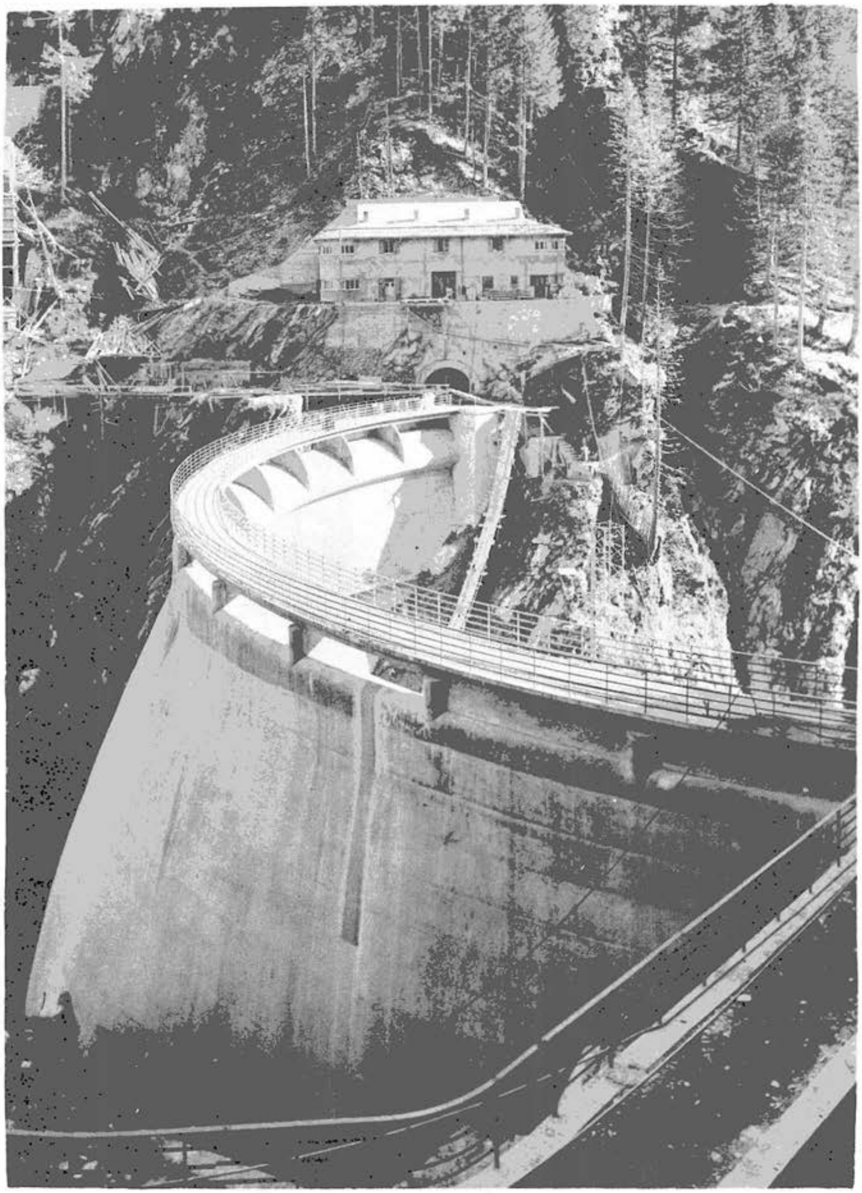


Fig. 4

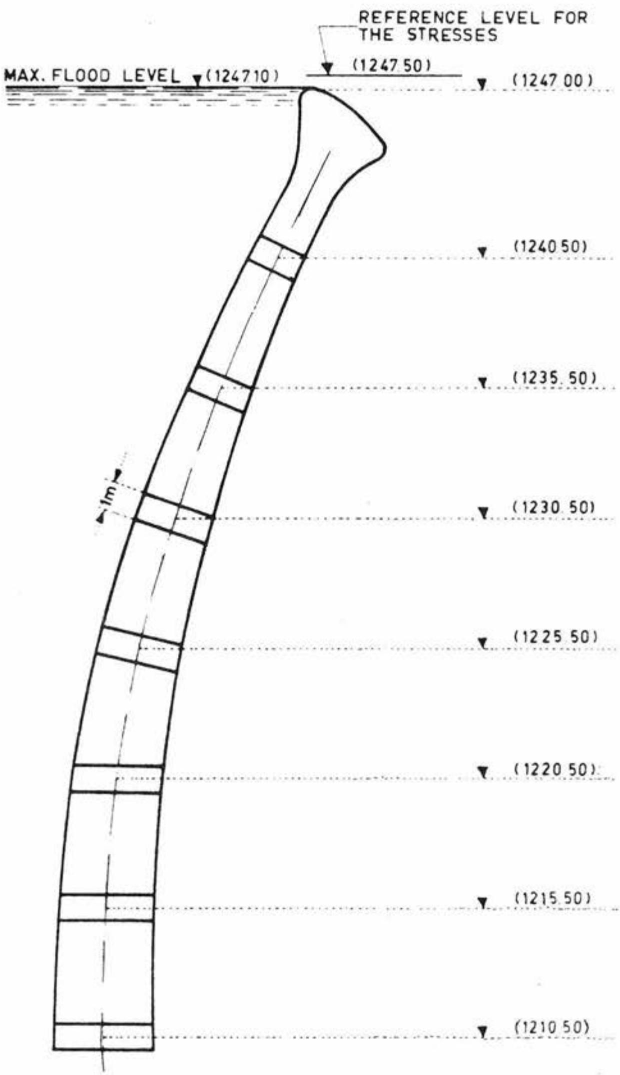


View of upstream face from the right bank. Along the dam crest, see the spans provided for allowing overflow over the dam. In the background the watchman house.

CROSS SECTION WITH THE TRACED PLANES
NORMAL TO THE MEDIUM FIBER
DETERMINING THE CHECK RINGS

SCALE 1:200

Fig. 5



ISOLATO DAM

For the stability analysis of the 2 deepest horizontal arches (at El.1210.50 and 1215.50) which were assumed as stiff arches, we have followed Résal's hypothesis.

As far as the upper arches are concerned, which have been considered as elastic independent arches, we have allowed at first approximately for a stiffening effect produced by the crest arch, ensuring the coincidence between the displacements at the crown of the crest arch and of the two underlying horizontal arches of equal thickness.

The values of the stresses calculated at the crown and in the abutments of the five elastic arches analyzed, have been shown on the Tables No. 1 and 2 here below.

The first table refers to the full reservoir, while the second refers to the empty reservoir and therefore shows only the conditions of the arches above maximum drawdown elevation.

The maximum compression, which occurs on the upstream face at the crown, does not exceed 35 kg/sq.cm; the maximum tensile stress, which occurs on the downstream face of the lowest arch among those considered elastic, is 6.62 Kg/sq.cm.

Model Tests

The 1:50 scale model of the dam was made of celluloid sheets jointed with cellulose acetate, and fastened to a bed made of liparpumice aggregate reproducing the profile of the abutments. The modulus of elasticity of celluloid was 17,500 to 18,000 Kg/sq.cm and that of the liparpumice mixture was of a similar value since the elastic characteristics of the gneis rock which forms the walls of the Isolato gorge, are such as to make advisable the adoption of an elastic modulus for the foundation rock approximately equal to the modulus of concrete.

The Poisson's modulus was instead $\nu = 0.40$ in the celluloid and $\nu = 0.15$ in the liparpumice.

The purpose of the tests was to measure the elastic deformations in the two faces of the model dam under the sole action of the hydrostatic load in the case of a maximum water level (El. 1246.80), using mercury as the loading liquid, enclosed in a rubber bag placed between the upstream face and a convenient reinforced-concrete buttress upstream of the pool.

These tests do not therefore take into account either the dead weight or the effects of temperature changes and shrinkage.

Figs. 6, 8 and 9 show the whole model as well as the installation of the measuring instruments on both upstream and downstream faces.

Deflectometers of different make were used to measure the rises in the model and the deformations have been checked by means of extensometers of the variable-electrical resistance type provided with a central measuring system. On the downstream face mechanical extensometers were used. These were placed in four directions, one horizontal, one vertical and the others two inclined at 45^0. In this way we were able to determine the stress tensor on the face in question and to make any necessary corrections in the readings.

TOTAL STRESSES ON THE INCLINED ELASTIC ARCHES WITH VARIABLE THICKNESSES AND VARIOUS TEMPERATURE DISTRIBUTIONS

TABLE No 1

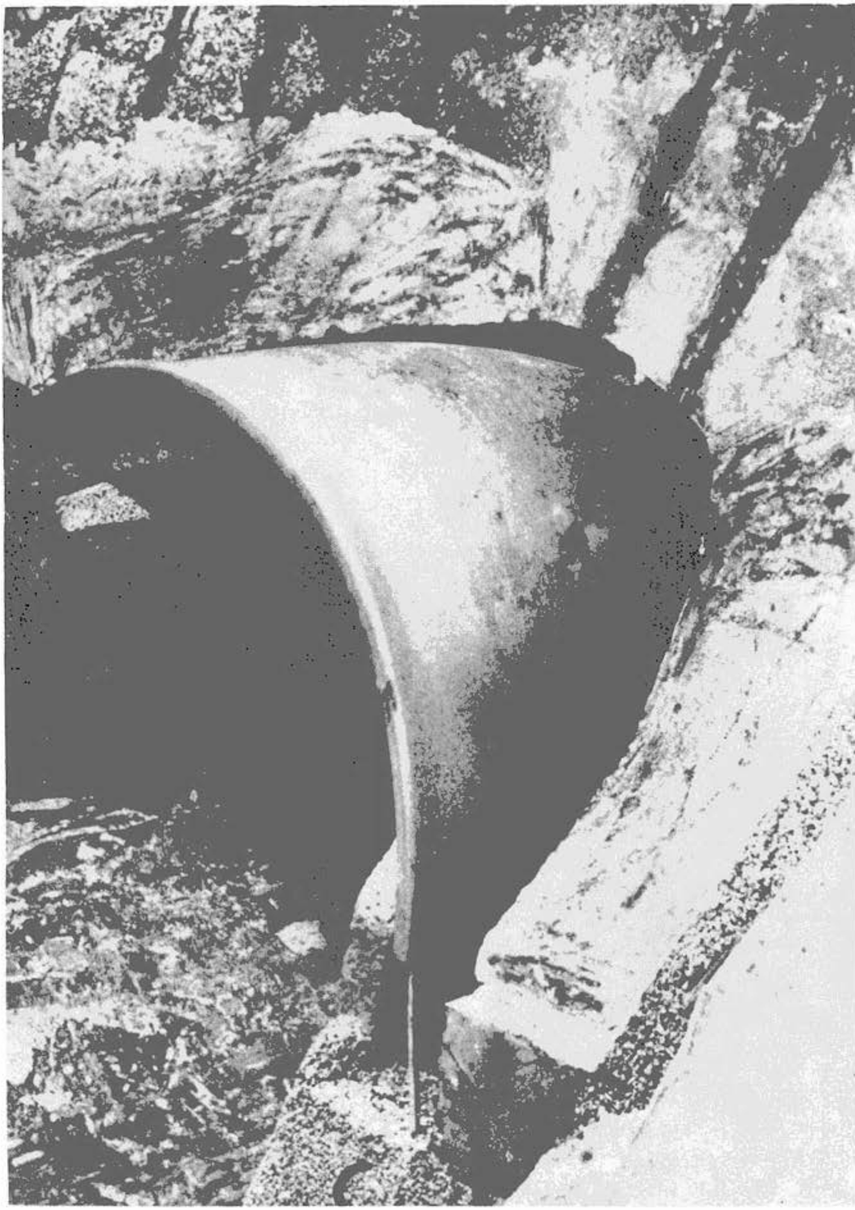
Stresses in Kg/sq.cm. Sign + means compression; Sign - means traction

	El. of arch center-line	FULL RESERVOIR							
		SUMMER				WINTER			
		Crown		Abutment		Crown		Abutment	
		Upstream	Downstream	Upstream	Downstream	Upstream	Downstream	Upstream	Downstream
without shrinkage	1220.50	+ 18.76	+ 13.34	+ 6.45	+ 18.66	+ 31.57	- 1.35	+ 4.69	+ 19.30
	1225.50	+ 15.55	+ 15.34	+ 8.15	+ 15.56	+ 32.15	- 3.00	+ 7.11	+ 15.98
	1230.50	+ 18.57	+ 12.54	+ 9.46	+ 13.80	+ 32.87	- 2.65	+ 10.74	+ 12.31
	1235.50	+ 17.71	+ 14.21	+ 10.68	+ 10.08	+ 32.64	- 1.34	+ 12.48	+ 8.13
	1240.50	+ 21.19	+ 7.32	+ 10.22	+ 9.98	+ 33.58	- 5.24	+ 13.65	+ 6.51
with shrinkage (-3°C)	1220.50	+ 20.90	+ 10.12	+ 2.59	+ 22.17	+ 33.71	- 4.57	+ 0.82	+ 23.31
	1225.50	+ 17.64	+ 12.46	+ 4.71	+ 18.71	+ 34.14	- 5.88	+ 3.67	+ 19.13
	1230.50	+ 20.18	+ 10.31	+ 6.61	+ 16.50	+ 34.48	- 4.88	+ 7.89	+ 15.00
	1235.50	+ 19.14	+ 12.38	+ 8.59	+ 12.07	+ 34.07	- 3.17	+ 10.40	+ 10.12
	1240.40	+ 22.28	+ 5.93	+ 8.22	+ 11.91	+ 34.67	- 6.62	+ 11.65	+ 8.44

TABLE No 2

	El. of arch center-line	EMPTY RESERVOIR							
		SUMMER				WINTER			
		Crown		Abutment		Crown		Abutment	
		Upstream	Downstr.	Upstream	Downstr.	Upstream	Downstr.	Upstream	Downstr.
without shrinkage	1235.50	- 4.70	+ 6.03	+ 6.87	- 6.55	+ 4.70	- 6.03	- 6.87	+ 6.55
	1240.50	- 4.14	+ 5.28	+ 6.78	- 6.53	+ 4.14	- 5.28	- 6.78	+ 6.53
with shrinkage (-3°C)	1235.50	- 3.27	+ 4.19	+ 4.78	- 4.56	+ 6.12	- 7.86	- 8.95	+ 8.54
	1240.50	- 3.05	+ 3.90	+ 4.78	- 4.60	+ 5.23	- 6.67	- 8.78	+ 8.45

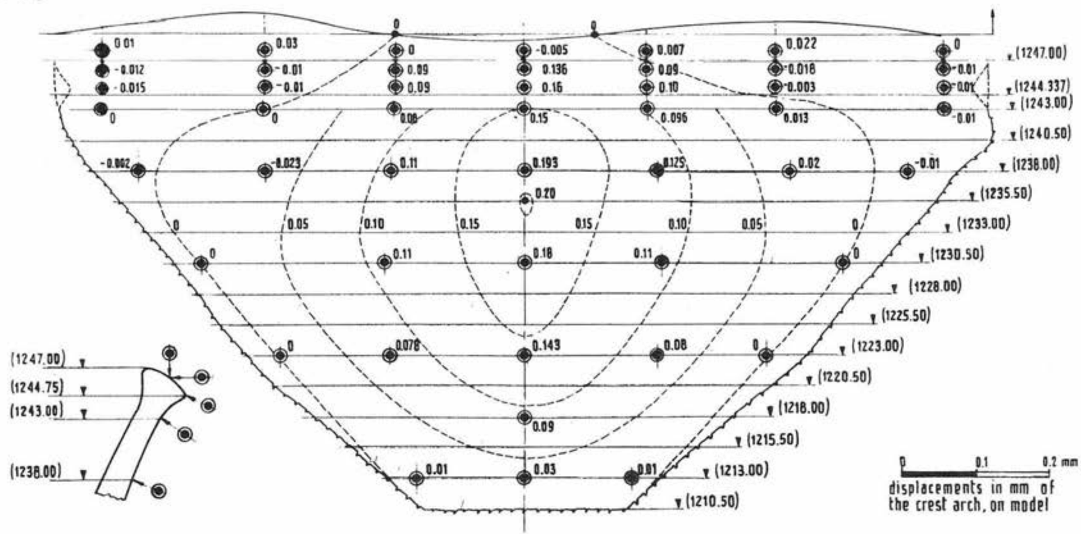
Fig.6



View of the upstream side of model constructed on the scale 1:50 with plates of celluloid jointed by means of aceto-cellulose and arranged on a proper bed reproducing the development of the abutments.

LENGTH OF DOWNSTREAM FACING

CURVES OF RADIAL DEFLECTION



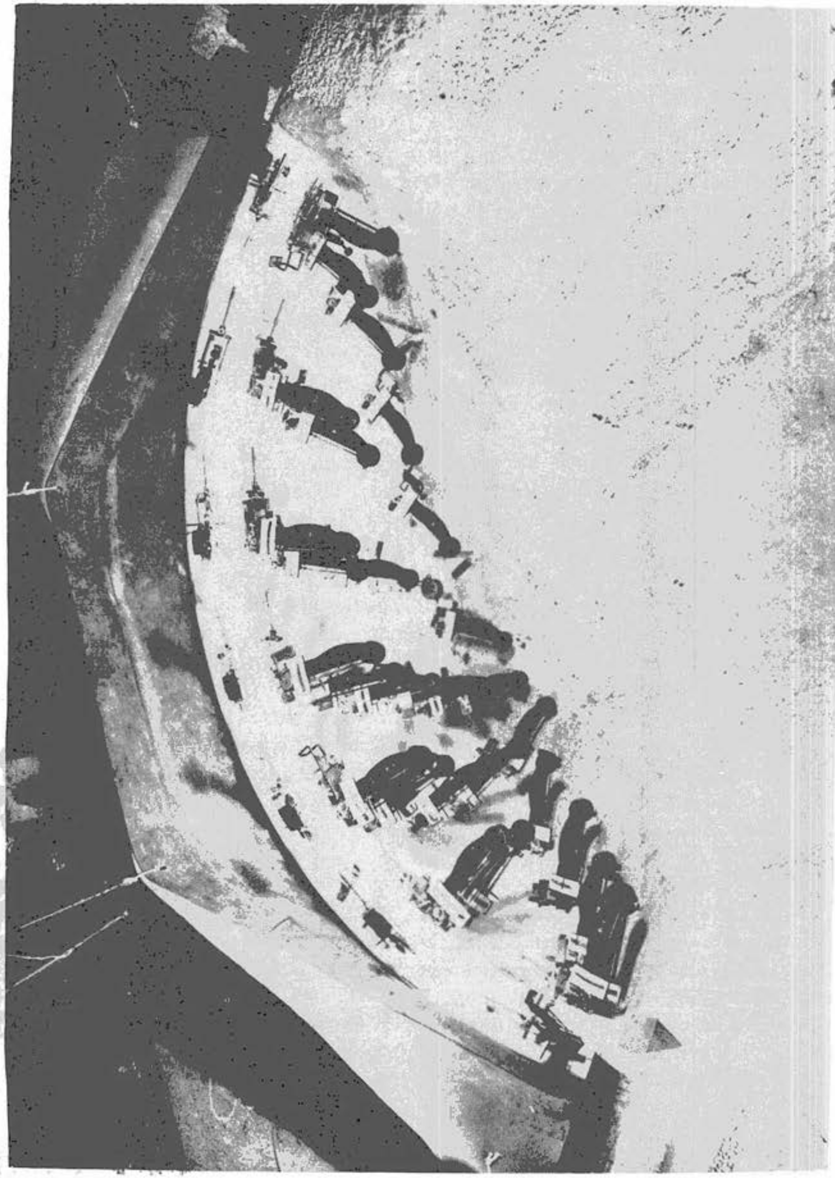
DIFFERENCE OF LEVEL 0.05 mm ON MODEL = 0.05 cm ON PROTOTYPE WITH Ec = 320.000 kg/sq.cm

FIG. 7

Fig. 8

View of the testing equipment installed on the upstream face of model.

Fig.9



View of the downstream face of model with the testing equipment arranged for measuring the deformations. See on the facing the extensometers and, upstream, the rubber bag containing the mercury used for applying the load on the structure.

The results obtained by this test are the following.

Radial deflections of the downstream face, after having drawn the necessary contour lines, show the essential symmetry of the structure, which acts like a double-curvature plate. The upper portion of the two shoulders shows the characteristics inversion of rises.

The maximum absolute value was found to be in the central part at El. 1235.50, i.e., at about 2/3 of the maximum height, and turned out to be approximately as small as 2 mm.

The horizontal stresses are compression stresses all over the upstream face and on the whole they are very small. The maximum stress is 42.7 Kg/sq.cm and was also found to be in the middle area as stated above.

By comparing this value with that of 25 Kg/sq.cm resulting from the calculation based on the assumption of independent arches, it may be seen how the double-curvature plate effect has raised the value of stresses in this area, consequently decreasing it in the peripheral zones.

Vertical stresses are always small; maximum compression (15.30 Kg/sq. cm) is found in the same area as that in which the maximum values in the arches have been recorded.

On the other hand tensile stresses occur at the bottom of the cantilever sections owing to the foundation being keyed. These tensile stresses however are not dangerous insofar as they are to be added to the compression stresses due to dead weight which have been estimated as being greater.

On the downstream face the horizontal stresses are on the whole compressive and are very weak, reaching the peak value of 20 Kg/sq.cm at the right abutment of the arch at El. 1223; tensile stresses occur only in the downstream middle area with a local peak of 5.3 Kg/sq.cm. Vertical stresses are weak and tensile in character all over a wide strip between El. 1230 and 1240 reaching El. 1215 in its lower central section; the absolute maximum of 5.1 Kg/sq.cm is found at El. 1223 while compressive stresses are present at the foundation of the cantilevers with a peak value around 12.50 Kg/sq.cm in the higher middle sections.

The stresses found on the centilevers have confirmed that these act as beams which are partially keyed at their foot and which elastically rest upon the upper arches.

Fig. 10 shows the direction and intensity of the main stresses on the downstream face. The maximum main compression stress turns out to be 22 Kg/sq.cm and the maximum tensile stress 5.1 Kg/sq.cm, at the same points as previously indicated.

The isostatic lines are very indicative because once again they confirm both the characteristic spacial behaviour of the double-curvature structure and the symmetry of the stress interplay.

Construction

The construction of the dam required particular care. As regards excavations, we had to be sure to extend them only to a sound rock level, and this in the presence of a remarkably schistous formation. Excavations had to be carried out with a minimum amount of explosives and mostly by cutting the

LENGTH OF DOWNSTREAM FACING

MAIN STRESSES AND ISOSTATIC LINES (kg/sq.cm)

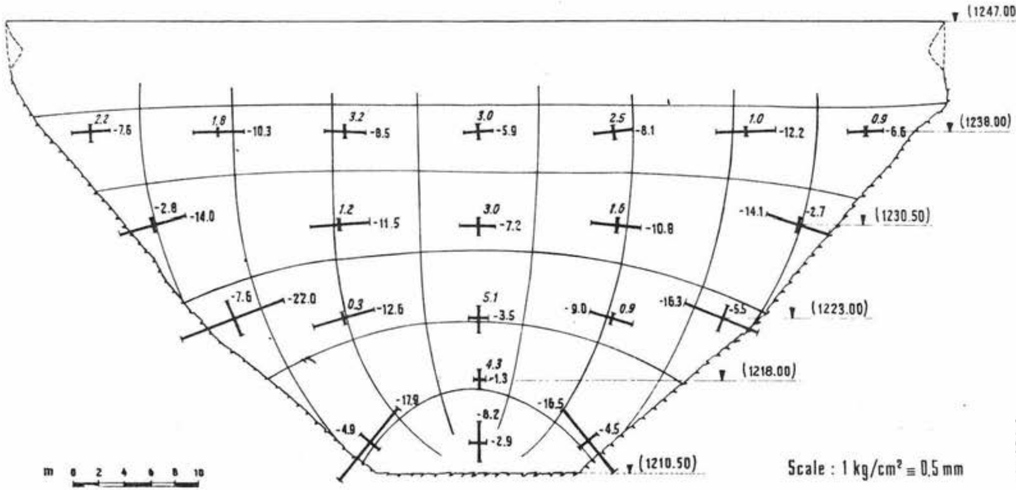


FIG. 10

rock with wedges and jackhammers, after having hammer-drilled a previous set of holes close to one another; these were not loaded.

As regards concrete pouring, care had to be taken owing to the overhanging surfaces and the double-curvature nature of the structure, which had to be made with great precision in respect to the very small thicknesses.

Concrete was made by using crushed limestone. The quarry was situated 350 m above the dam and upstream of the dam. The primary crushers were installed in the quarry. Then the materials were conveyed by continuous cableway to a secondary crushing plant and to the mixers.

The proportion of the aggregates was made by volume, using dosing machines controlled by an electro-pneumatic station.

Cement was stored in a 10 ton steel-plate silo and was proportioned by weight.

Double-cone type mixers were used, with a concrete capacity of 750 lt. The pouring was made using a 3,000 Kg. derrick made of tubular framework and provided with a 50 m boom. Blow-Knox buckets were used.

Steel forms were used, consisting of 0.50 x 0.50 m and 0.50 x 1.00 m panels, and horizontal and vertical section iron frames. These allowed a perfect reproduction of the dam faces. A 1 m high pour was made on the same block every three days; maximum daily concrete poured was 150 cu.m.

The concrete, made with 300 Kg/cu.m of 500 Kg/sq.cm cement and very rich in fine materials, resulted to be particularly compact and impervious and its strength resulted in all cases higher than 250 Kg/sq.cm after 28 days.

The waterproofing and bond grouting have not been of great importance.

Measurements on the Dam

The horizontal deflections of the dam have been recorded with a Fennel collimator. On the crest 2 adjustable collimation targets were embedded, whereas on the gorge walls 2 fixed sighting marks and 2 collimator stations were installed.

The daily readings are shown in Fig. 12.

Their values are clearly affected by temperature changes and therefore a comparison with the rises found on the model dam requires a preliminary analysis of temperature influence, now being carried out.

Furthermore a network of triangulations, referred to 7 triangulation bases, allows the measurement of the deflections of the dam by means of 18 triangulation targets installed on the downstream face.

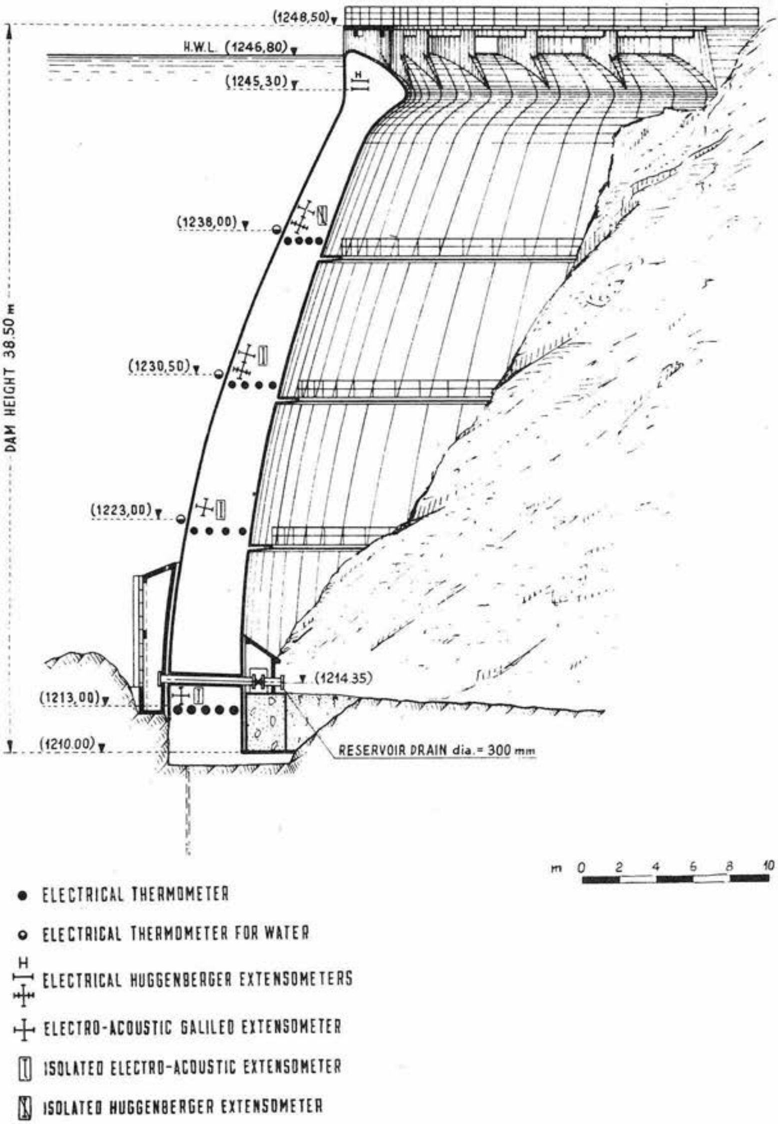
The corresponding measurements are taken monthly.

During concrete placing, electrical temperature detectors have been embedded in the dam, to measure the concrete and water temperatures. Also electroacustic extensometer ranged in vertical and horizontal directions as well as at inclinations of 45^0, for a complete determination of the stress tensor work.

Electroacustic extensometers isolated from the structure record the deformations due to temperature and humidity.

Fig. 11 shows the instrument embedded in the dam along the crown section.

POSITION OF MEASURING INSTRUMENTS FIG. 11



CHECK MEASUREMENTS ON THE BEHAVIOUR OF THE DAM

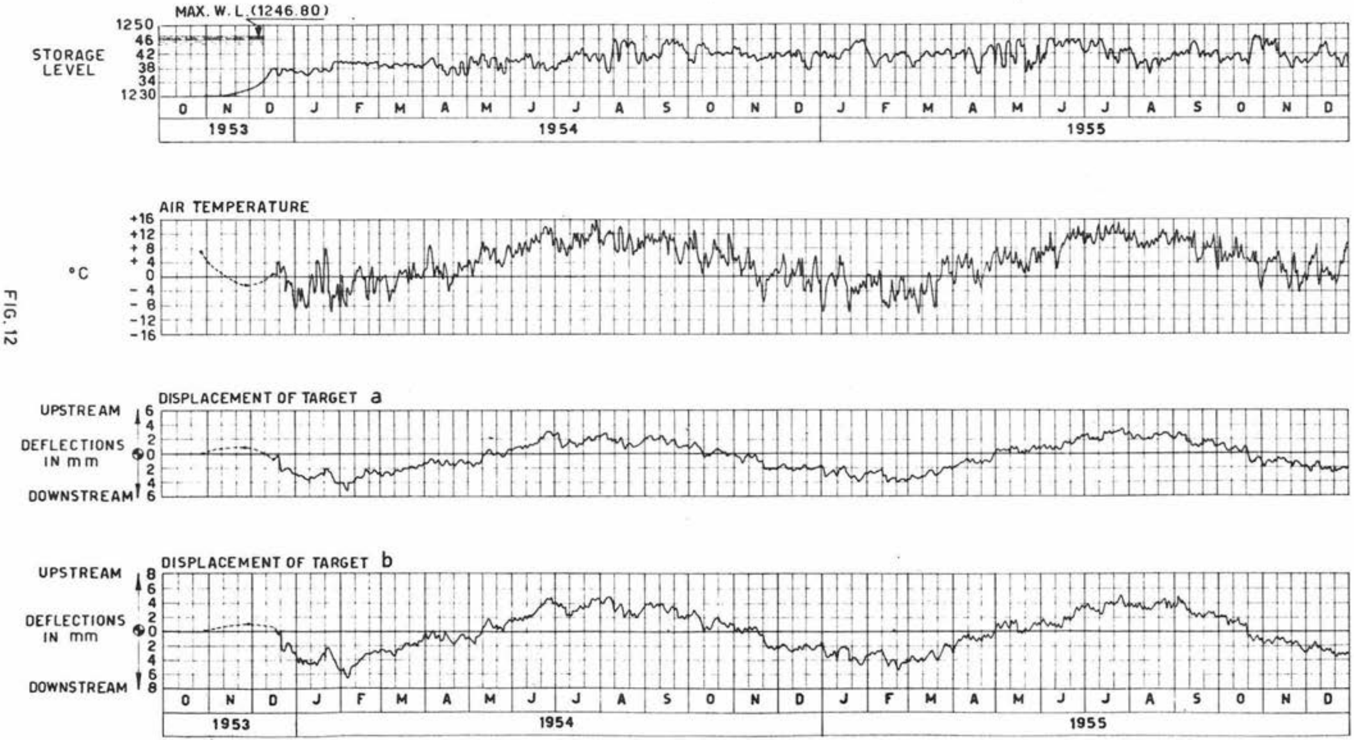


FIG. 12

A set of marks have been installed on the downstream face in order to measure the deformations along 4 directions at 45°, by means of a removable extensometer.

The dam design and the work direction were carried out by the Hydroelectric Plant Construction Department of Soc. Edison of Milan, which is directed by the writer.

The construction was entrusted to the contractor S.A.L.C.I. of Milan.

Journal of the

POWER DIVISION

Proceedings of the American Society of Civil Engineers

ARCH DAMS: RIO FREDDO DAM WITH GRAVITY ABUTMENTS AND CUT-OFFS

Claudio Marcello,[1] M. ASCE

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams held in April, 1939, much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be included in the Symposium. So far, nine papers have been approved: "Arch Dams: Their Philosophy" (Proc. Paper 959) by Andre Coyne; "Arch Dams: Trial Load Studies for Hungry Horse Dam" (Proc. Paper 960) by R. E. Glover and Merlin D. Copen; "Arch Dams: Portuguese Experience with Overflow Arch Dams" (Proc. Paper 990) by A. C. Xerez; "Arch Dams: Theory, Methods, and Details of Joint Grouting" (Proc. Paper 991) by A. Warren Simonds; "Arch Dams: Santa Giustina Single-Curvature Arch Dam" (Proc. Paper 992) by Claudio Marcello; "Arch Dams: Measurements and Studies on Santa Giustina Dam" (Proc. Paper 993) by Claudio Marcello; "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam" (Proc. Paper 994) by Claudio Marcello; "Arch Dams: Isolato Double-Curvature Arch Dam" (Proc. Paper 995) by Claudio Marcello; and "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs" (Proc. Paper 996) by Claudio Marcello.

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

Note: Discussion open until November 1, 1956. Paper 996 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 3, June, 1956.

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SYNOPSIS

This paper outlines the design and construction of an Italian double-curvature arch dam with gravity abutments and lateral cut-off wings. Briefly discussed are the theoretical stress analysis, structural model testing and construction plant. Described is the instrumentation for determining the actual behavior of the dam.

The dam was constructed in the years 1954-55 with the purpose of creating a reservoir on the Rio Freddo, in the Maritime Alps, having a net storage capacity of 325,000 cubic meters, to be used for daily and weekly regulation of the energy derived from the water of the Stura di Demonte basin in the Vinadio power station, which is owned by Soc. Edison.

Owing to the particular morphological structure of the valley at the site selected for the dam, double curvature thin arch structure was adopted, abutting directly on the rock up to a height corresponding to El. 1177 and on two concrete gravity thrust blocks for the upper portion, up to El. 1206 corresponding to crest level.

The portions of the valley section left free by the arch dam were filled in on the right bank by means of a solid gravity straight-axis wing and on the left bank by a spillway structure (Fig. 1).

The vault, i.e., the central arched part of the dam, has a maximum height of 40.50 m and thickness varying from 1.40 at the top to 5 m at the bottom. The horizontal arches are circular with single center for both intrados and extrados; consequently, thickness is constant from crown to abutments.

The length at the crest is of 120.90 m, and the chord corresponding to the extrados at the crest is about 100 m.

The principal geometrical data of the vault are:

— Vertical curvature radii of the crown section centerline	$59.21 \div 118.42$ m
— Horizontal extrados radii	$42.52 \div 58.28$ m
— Horizontal intrados radii	$38.53 \div 55.78$ m
— Angle measurements	$40.00 \div 132.06$ centesimal degrees
— Volume of the vault	10,000 m^3

Both facings of the vault are in concrete and steel forms were used for pouring. No special plaster was applied and the facings were provided with a light horizontal and vertical stress spreading reinforcement (Fig. 2).

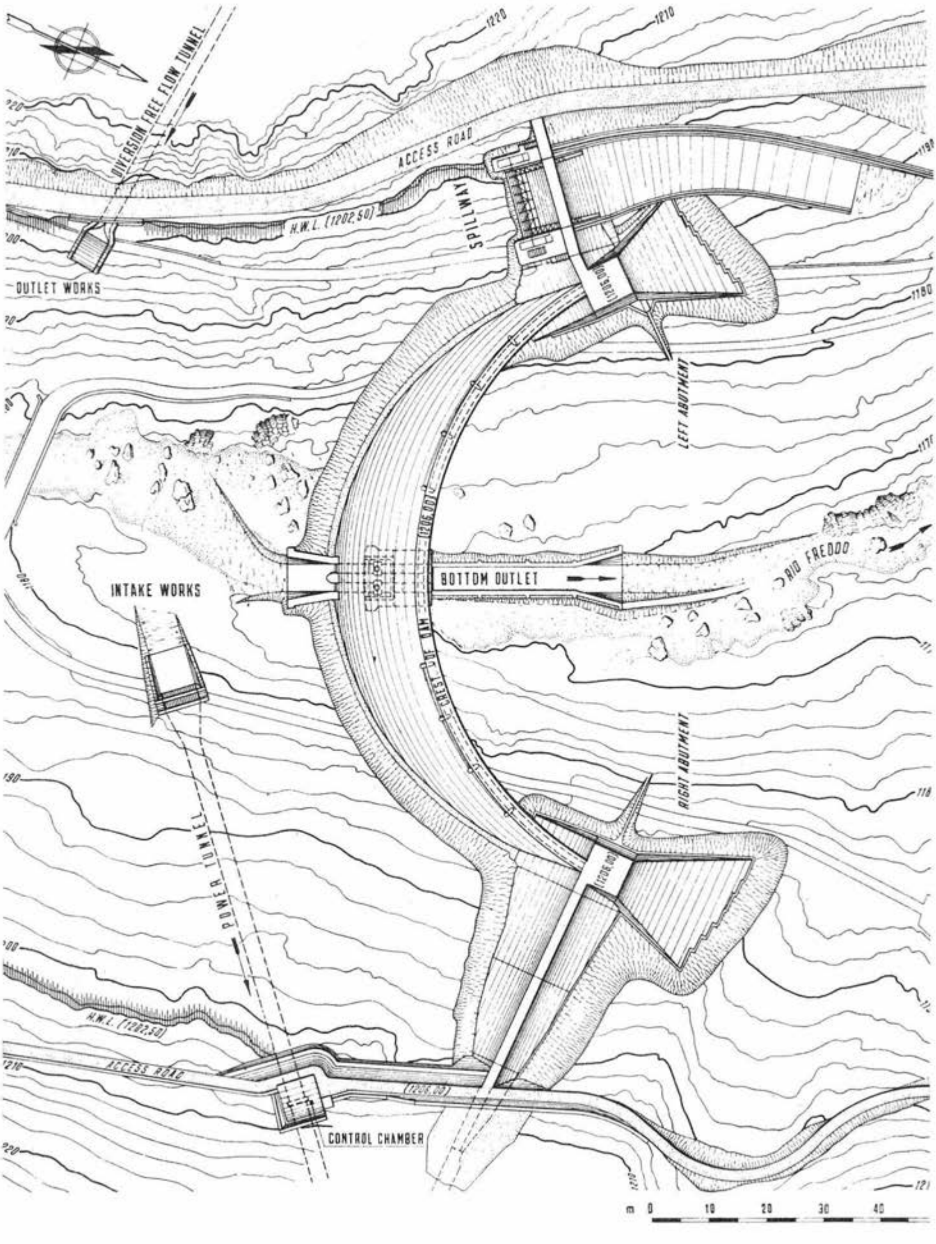
During construction, the vault was divided into 9 sections by means of 8 temporary vertical joints which were cement-grouted after shrinkage. Grout leakage is prevented by use of U-shaped 2 mm thick copper strips keyed on both sides into the concrete near the upstream and downstream facings along the joint.

The dam crest is established at El. 1204, which is the normal max. flood level; the max. exceptional flood level, which is expected to reach El. 1205, would cause the dam to overflow with a head of 1 m.

The exceptional overflow of the dam is allowed because of the considerable inclination of the vault towards downstream (about 9 m at the crown section)

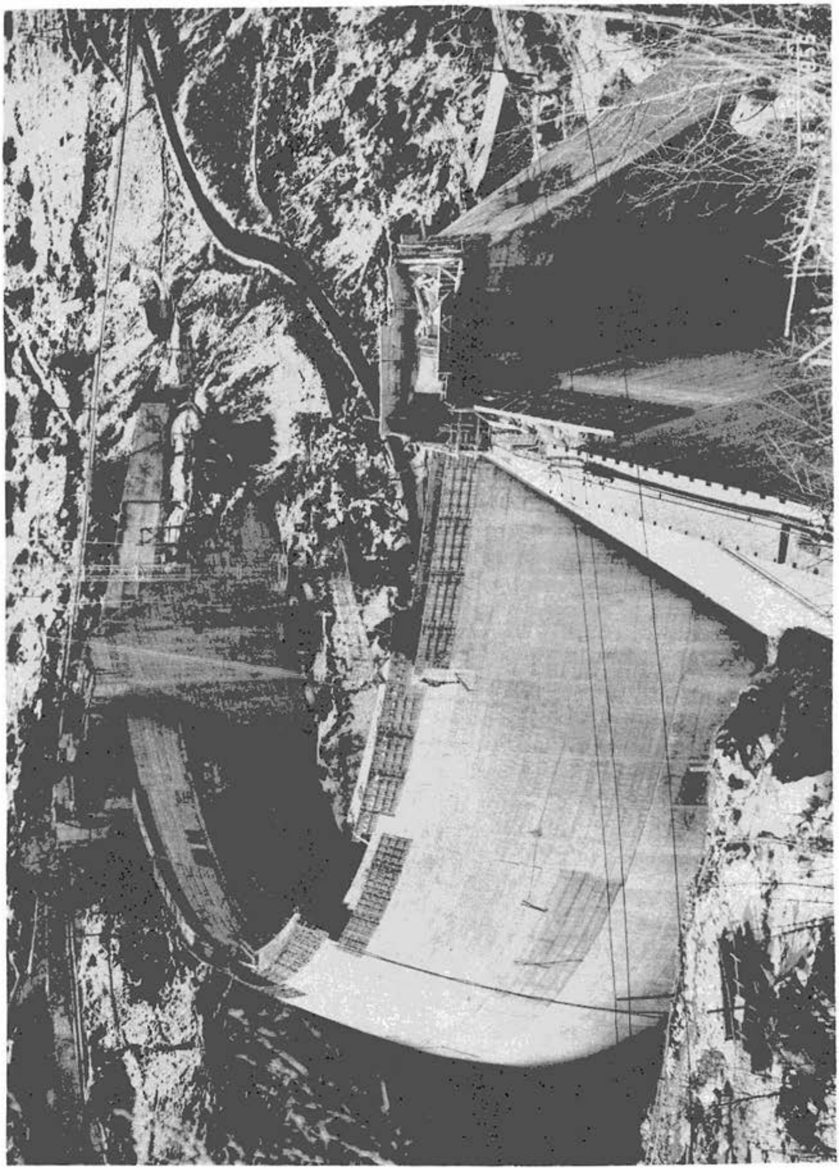
FIG. 1

GENERAL LAYOUT



RIO FREDDO DAM

FI



View from the right bank. This dam is a concrete double-curvature arch-dam with abutments and lateral gravity sections in concrete. The maximum height of the arch if 40.50 m.

and of the peculiar shape of the cross-section of the crest arch, both of which features contribute to the discharge of the overflowing water at a sufficient distance from the foot of the dam (Fig. 3).

On the downstream face 4 horizontal inspection footbridges are arranged at El. 1196, 1187, 1178 and 1174 respectively.

The solid gravity shoulders, onto which the top portion of the vault is abutted, have a mean height of 27 m; the slope of the upstream face is 0.535, the slope of the downstream face 0.70 and the mean thickness 7.80 m.

Special care has been taken in designing the shoulders, with the purpose of ensuring at the same time a suitable abutment for the vault and good stability conditions in the shoulders themselves. The orientation of the axis of the shoulder was chosen to make the mean stress plane coincide as much as possible with the symmetrical plane of the shoulder.

The volume of each shoulder block is about 6,000 m^3.

Statical Analysis for the Vault and Shoulders

a) Vault

The vault has been considered as consisting of an assembly of independent arches, keyed at the abutments; these arches are subject to hydrostatic pressure, dead-weight and changes in temperature.

The effect on resistance of vertical elements the (cantilevers) was therefore not taken into account.

In order to make an imaginary subdivision of the vault for purposes of design, based on the results of recent experience carried out on models of arch dams, it was assumed that the isostatic lines of the structure lie on slanting planes instead of on horizontal ones which show an increase in dip with a corresponding decrease in elevation. Consequently the unitary eight arches which were being analyzed, are the ones resulting from the intersection of the intrados and extrados surfaces of the dam with planes belonging to a group coming from a horizontal axis normal to the plane of symmetry of the vault (Fig. 6).

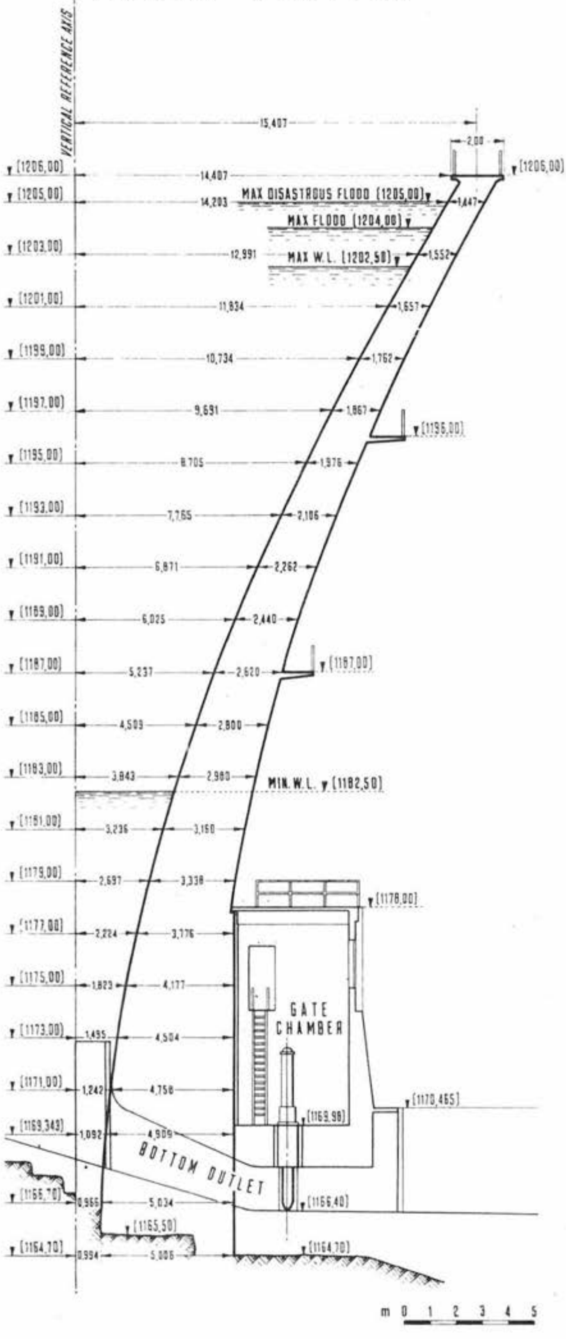
Owing to the considerable fineness of the upper portion of the vault, it was assumed that the related isostatic lines lie in horizontal planes; consequently the arches between El. 1206 and 1202 are limited by horizontal planes; the horizontal axis which supports the inclined planes determining lower arches was set at El. 1202.

The minimum horizontal distance D between the horizontal axis supporting the inclined planes and the top of the crown centerline section was determined by the conditions necessary to the equilibrium of the elemental arches in every point of this section, each one of the arches being subject to hydrostatic load, to the forces transmitted by the sections above and below each element, as well as to its deadweight.

For the sole calculation of this distance D the upstream face of the dam was assumed cylindrical with a vertical axis, thus leaving out the effect of the weight of water pressing vertically on the face. This is a conservative assumption for the calculation of the thrusts transmitted by the arches onto the shoulders, since that weight of water would actually give the isostatic lines a greater inclination.

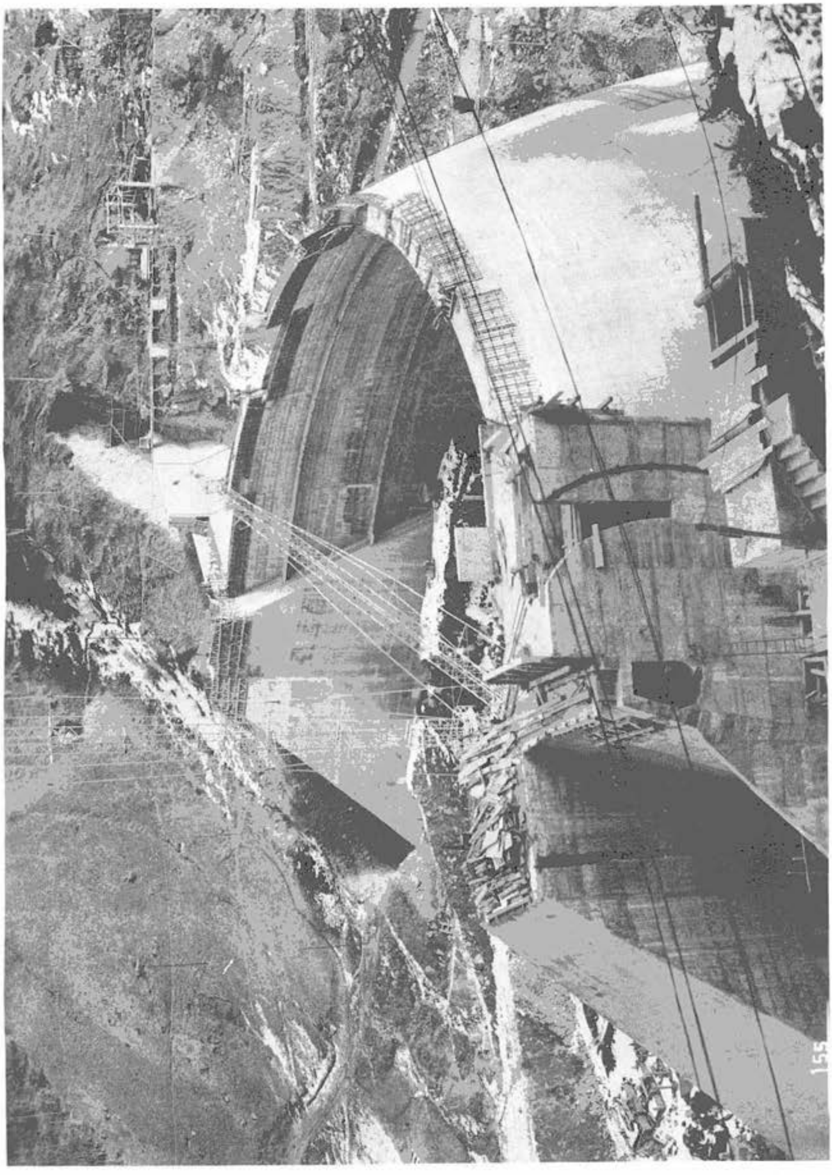
CROSS SECTION

FIG. 3



FREDDO DAM

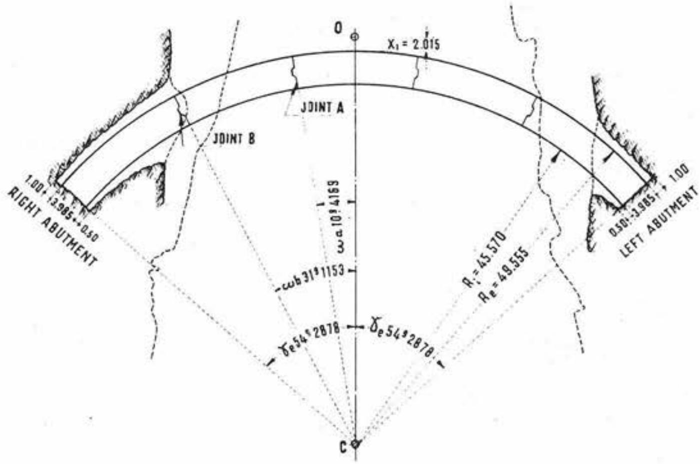
FIG.4



View from the left bank. In foreground see the gravity section with the spillway. The height of the abutments is about 27 m.

FIG. 5

HORIZONTAL SECTION AT EL. 1176.00



HORIZONTAL SECTION AT EL. 1188.00

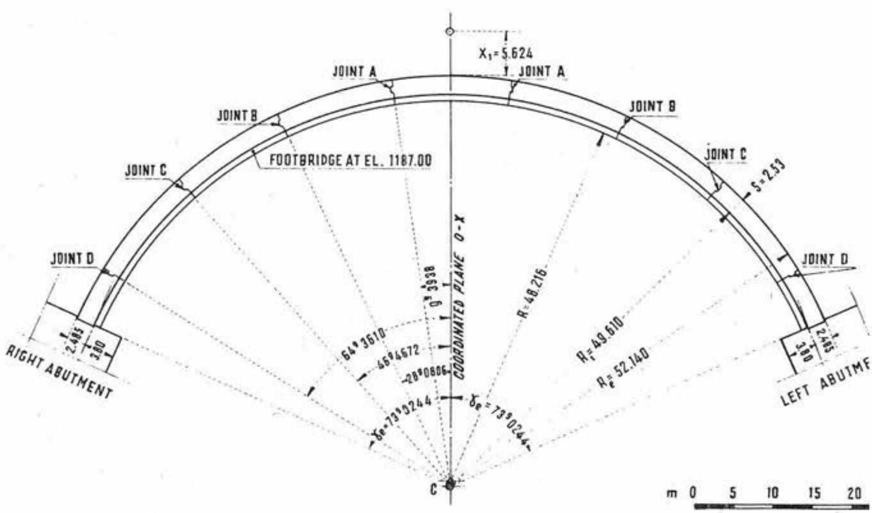
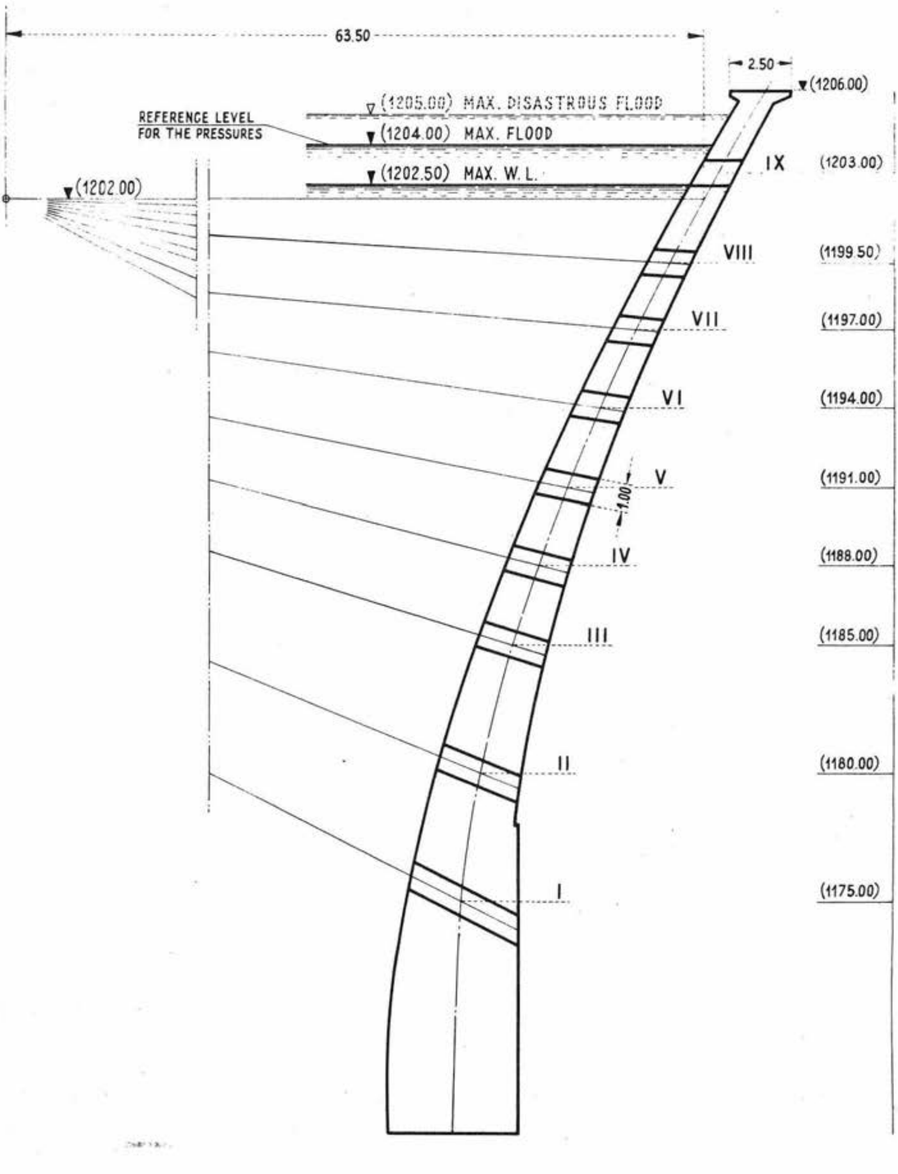


FIG. 6

SECTION AT THE CROWN WITH THE LINES OF THE PLANES DETERMINING THE CHECK ARCHES

1 : 200



Nine arches were analyzed, 8 of which dip; the one at El. 1203 is horizontal.

These arches were assumed to be subject to the entire hydrostatic pressure acting on each one of their points; this pressure therefore increases from crown to abutments for the arches below El. 1202.

The stability analysis of the nine arches was carried out in agreement with the principles of the theory of elasticity and with the assumptions that keying at the abutments was perfect and that the concrete only was resistant. Further suitable simplifying assumptions were introduced.

The stress factors assumed in the calculation are:

– Specific gravity of water	1 t/m^3
– Specific gravity of concrete	2.5 t/m^3
– Yearly temperature variation	$\pm$ 10° C
– Daily temperature variation	$\pm$ 6° C
– Shrinkage, made equal to a temperature decrease of	3° C

The results of the stability analysis are summarized in the annexed table.

Assuming that the arches are subject only to maximum hydrostatic pressure and to deadweight, compressive stresses reach a value of 37 Kg/cm^2.

On the other hand, assuming simultaneous action of maximum hydrostatic pressure, deadweight, temperature fluctuation and shrinkage at their assumed maximum values, compressive stresses amount to 71.7 Kg/cm^2 and tensile stresses to 5 Kg/cm^2.

In case of empty reservoir, when the arches are subject only to yearly and daily temperature fluctuations at their maximum values, and to shrinkage, compressive stresses reach a value of 4.7 Kg/cm^2 and tensile stresses that of 4.9 Kg/cm^2.

b) Shoulders

The stability analysis for the shoulders was carried out on the basis of the 7 spatial vectors resulting from the analysis of the arches. These vectors represent the thrust transmitted by the arches to well defined points of the upstream facing of the shoulders. Since the transverse components of the said vectors and the distance of their point of application from the centerline of the shoulder are small, the system of spatial forces was reduced to a system of forces all acting in the same vertical plane.

The tests carried out have shown that the curve of pressures lies always inside the core of the section and consequently there are no vertical tensile stresses.

In calculating the principal stresses on the upstream facing, where the vault is abutted, the isostatic were assumed to be normal to the contact surface at this point and therefore the main compression and tension directions were, respectively, normal and parallel to the facing, as is the case for gravity dams. The pressure diagram of the unitary thrust transmitted by the arch was then assumed to be rectangular.

The principal tensile stresses resulting from these analysis have turned out to be from 2 to 6 Kg/cm^2. The effect of displacement of the middle vertical stress plane in regard to the middle plane of the shoulder was also taken

RIO FREDDO DAM

TABLE OF STRESSES

Kg/ sq.cm

Elevation of arch center-line at crown	FULL RESERVOIR												EMPTY RESERVOIR							
	Hydrostatic pressure				Hydrostatic pressure + temp.fluctuation + shrink.								Temperature fluctuation + shrinkage							
					Summer				Winter				Summer				Winter			
	Crown		Abutment		Crown		Abutment		Crown		Abutment		Crown		Abutment		Crown		Abutment	
	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream
1175.00	44.95	8.51	8.37	36.93	43.27	8.88	3.50	41.43	53.83	-5.00	-0.22	44.41	–	–	–	–	–	–	–	–
1180.00	64.40	8.79	15.92	30.26	60.24	12.68	12.36	33.85	71.72	0.56	14.02	31.99	–	–	–	–	–	–	–	–
1185.00	34.94	44.53	23.90	37.17	29.45	49.96	20.15	40.90	41.91	37.42	25.35	35.68	-1.67	1.89	2.56	-2.49	3.15	-3.57	-4.86	4.73
1188.00	52.13	22.17	25.11	32.50	46.74	27.53	21.44	36.16	58.84	15.35	26.74	30.84	-1.51	1.70	2.36	-2.31	2.83	-3.16	-4.40	4.31
1191.00	50.65	21.16	25.18	31.35	45.62	26.15	21.62	34.89	56.78	14.95	26.96	29.55	-1.33	1.45	2.12	-2.09	2.43	-2.67	-3.90	3.83
1194.00	36.06	24.17	18.18	33.49	31.14	29.09	14.72	35.94	41.94	18.23	19.98	31.66	-1.17	1.27	2.02	-1.99	2.13	-2.29	-3.68	3.61
1197.00	20.43	23.99	11.54	28.61	15.67	28.76	8.34	31.81	26.03	18.32	13.32	26.81	-1.04	1.11	1.76	-1.74	1.88	-2.01	-3.18	3.14
1199.50	14.89	12.92	11.35	16.50	10.62	17.21	8.32	19.54	19.72	8.03	12.88	14.94	-0.69	0.74	1.90	-1.88	1.25	-1.34	-3.40	3.36
1203.00	3.94	3.44	3.08	4.21	-0.09	8.10	0.30	6.99	8.63	-1.90	4.60	2.67	-0.83	0.88	1.62	-1.60	1.49	-1.56	-2.88	2.84

+ compressive stress

- tensile stress

into consideration and the ensuing bending and twisting moments were also calculated. The relative stresses were found to be very small with maximum values of 0.3 and 1.25 Kg/cm^2 respectively.

Model Testing

To sustain and complete the calculation, tests were carried out at the I.S.M.E.S. institute of Bergamo on a 1:50 scale model made of a pumice cement mixture (Fig. 8).

The effect of deadweight in the shoulders was reproduced by means of steel tension cable. Hydrostatic pressure was obtained by means of hydraulic winches with suitable load spreaders.

Elastic tests and ultimate strength tests were made on the model and they showed that the dam acts as a double-curvature plate.

The elastic deformations measured (under a load about 4 times the maximum hydrostatic head) showed medium horizontal compressive stresses. The maximum values were 28.5 Kg/cm^2 at the upstream facing and 31 Kg/cm^2 at the downstream facing.

The vertical stresses at the upstream face, except for limited areas near the abutments, where small tensile stresses appear, were always compressive stresses.

At the downstream facing the stresses were on the whole tensile ones with a maximum of about 15 Kg/cm^2 in the same region where maximum horizontal compressive stresses also appear. However, if the effect of deadweight had been taken into consideration, these stresses would have been smaller.

The ultimate strength tests showed a practically linear elastic behaviour up to about 7 times the maximum hydrostatic head, while the first breaks on the downstream side were observed with a load of only 5.7 times the maximum one.

Load tests were then carried out as far as the test installation allowed (the load was almost 10 times the maximum hydrostatic head) and under this condition too the elasticity of the model was almost perfect.

The tests carried out on the shoulders showed very moderate stresses.

For the left shoulder, a maximum compression amounting to about 10 kg/cm^2 was recorded in the lower larger part, while the stresses at the upstream facing amounted to 2 kg/cm^2 on the inside (wet side) and 5 kg/cm^2 on the outside (dry side).

For the right shoulder the maximum stresses were 7.5 kg/cm^2 in the lower larger part and on the upstream facing approximately zero on the inside and 3 kg/cm^2 on the outside.

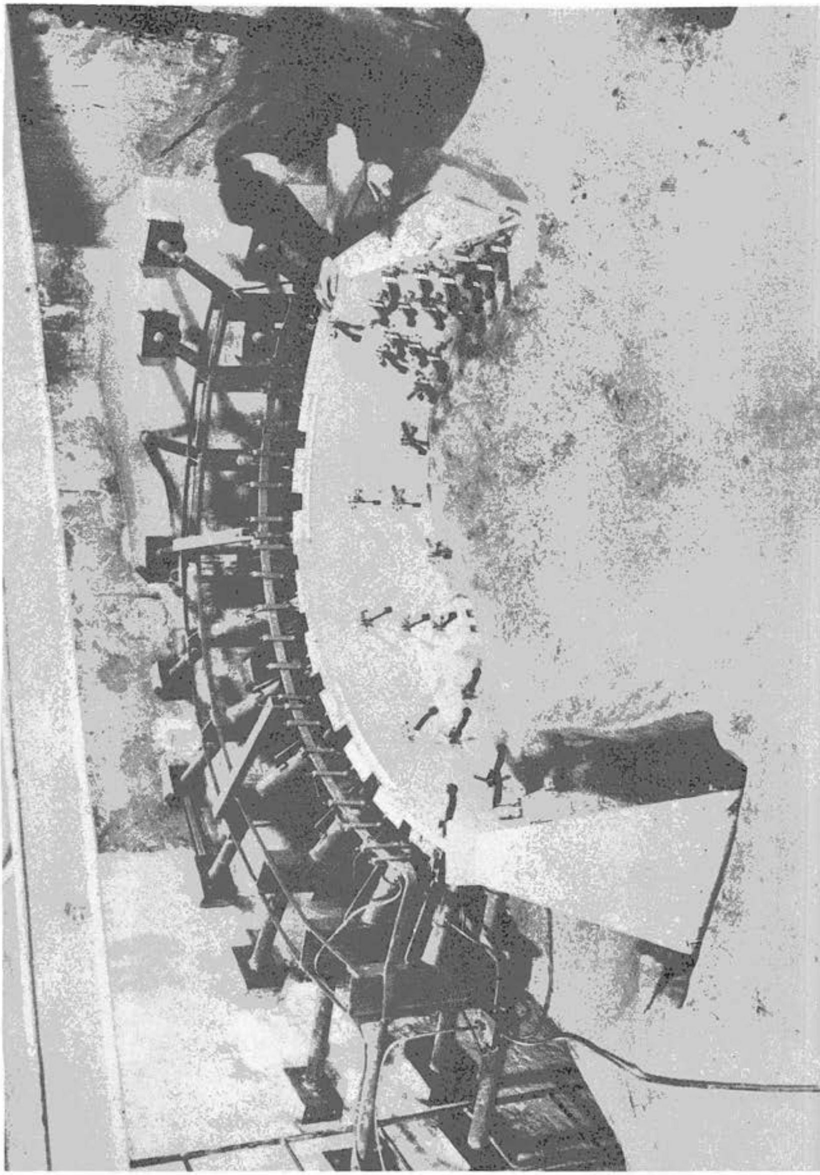
The isostatic lines plotted coincided sufficiently closely with those previously assumed as may be seen in Fig. 8.

Construction

The foundation excavations for the arch dam, shoulders and gravity wings were driven to a sufficient depth to make sure that the whole structure was founded on sound rock. Non-disruptive explosives were used and the

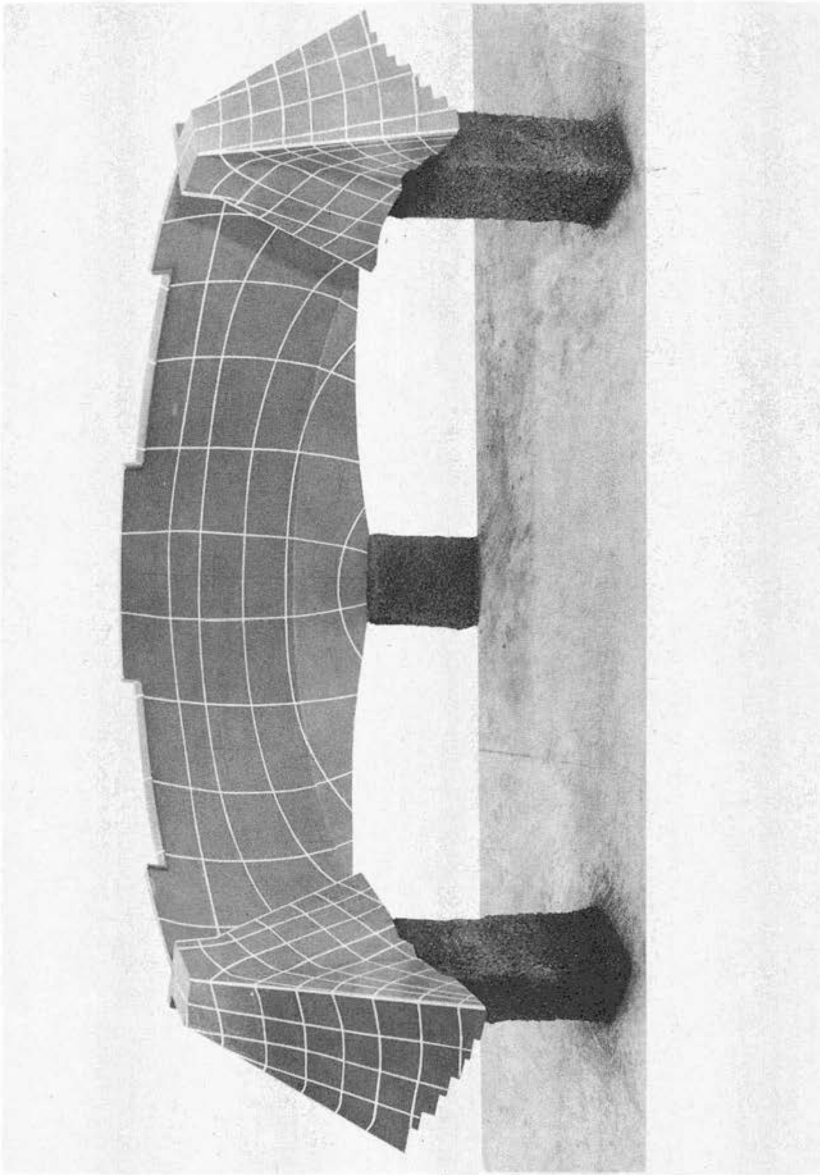
RIO FREDDO DAM

FIG. 7



Testing model constructed on the scale 1:50 complete with the equipment for applying the load and with the instruments for measuring the deformations.

RIO FREDDO DAM FIC



The isostatics on the downstream face of the arch-dam and on the abutments, as resulting from the load tests carried out on the model.

excavations were finished by hand using demolishing tools, without the use of explosives.

The aggregates for concrete were taken from borrow pits on the shore of the river Stura di Demonte; here they were sorted by means of rotary screens, after washing under pressure and the addition of material taken from the same borrow pit and suitably crushed.

The grading was: from 0 to 3 mm, from 3 to 10 mm, from 10 to 30 mm and from 30 to 60 mm.

The graded aggregates were then taken to the construction plant by truck, stored in bins and subsequently batched by weight.

Slow-setting cement was used, with standard mortar strength equal to 500 kg/cm^2 after 28 days.

The cement was batched by weight and proportioned in 300 Kg. per cubic metre for the vault and 200-250 Kg per cubic metre for the gravity wings.

Samples were taken daily from the concrete and submitted to strength tests. The tests were satisfactory having shown the following values at ultimate strength after a 7 day curing:

- 207 Kg/cm^2 for concrete at 250 Kg/m^3, with peak values of 277 and minimum values of 128
- 318 Kg/cm^2 for concrete at 300 Kg/m^3, with peak values of 417 and minimum values of 273.

The test samples taken after a 28 day curing gave instead the following figures:

- 266 Kg/cm^2 for concrete at 250 Kg/m^3, with peak values of 329 and minimum values of 177
- 372 Kg/cm^2 for concrete at 300 Kg/m^3, with peak values of 474 and minimum values of 314.

The concrete pouring was made by means of a main derrick with a 60 m boom placed in a central position so as to serve both the shoulders and the vault. For the right bank solid gravity wing an auxiliary derrick was used, with a 30 m boom (see Fig. 12).

The concrete was poured in steel forms and thoroughly acted on by vibrators.

Waterproofing grouting and bond grouting need no mention.

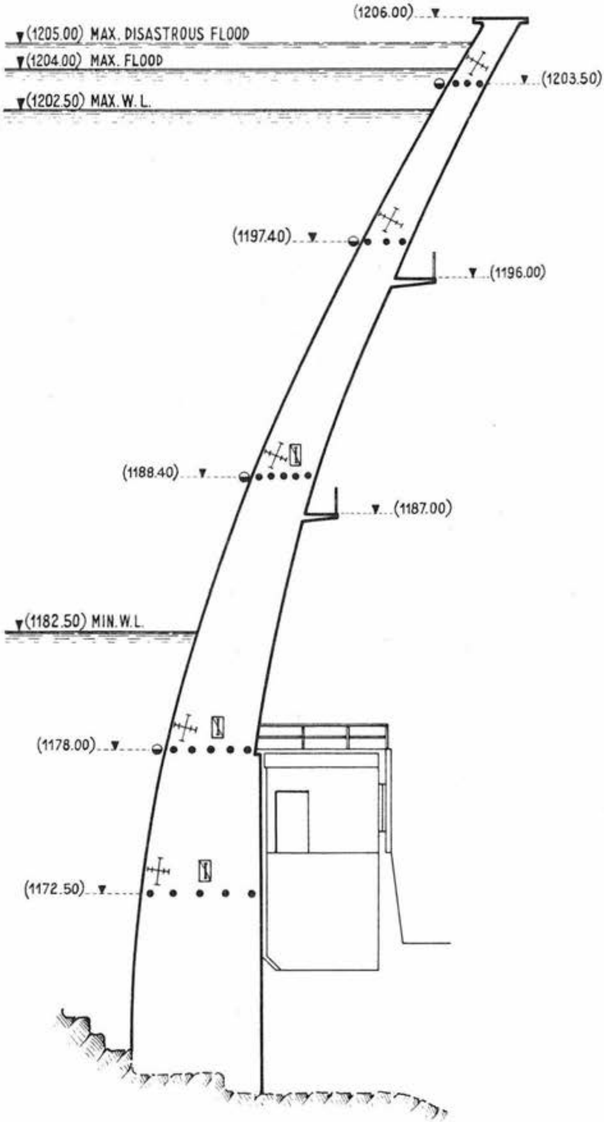
Measurements on the Dam

The following instruments were installed in the structure with the purpose of studying the behaviour of the dam (Figs. 9 and 10).

1) 31 electric thermometers with a central reading system, 27 of which were to measure the temperature inside the dam and 4 to measure water temperature at different elevations.
2) 90 electrically controlled thermal-extensometers were inclosed in the concrete about 40 cm from the upstream facing to measure temperatures and temperature-deformations. Five of these extensometers were isolated from the structure; they can detect both temperature and deformations due to thermal effect alone and to humidity.

CROSS SECTION
1 : 200

FIG



● ELECTRICAL THERMOMETER

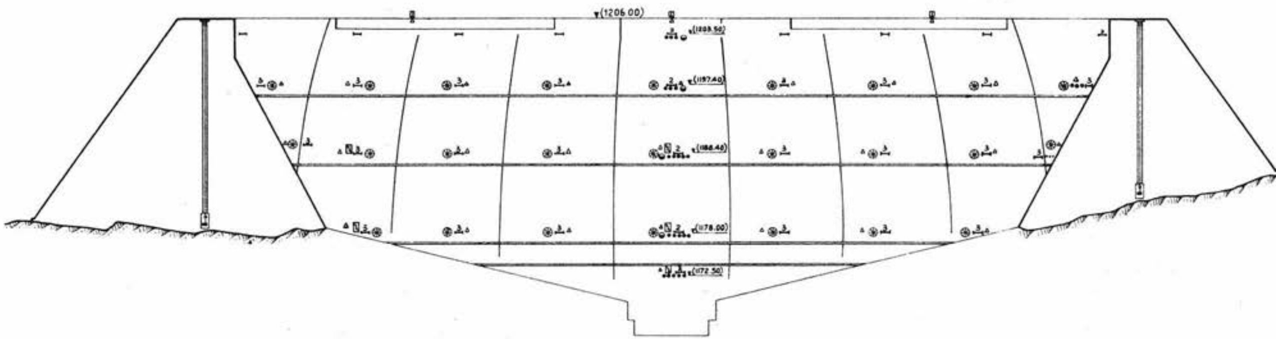
ELECTRICAL THERMOMETER FOR WATER

ELECTRICAL HUGGENBERGER EXTENSOMETER

ISOLATED HUGGENBERGER EXTENSOMETER

POSITION OF MEASURING INSTRUMENTS

DOWNSTREAM FACE

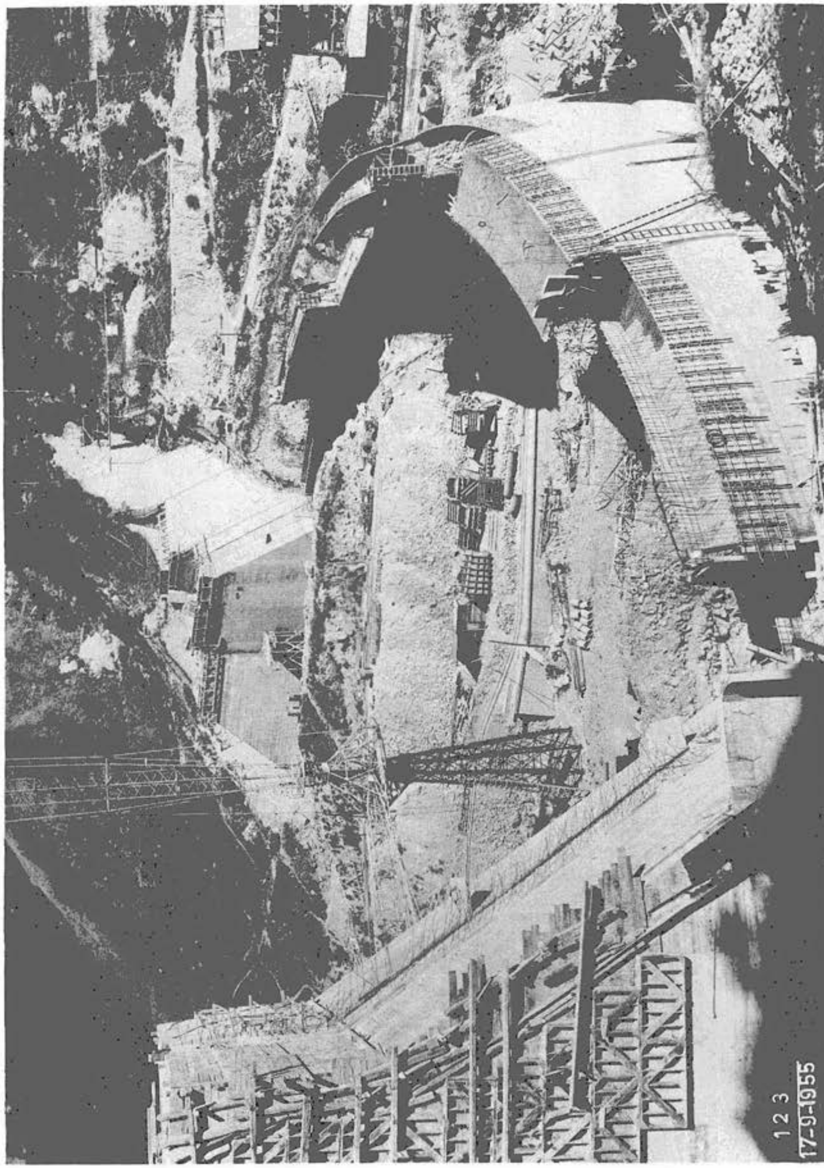


- ⊛ SET OF REMOVABLE EXTENSOMETERS
- ⟝ ELECTRICAL EXTENSOMETER
- ISOLATED EXTENSOMETER
- △ TRIANGULATION TARGET
- • ELECTRICAL THERMOMETER
- ◒ ELECTRICAL THERMOMETER FOR WATER
- ADJUSTABLE COLLIMATOR TARGET
- PENDULUM

MARCELLO

FIG. 10

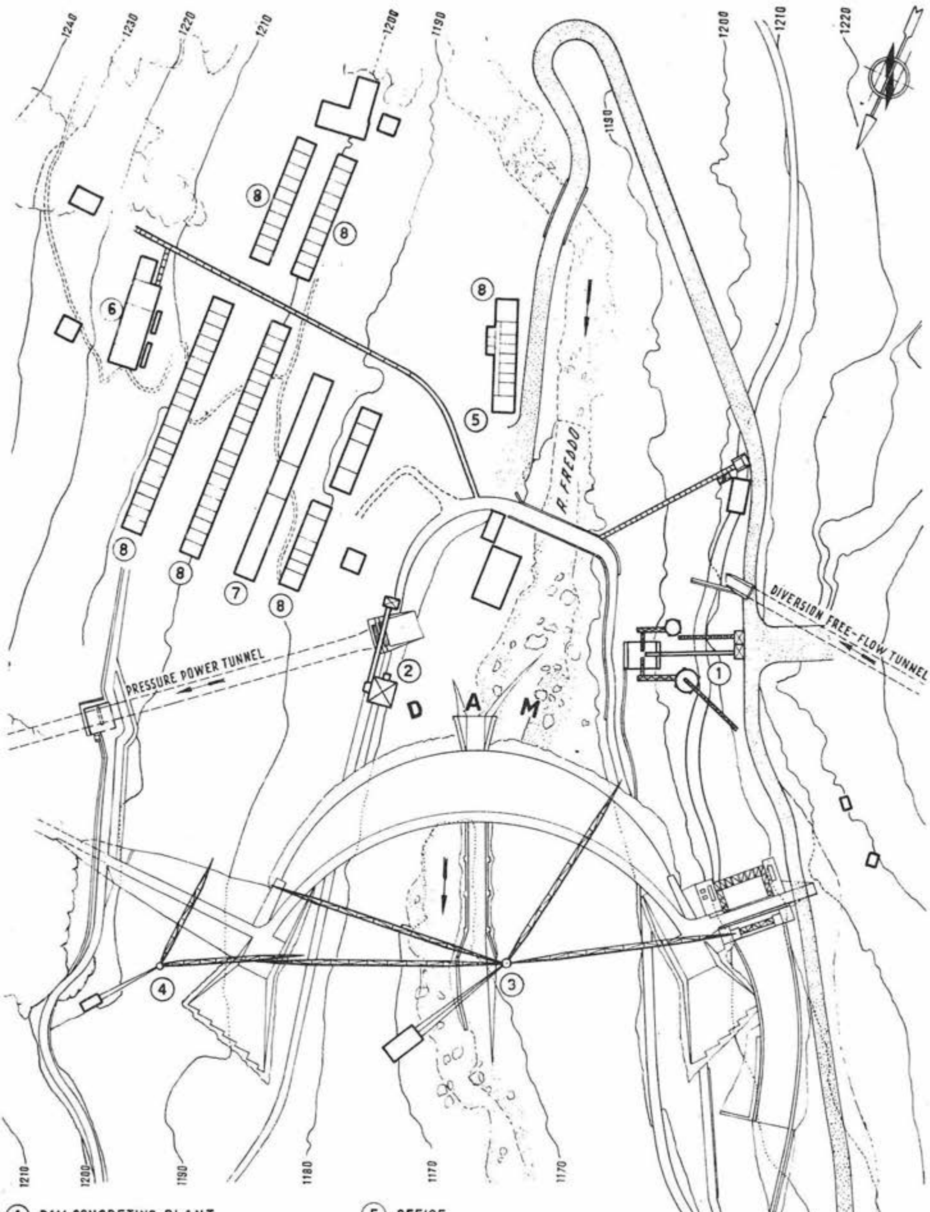
RIO FREDDO DAM FIG.1



View of the dam under construction. In center foreground see the main derrick with a 60 m boom for the distribution of the concrete. The concrete was poured in steel forms.

CONSTRUCTION PLANT
LAYOUT

FIG. 12



1. DAM CONCRETING PLANT
2. TUNNEL CONCRETING PLANT
3. MAIN DERRICK - BOOM 60.00 m
4. AUXILIARY DERRICK - BOOM 30.00 m
5. OFFICE
6. WORKSHOP
7. CANTEEN AND KITCHEN
8. DORMITORIES

RIO FREDDO DAM

3) 25 rosette-shaped bases for removable extensometers were installed on the downstream facing at points corresponding to the measurement point on the upstream facing.
4) 2 coordimeters in the shoulders to measure local deformations.

On the downstream facing, triangulation targets were also installed, which were related to a system of 9 triangulation bases.

Three adjustable collimation targets were fixed on the crest of the dam to detect horizontal deflection of the crest; these were made in reference to stationary targets on the banks.

The dam design and the work direction were carried out by the Hydroelectric Plant Construction Department of Soc. Edison of Milan, under the direction of the writer.

The construction was entrusted to the Contractor Girola of Milan.

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POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: DESIGN AND OBSERVATION OF ARCH DAMS IN PORTUGAL

M. Rocha,* M. ASCE, J. Laginha Serafim,** A.M. ASCE, and A. F. da Silveira***
(Proc. Paper 997)

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams held in April,1939, much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be included in the Symposium. So far, ten papers have been approved: "Arch Dams: Their Philosophy" (Proc. Paper 959) by Andre Coyne; "Arch Dams: Trial Load Studies for Hungry Horse Dam" (Proc. Paper 960) by R. E. Glover and Merlin D. Copen; "Arch Dams: Portuguese Experience with Overflow Arch Dams" (Proc. Paper 990) by A. C. Xerez; "Arch Dams: Theory, Methods, and Details of Joint Grouting" (Proc. Paper 991) by A. Warren Simonds; "Arch Dams: Santa Giustina Singlè-Curvature Arch Dam" (Proc. Paper 992) by Claudio Marcello; "Arch Dams: Measurements and Studies on Santa Giustina Dam" (Proc. Paper 993) by Claudio Marcello; "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam" (Proc. Paper 994) by Claudio Marcello; "Arch Dams: Isolato Double-Curvature Arch Dam" (Proc. Paper 995) by Claudio Marcello; "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs" (Proc. Paper 996) by Claudio Marcello, and "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira.

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As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

SYNOPSIS

Data are presented on design model testing and field observation of six arch dams ranging in height from 60 m to 135 m. This has enabled later designs to be modified to gain increased economy with safety. Model studies determined stress conditions at irregularities in foundation and around special structures in arch.

INTRODUCTION

The lack of mineral fuel in Portugal has lead to a large program of hydroelectric undertakings, which started in 1946. In view of the hydrologic regimen of the rivers being characterised by large flows in the winter and very small flows in the summer, it was necessary to create large reservoirs which call for high dams.

As so far narrow valleys have been available, in which the ratio between the length and the height has not been greater than four, and, as in the great majority of these sites there were rocks of suitable quality, arch dams have been constructed. The undoubted safety and economy of this type of dam has also contributed largely to its adoption, even in cases where the site has not appeared very favourable.

The designs of all the dams built have been accompanied by model studies and the detailed observation of their behaviour carried out by the Laboratório Nacional de Engenharia Civil. This has made it possible to evaluate the methods for analysing arch dams and also to obtain fundamental data on the most suitable shapes. Thus, whilst the two first arch dams built are of the arch gravity type with a cylindrical upstream face, the latter ones are of the cupola type, that is, with a vertical curvature and varying radius arches.

Characteristics of the Dams and Their Foundations

This paper describes some of the most important studies carried out on the main Portuguese arch dams built since 1946 (Fig. 1), whose principal characteristics are given in Table I.

In the construction of these dams, use was made of Portuguese cements, similar to type II of the A.S.T.M. standards, and concretes having aggregate with a maximum dimension of 15 cm. They were built in separate blocks having radial joints with a maximum distance between the joints of 15 m. Lifts of one and a half or two meters were laid, with an interval between each lift of 4 to 5 days and high frequency vibrators were used. Only in the Castelo do Bode and Cabril dams was cold water and ice used in mixing the concrete. No artificial cooling of the concrete in place was done.

The foundation characteristics were surveyed geologically and by borings and galleries. However, in view of the importance of the deformability of the foundation on the behaviour of arch dams, the need was recognised of

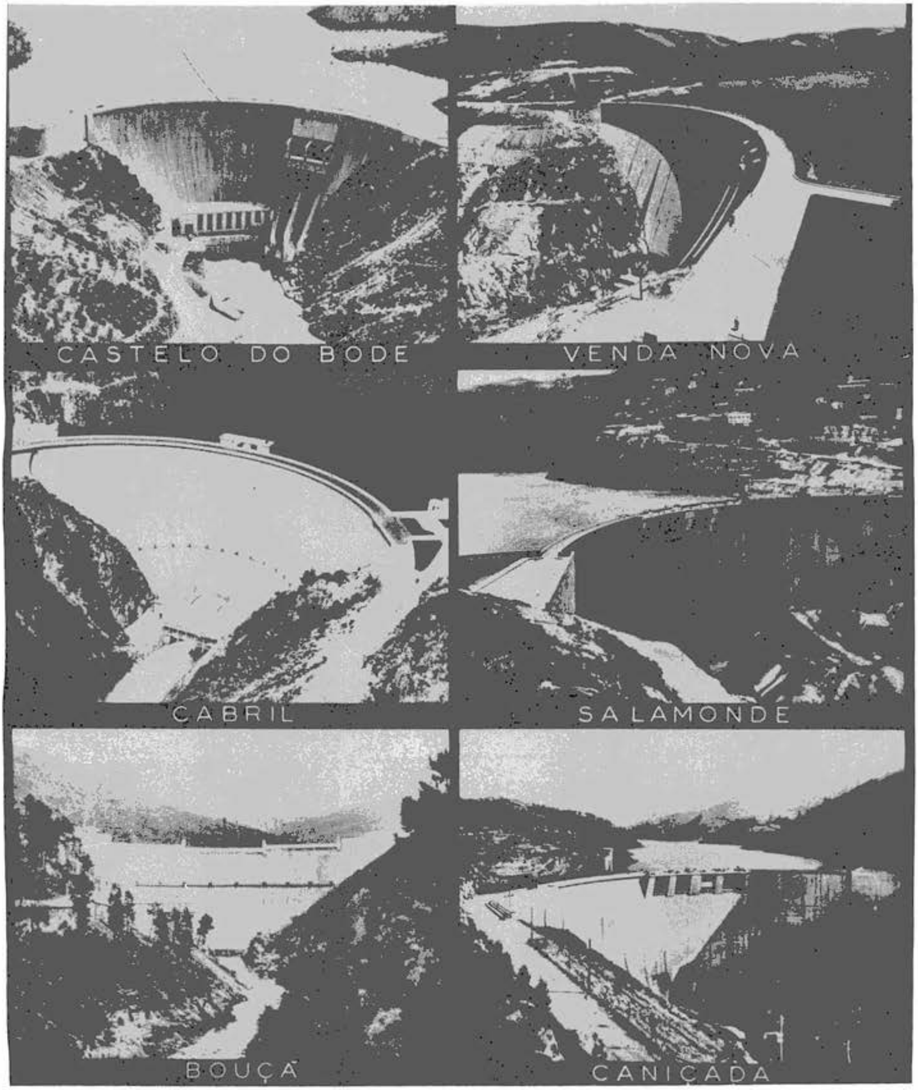


FIG. 1 - Main Portuguese arch dams.

TABLE I

CHARACTERISTICS OF THE MAIN PORTUGUESE ARCH DAMS

Name of Dam (owner)	Type	Height (m)	Thickness at the base (m)	Upstream radius at the crest (m)	Angle at centre (degrees)	Volume of concrete (m^3)	Date of Construction: Commencement of laying concrete	Date of Construction: Conclusion	Designer
Castelo do Bode (Hidro-Eléctrica do Zêzere)	AG,c.r.	115	34	150	112^{o}	450.000	Jun.48	Aug.50	A. Coyne
Venda Nova (Hidro-Eléctrica do Cávado)	AG,c.r.	97	33	154,5	95^{o}	220.000	Oct.49	May 51	A. Coyne
Cabril (Hidro-Eléctrica do Zêzere)	A, v.r.	135	19	155	110^{o}	360.000	Jun.52	Dec.53	Owner
Salamonde (Hidro-Eléctrica do Cávado)	A, v.r.	75	8	115	100^{o}	93.000	Apr.52	Apr.53	A. Coyne
Caniçada (Hidro-Eléctrica	A, v.r.	76	8	105	107^{o}	80.000	Jul.53	Aug.54	Owner
Bouçã (Hidro-Eléctrica do Zêzere)	A, v.r.	60	7,5	95	$102^{o}50'$	35.000	Nov.54	Aug.55	Owner

AG - arch-gravity; A-arch; c.r.- Constant radius; v.r.-variable radius determining these characteristics both by laboratory and site tests. The laboratory tests consisted in the determination of the modulus of elasticity of the cores obtained from the various borings after choosing them judiciously and also of prisms from the galleries. The tests on site were carried out on all dam foundations mentioned here, except Venda Nova. The methods used in these tests and the results obtained have been described elsewhere.(1) One method consists in loading a section of a circular gallery with water under pressure and measuring the diameter variations. The other consists in loading simultaneously two opposite walls of a section of a gallery by means of jacks and metallic cushions filled with oil to distribute the load on the rock uniformly. These tests were, in many cases, carried out before and after grouting the rock round the galleries.

Table II shows the foundation characteristics of the dams described in this paper, and the values $\underline{E_r}$ of the moduli of elasticity of the rock along the valley, deduced from all the tests carried out. It also shows the type and number of tests made.

Design of Arch Dams. Model Tests

As present day designs are based on the values of the stresses produced by the loadings, it becomes necessary, especially for structures involving great responsibility and cost, to seek methods which give the value of these stresses with reasonable accuracy.

When the construction of large arch dams was begun in Portugal, there was already some experience of model studies obtained from model tests of Santa Luzia Dam.(2) One criticism that could be made then of the current analytical methods for dam design was that they were either insufficient or very complex. It was concluded that, among the existing methods, the "trial load" method was the one which could supply the most reliable results, even though its full application, that is, with radial, tangential and twist adjustments, was very laborious and lengthy. In principle, this method can be applied to concrete dams of any shape and take into consideration the true foundation deformability. However, when the valley is considerably assymmetric or the foundation profile irregular, or when the dam has important gravity abutments or large openings—for example, spillway openings—or even when the foundations are heterogeneous, then the number of arches and cantilevers which have to be considered, as well as the number of trials, entail excessive work and time.

Besides this, whenever the hypotheses of Strength of Materials are inadequate, as happens in the fields of stress which exist round very irregular shapes, the method, by its very nature, cannot give satisfactory results.

In any case, the method was applied to all the dams mentioned in this paper, but in its abridged form, that is with radial adjustment of the arch and cantilever displacements only. The comparisons which were made between the results obtained by this abridged method and the results from models and observation(3) led to the conclusion that it can only be used for an estimate of the stresses in a preliminary design. As a rule, in the case of regular shapes, the stresses determined in the arches by this method are from ten to twenty per cent greater than those obtained from models, whilst for stresses at the base cantilevers the former stresses are much greater than the latter. It should also be noted that the maximum arch compressive stresses given by

TABLE II

CHARACTERISTICS OF THE FOUNDATIONS OF THE MAIN PORTUGUESE ARCH DAMS

Dam			Castelo do Bode	Venda Nova	Cabril	Salamonde	Caniçada	Bouçã
Right bank	Upper part	Rock	Altered schist	Schist (on top)	Granite	Granite	Granite	Altered schist
		Er ($kgcm^{-2}$)	40.000	40.000	150.000	80.000	100.000	20.000
	Lower part	Rock	Cristal. schist	Granite	Hard granite	Hard granite	Hard granite	Schist
		Er ($kgcm^{-2}$)	110.000	100.000	200.000	200.000	200.000	60.000
Bottom of the valley		Rock	Cristal. schist	Hard granite	Hard granite	Hard granite	Hard granite	Schist granite
		Er ($kgcm^{-2}$)	110.000	200.000	220.000	200.000	200.000	100.000
Left bank	Lower part	Rock	Cristal. schist	Hard granite	Hard granite	Granite	Hard granite	Granite
		Er ($kgcm^{-2}$)	110.000	200.000	200.000	80.000	150.000	100.000
	Upper part	Rock	Hard cristal. schist	Altered granite (on top)	Granite	Altered granite	Altered granite	Altered granite
		Er ($kgcm^{-2}$)	200.000	50.000	70.000 and 150.000 (upper)	20.000	50.000	20.000
Tests Carried out		In the Laboratory	41 cores 12 prisms	73 cores 10 prisms	78 cores 16 prisms	30 cores 11 prisms	55 cores 6 prisms	78 cores 34 prisms
		On Site	1 shaft with water	---	5 galleries with jacks	5 galleries with jacks 2 galleries with water	2 galleries with jacks	2 galleries with jacks

calculation and by the models, even when being of the same magnitude, are not found to be at the same points. The radial displacements—the only ones which are determined—are also from ten to twenty per cent greater than those of the models.

These results led to the use of model tests as the basis for the design of the Portuguese dams. As a matter of fact, the improvement in test techniques not only made it possible to be confident of the experimental results but also to reduce the time of test, which is now less than that needed for the use of the abridged "trial load" method. Thus, in the majority of the dams, the final design was based on the successive studies of models in which the shapes were progressively improved. These studies have been started as soon as the preliminary design of the structure was made. The test techniques used at present have already been described.[3,4] Table III shows the number and characteristics of the models, the duration of the tests and the cost of the model tests. As can be seen from this Table it is the practice at present to test at least two equal models symultaneously for each shape of dam analysed. By this means the reliability of the results is increased and the accuracy is improved by taking the mean values of the two models.

In model studies, as in calculations, it is assumed that the dam is a homogeneous, isotropic, elastic and continuous solid supported on an elastic foundation.

Normally the models are only analysed for hydrostatic pressure but in some cases the stresses due to the weight of parts of the structure itself were also determined. Usually the stresses due to this loading are calculated by assuming that the dam is built in separate blocks. The effects of the remaining loadings, such as temperature, earthquakes and uplift, are also calculated analytically and added to the stresses due to hydrostatic pressure. It should be noted that usually the stresses due to hydrostatic pressure are very much greater than the stresses due to the other loadings together.

The stresses in the models are determined by electrical strain gauges, the SR-4 Baldwin types A7 and A8 being particularly used.

Previously the isostatics of the two faces are determined with brittle lacquers. Radial, tangential and vertical displacements are also measured.

The measurements are nearly always determined for various water levels in the reservoir which is particularly important for their comparison with the prototype observations. This fact is a further advantage of the models over the calculation methods.

Recently a supplementary experimental method was investigated in the search for the best shapes of arch dams. It consists in determining the shapes taken by a rubber membrane fixed across a valley and loaded from downstream with water and with upward forces proportional to the weight of the concrete.[5]

The Value of Observations

The observation of dams both during and after construction has two objectives, the control of safety and the investigation of the behaviour. The observation can check the methods used in design, whether analytical or experimental, and also judge the building methods followed.

For the task of observation to be fruitful the equipment must be carefully chosen and the work of installing and reading the instruments, calculating and

TABLE III

MODELS OF MAIN PORTUGUESE ARCH DAMS

Name	Studies carried out	No. of models tested	Scale of the models	Material of the model and loading system	Duration of the tests (months)	Cost of the tests (dollars)
Castelo do Bode	Study I Study II	1 4	1/ 75 1/500	Pd — M Pd — J	8 4	4,800
Venda Nova	Study I Study II Study III	1 2 2	1/100 1/300 1/ 30	Pd — M Pd — J Pd — J	8 4 3	6,000
Cabril	Study I Study II Study III Study IV Study V	2 2 2 1 1	1/300 1/300 1/300 1/400 1/400	Pd — M Pd — M Pd — M A — M A — M	6 4 4 6 3	7,800
Salamonde	Study I Study II Study III Study IV	2 2 2 2*	1/200 1/200 1/200 1/200	Pd — M Pd — M Pd — M Pd — M	6 4 4 4	7,200
Caniçada	Study I Study II	2 2	1/200 1/200	Pd — M Pd — M	6	4,200
Bouçã	Study I	3	1/200	Pd — M	6	3,000

Notes

Pd - Mixture of plaster of Paris and diatomite.

A - "Alkathene".

M - Mercury contained in a rubber bag.

J - Set of jacks.

* - A" Marco" model was also built (Scale 1/250).

interpreting the results has to be carried out by fully trained specialist teams supported by a laboratory where the various tests both of the instruments and of the materials of the dam can be made.

However, in comparing the results of observation of a structure with the results of model tests a difficulty arises from the fact that the prototype is subject to many actions which are not easily reproducible in the model, such as temperature variations either from external variations or from dissipation of the heat of hydration of the cement, the anelasticity of the concrete and rock, foundation settlements, movements of the reservoir banks etc.

The Laboratorio Nacional de Engenharia Civil carries out[6,7] all the observation work in which other Government departments and the owners of the dams take part. An elaborate plan for observation is drawn up by the L.N.E.C. which also maintains a team on the site for placing and reading apparatus. The readings are sent regularly to the Laboratory where their calculations, plotting and interpretations are carried out, which calls for numerous tests on the concrete.

The values which have been observed so far are: absolute dislacements of points of the dam and on the ground by geodetic and alignment methods;[8] vertical displacements by precision levelling of the crest of the dam and at points on the rock downstream;[9] relative displacements by pendulums;[10] rotations with clinometers; joint movements, inside with joint meters and outside with extensometers;[11] air, concrete and reservoir water temperatures by thermo-electric couples; strains by means of strain meters;[12] compressive stresses with Carlson stress-meters; neutral pressures by Carlson meters; humidity with humidity measuring instruments, and uplift pressures with pressure tubes equiped with manometers.

Table IV gives the apparatus used in the arch dams referred to in this paper.

Results of Studies

Castelo do Bode

The Castelo do Bode dam (Figs. 1 and 2 and Table I) presents some important features. In the first place the right bank is a very deformable promontory which, in the upper part, supports a large artificial abutment. Though it is approximately symmetrical, the dam has two flood spillway openings of 15 x 12 m. Above these openings the dam was thickened in order to transmit the forces from one side of the opening to the other more effectively. The dam was calculated by radial adjustments between four arches and three cantilevers.

Model studies were undertaken to determine the general field of stresses, the most suitable dimensions for the thickening of the dam in the neighbourhood of the spillway openings and the dimensions of the artificial abutment. A first model was built (Table III) to a scale of 1/75 in which the various mechanical properties of the foundation were reproduced. The failure of this model, which occured in the third load test, showed the advisability of increasing the dimensions of the artificial abutment.

In order to expedite the construction and drying, four smaller models were built later to a scale of 1/500. These models were loaded with a system of 26 hydraulic jacks of different diameters.

The results of the tests (Fig. 3) showed that in the spillway area the

TABLES IV

APPARATUS PLACED IN PORTUGUESE ARCH DAMS

	Dams	Castelo do Bode	Venda Nova	Sala-monde	Cabril	Caniçada	Bouça
Displacements by geodetic method	Theodolite stations	4	3	4	4	4	4
	Sighting marks	17	18	14	19	14	14
	Check points	4	5	4	5	4	5
	Alignments	4	-	6	3	6	6
	Levelling points	25	15	10	21	10	10
Other Observations	Plumb-wires	3	3	1	2	1	-
	Coordinometer bases	-	-	-	-	-	-
	Coord.-Clin. bases	20	-	6	6	6	-
	Clinometer bases	3	3	14	14	14	10
	Strain--gauges	164 (a)	181 (a)	140 (b)	189 (c)	184 (c)	179 (c)
	Carlson stress-meters	-	-	-	-	6	9
	Resistance thermometers	71	80	-	-	-	-
	Thermo-electric couples	44	53	50	50	26	33
	Uplift points	-	-	-	12	-	-
	Carlson pressure meters	-	-	-	-	6	6
	Joints. Detachable gauges	151	55	43	100	50	50
	Joint-meters	-	-	21 (b)	55 (c)	21 (c)	49 (c)
	Moisture content meters	-	-	-	6	6	6
	Length of cables (m)	15.000	8.000	3.000	7.000	6.000	11.500

(a) Telemac (b) Galileo (c) Carlson

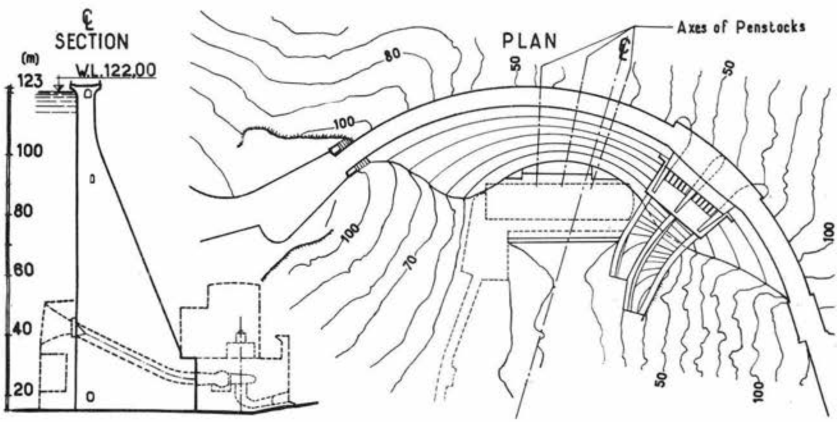


FIG. 2 – CASTELO DO BODE DAM. Plan and section.

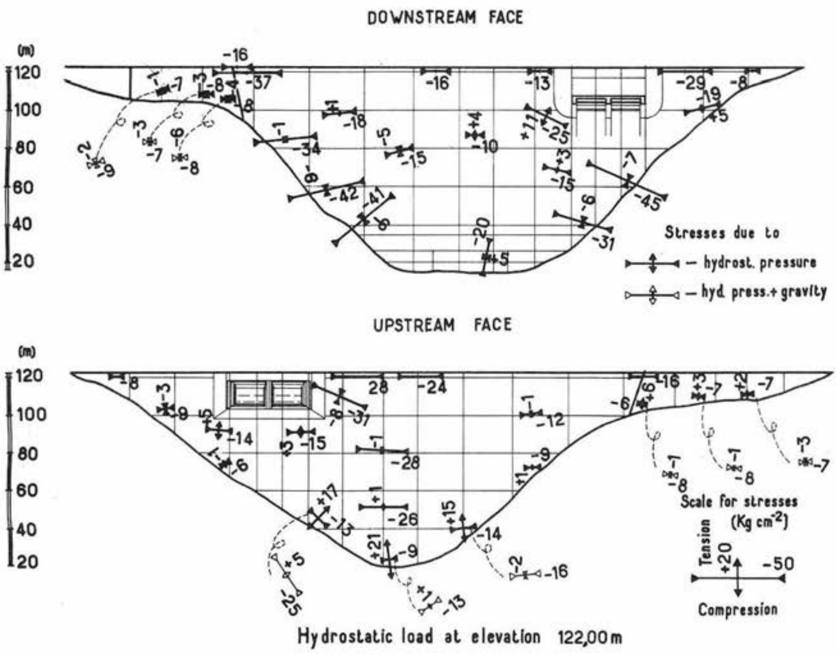


FIG. 3 – CASTELO DO BODE DAM. Stresses from model studies.

directions of the principal stresses were diverted by the openings made in the dam. The compressive stresses due to hydrostatic pressure did not exceed 45 $kgcm^{-2}$ on the upstream face or 28 $kgcm^{-2}$ on the downstream face. For these regions the calculations gave maximum horizontal stresses, in the arches, of 42 and 43 $kgcm^{-2}$ respectively. The maximum tensile stresses observed in the heel of the dam for the hydrostatic pressure were 21 $kgcm^{-2}$.

For points in the upstream face of the dam base, Fig. 3 also gives the sum of the stresses due to the hydrostatic pressure with those of the weight of the dam itself calculated analytically. The tensile stresses due to hydrostatic pressure are practically anulled. The same happens with the tensile stresses in the base of the abutment of the right bank.

As to the spillway openings, it was concluded that, in spite of their large dimensions, they only influence the stresses of the dam locally, the continuity being assured by the arch passing above these openings. The thickness of this arch should have been reduced as, in view of its rigidity, the stresses developed in its neighbourhood were high.

The model tests also made it possible to design the reinforcement to be placed inside the vertical walls of the openings in order to support the high tensile stresses which are developed there.[4]

Geodetic observations of the dam displacements (Fig. 4) show a movement of the whole dam toward the right bank and important movements of the abutment which reach a value of 2 cm. These displacements are a result of hydrostatic pressure on the actual promontory supporting the dam and also of the low modulus of elasticity of the rock in this region (about 40,000 $kgcm^{-2}$). However, as the displacements are being assymptotic with time it can be concluded that the structure is safe.

It was noted that the displacements observed in the upper part of the dam are greatly influenced by air temperature variations.

Fig. 5 shows some typical results of observations with strain meters. These strain meters are arranged in 45^{o} rosettes of four meters and no-stress meters.[13]

The analysis of the diagrams shows a considerable difference in their development. The diagrams of downstream groups 2d and 3d accompany the air temperature variations showing compression with a rise in temperature. The diagrams of groups 1u, 2u and 3u, placed at one meter from upstream face, and the group 1d, which is protected by the power house, show very little influence of temperature variations.

Group 1u, during 1949, registered a continuous increase of negative strains due to the increase of the weight during construction. At the end of that year there was a reduction of the compression due to temperature fall and then a further increase in negative strain due to temperature rise. Since the reservoir was filled the effect of air temperature variations was no longer apparent on the behaviour of the diagrams and the strain meters only registered slight effects of water level variations and of the general cooling of the dam.

The group 1d never registered large strain variations but the effect of reservoir level variations is noticeable, principally in the vertical meter.

The annual air temperature cycle, whose effect is recorded without any appreciable time lag in groups 2d and 3d, produces a state of hydrostatic stress as is shown by the parallel development of the diagrams of the 45^{o} meters in group 2d or of the vertical and horizontal ones in group 3d. These directions are those least affected in the two groups by variation of hydrostatic pressure. On the other hand the diagrams of the vertical and horizontal

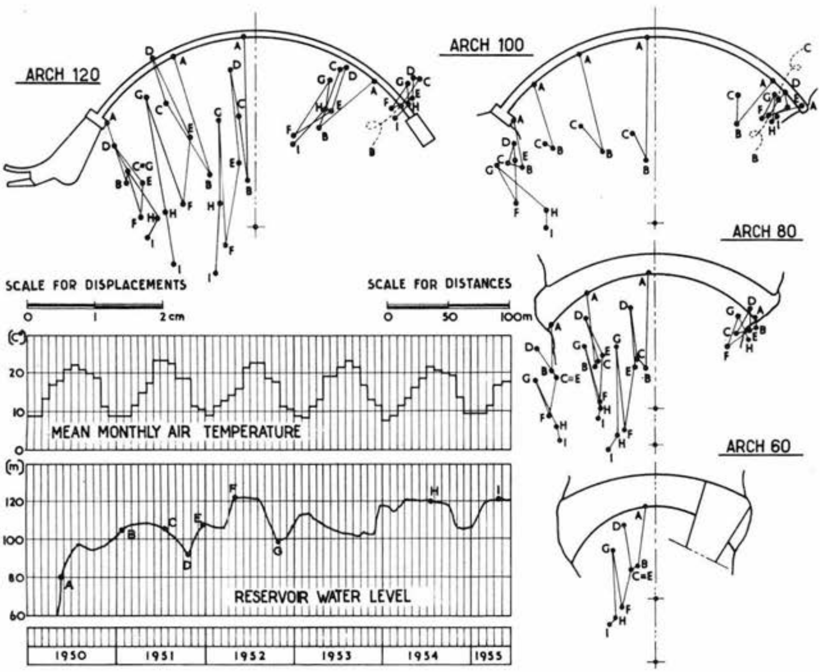


FIG. 4 - CASTELO DO BODE DAM. Horizontal displacements observed by the Geodetic Method.

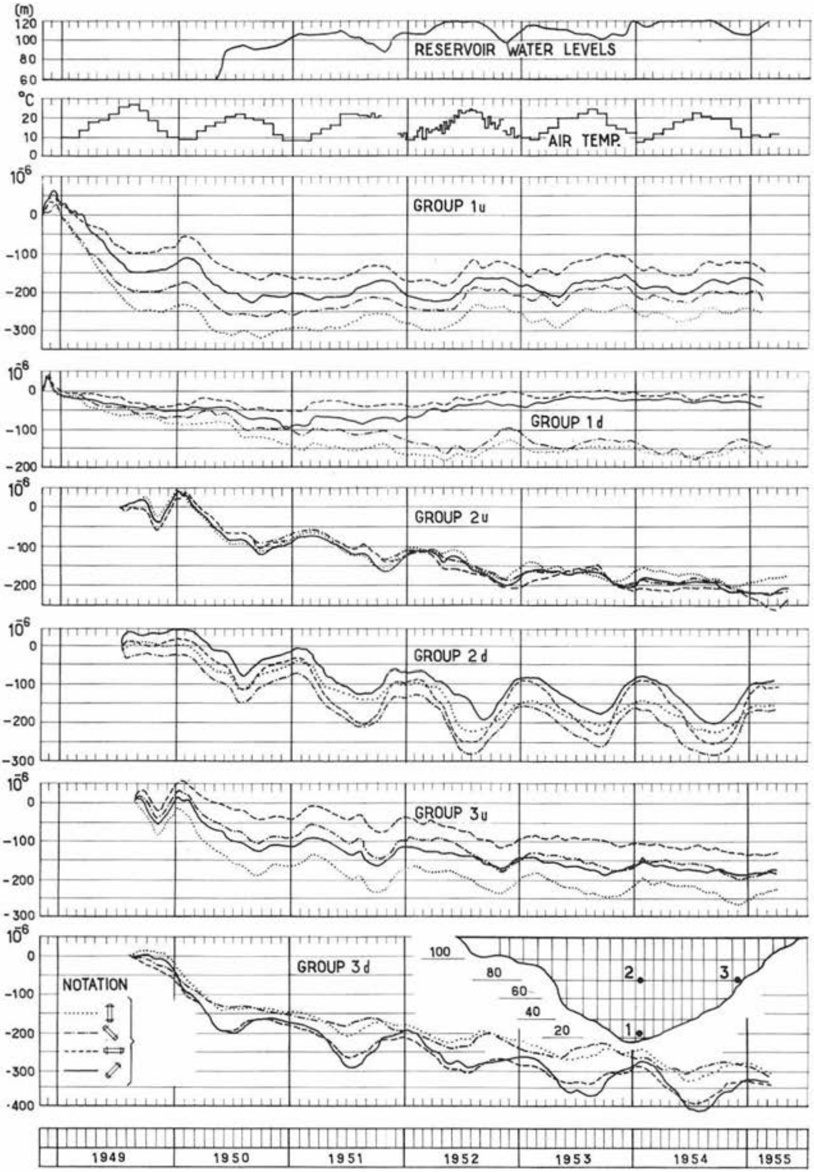


FIG. 5 – CASTELO DO BODE DAM. Corrected observed strains at points 1m from the upstream and downstream faces.

meters of group 2d and those of 45° in group 3d, which are those most affected by the variation in level of the reservoir, approach one another and separate with such variations. The same can be said in relation to group 3u although in this case the variations in strain due to temperature are small.

The isotherms and flow lines in Castelo do Bode dam and in the reservoir water are plotted in Fig. 6 for four typical dates of 1954 when the heat of hydration of the cement had already dissipated. The mean monthly air temperatures (T_A) of the month prior to the day in which readings were taken are given.

The analysis of the figure shows that the reservoir water still undergoes appreciable temperature variations to a depth of about 50 m. On the other hand the isotherms of the water round the intakes (Fig. 2 and 6) change their direction and became vertical, which is attributed to the flow of water at the entrance to the penstocks. This fact causes unusual heat flows near the heel of the dam, as can be seen in the diagrams relative to 30th October and 30th December 1954.

The flow lines in the dam show that it gives up heat to the water and the foundation rock all the year round. With regard to heat exchange at the downstream face, the dam receives heat during the summer months and gives it up during the winter. The influence of the solar radiation on the downstream surface is very accentuated as it faces southwest. Particularly round the crest, where the sun's heat falls on the top and on the downstream face, the temperatures reach very high values. Differences of 12°C have been observed between the temperature at the surface of the concrete and the air temperature one meter away from this surface.

Venda Nova

Venda Nova dam (Fig. 1) is a structure very similar to that of Castelo do Bode except that the lower part of the valley is much narrower. It also has a lateral spillway for which it was necessary to leave two openings of 8 x 6 m each in the structure. The dam was calculated by the abridged "trial load" method with four arches and three cantilevers.

The first model studied had a projecting rock in the lower upstream left bank as is represented by a broken line in Fig. 7 (Study I). A tension due to hydrostatic pressure of 43 $kgcm^{-2}$ was measured in this region on the model and after superimposing the weight a stress of 32 $kgcm^{-2}$ still remained. For this reason the canyon profile was made more regular by cutting the projecting rock and Study II was carried out in which tensile stresses of 15 $kgcm^{-2}$ were derived for the combined effect of hydrostatic pressure and weight. It should be mentioned that the studies on models were begun when construction was already started and therefore with the foundation opened up, that is when it was not possible to make large modifications in the design. It was therefore decided to make a joint ending in a gallery in the upstream face (Fig. 7, Study III). The object of this joint was to prevent the dam from cracking which would endanger its water-tightness. Once the joint opens the stresses due to the weight of the dam compensate for the tensile stresses due to the hydrostatic pressure at the inside end of the joint. In models of this solution (Study III) the stresses at the bottom of this joint and at various points on the two faces of the dam were determined. The tensile stresses in the middle region were in fact anulled by the joint but near the ends of the joint there were still tensions of 13 $kgcm^{-2}$ for the hydrostatic pressure and the weight.

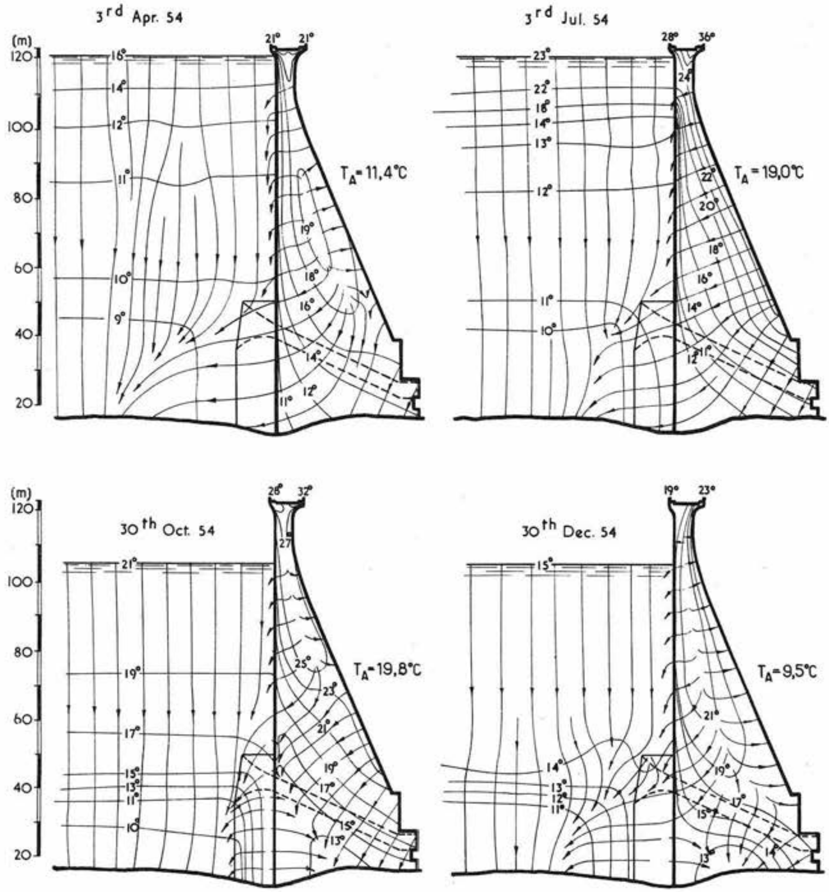


FIG. 6 – CASTELO DO BODE DAM. Flow lines and isotherms.

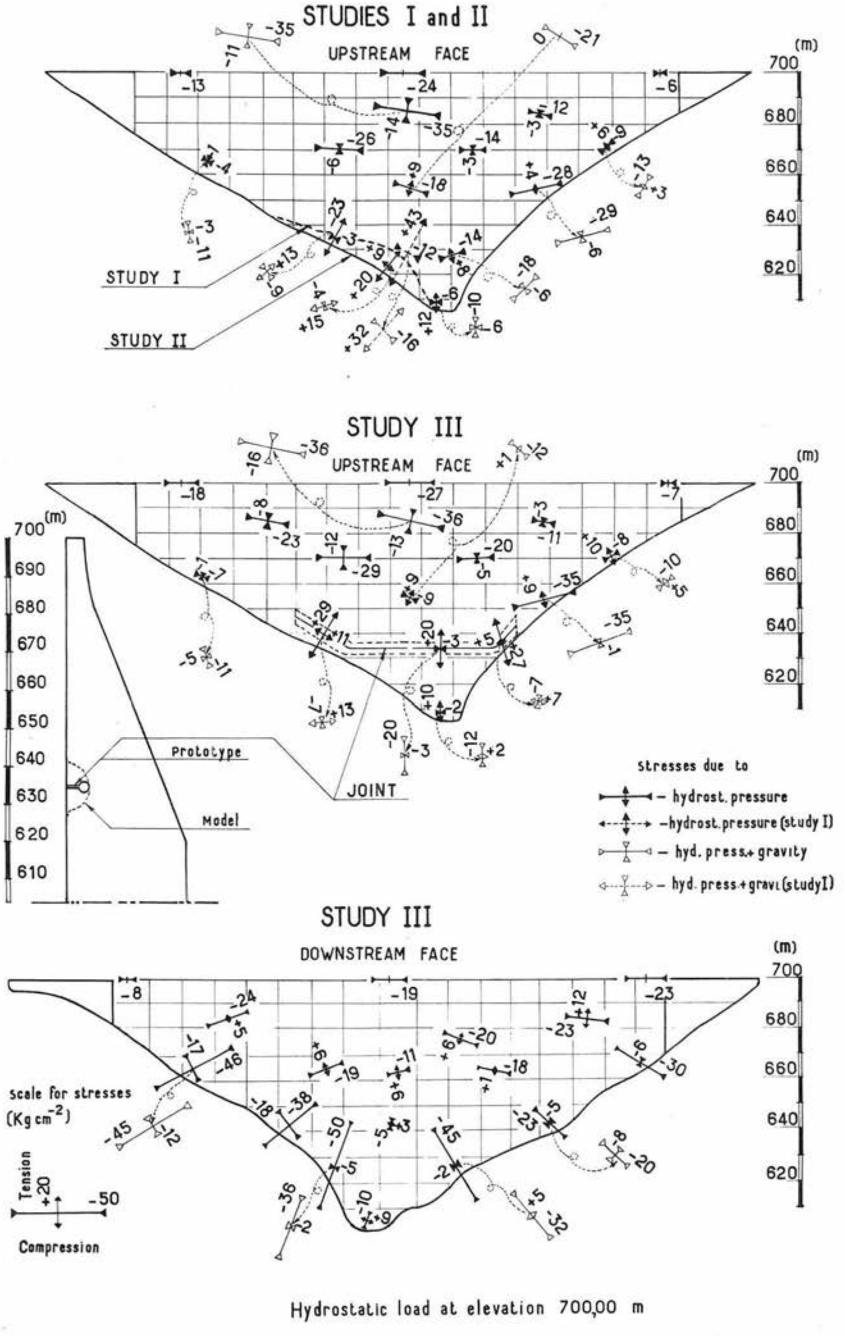


FIG. 7 —VENDA NOVA DAM. Stresses from model studies.

Also as a result of the considerable change of the field of stresses due to the weight itself when the joint opens, the downstream compression stresses due to hydrostatic pressure are very reduced, the maximum compression becoming 36 $kgcm^{-2}$ instead of 50 $kgcm^{-2}$.

Fig. 8 shows the values of the vertical stresses in a horizontal section in the region of the flood spillway at level 690 determined on the model for the hydrostatic pressure. These values, after being combined with the weight stresses enabled the reinforcement needed to be designed. The stresses in the prototype obtained by means of vertical vibrating wire strain meters left in the concrete at one meter from the surface are also indicated. Note that the stresses in the prototype were measured when the water level rose in the reservoir from 668.00 m (30th Oct. 1954) to 700.00 m (31st Jan. 1955), whilst in the model the stresses were determined for full hydrostatic pressure (reservoir empty to level 700,00).

The radial and tangential displacements observed in the structure (Fig. 9) show that the dam has symmetrical and elastic behaviour. The reservoir reached levels near the maximum on dates d, g and h, for which the radial displacements were maximum. On date e the lowest level since the beginning of the filling was reached and the points representing radial displacements at this date are situated a little downstream those for date b and practically coincide with those for date c. This fact is probably due to the cooling of the dam. The effect of this cooling is apparent from date b to c as the level for the two dates was the same and as the points for date c are downstream to those for date b. The same is seen for the dates d and h. Following date e, when the dam reached thermic equilibrium, it has had a completely reversible behaviour as can be seen by comparing curves for dates g and h. On date h the points are a little downstream as, though the temperature was the same, the water level was a little higher.

Conclusions identical to those above are reached from the comparison of the diagrams of tangential displacements. However, a certain anomaly in the shape of the curves for dates b, c, d and e can be seen which may be due to cooling of the dam and to tangential movements of the foundation. After date e the curves have had regular shapes.

The radial displacements given by the model (Fig. 9) are considerably smaller than those measured in the prototype. The apparent agreement at levels 700 and 680 is due to the fact that the initial measuring date at these two levels is date b for which the reservoir level was already high. The greatest contribution for the difference in displacements is that due to the concrete cooling which occured mainly between dates a and d.

Salamonde

Fig. 10 shows the cross section and upstream elevation corresponding to four model studies carried out for the Salamonde dam. This is a thin symmetrical arch dam (Fig. 1). The arches have variable thicknesses without fillets. Although the foundation of the dam has very low moduli of elasticity in the upper left bank (Table II), the model studies were carried out assuming the dam and the foundation to have the same modulus of elasticity. For the final design however a model was built of "Marco" resin in which the foundation deformabilities of Table II were taken into consideration.

In the first preliminary design (Study I) a reduction of the cross section at the heel and the construction of a socket to support the dam had been

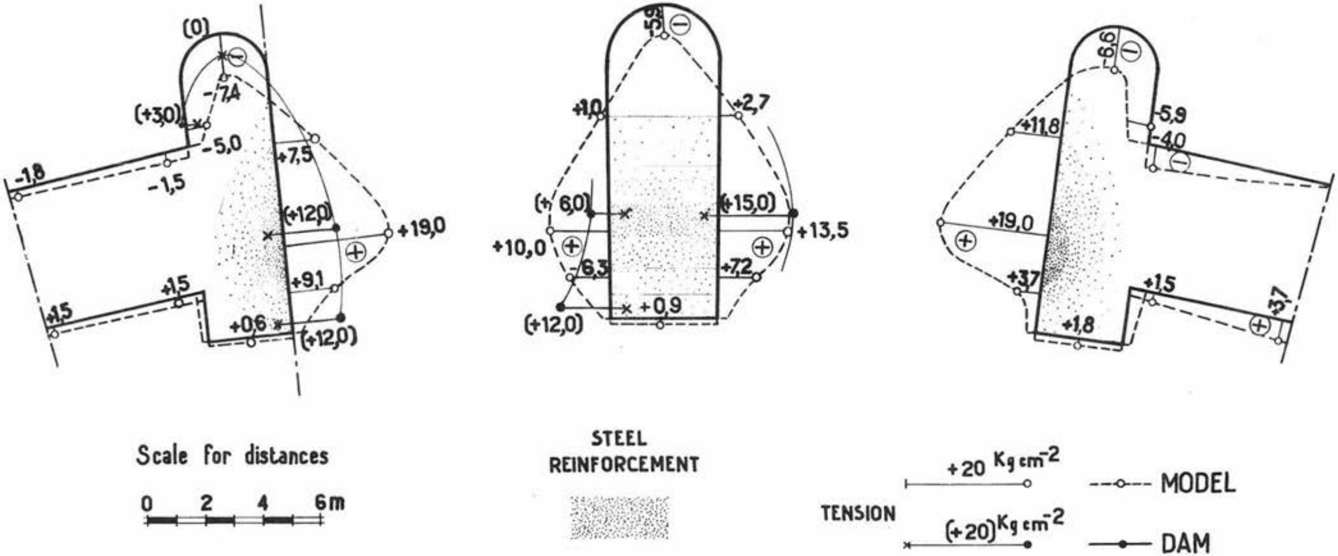


FIG. 8 –VENDA NOVA DAM. Vertical stresses in a horizontal section of the spillway openings.

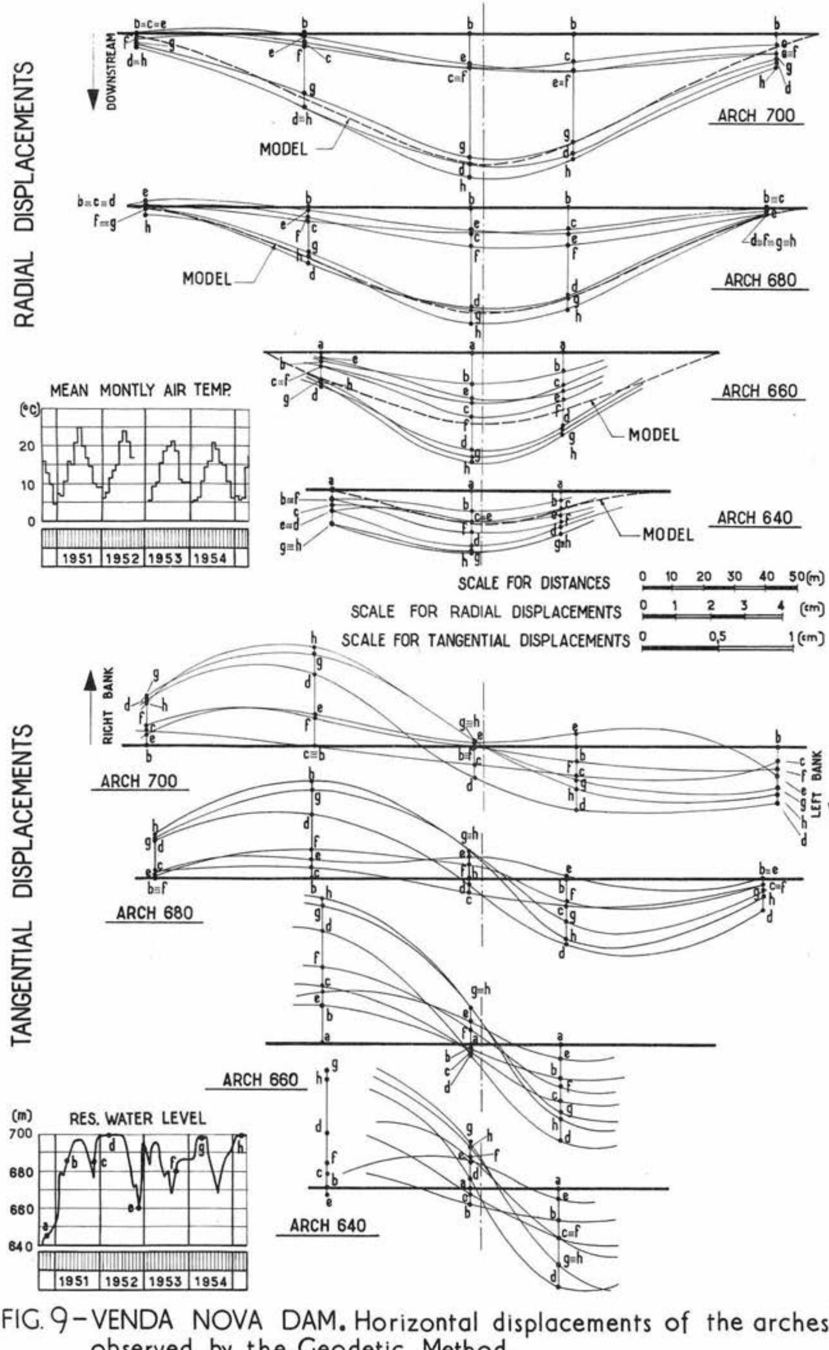


FIG. 9 – VENDA NOVA DAM. Horizontal displacements of the arches observed by the Geodetic Method.

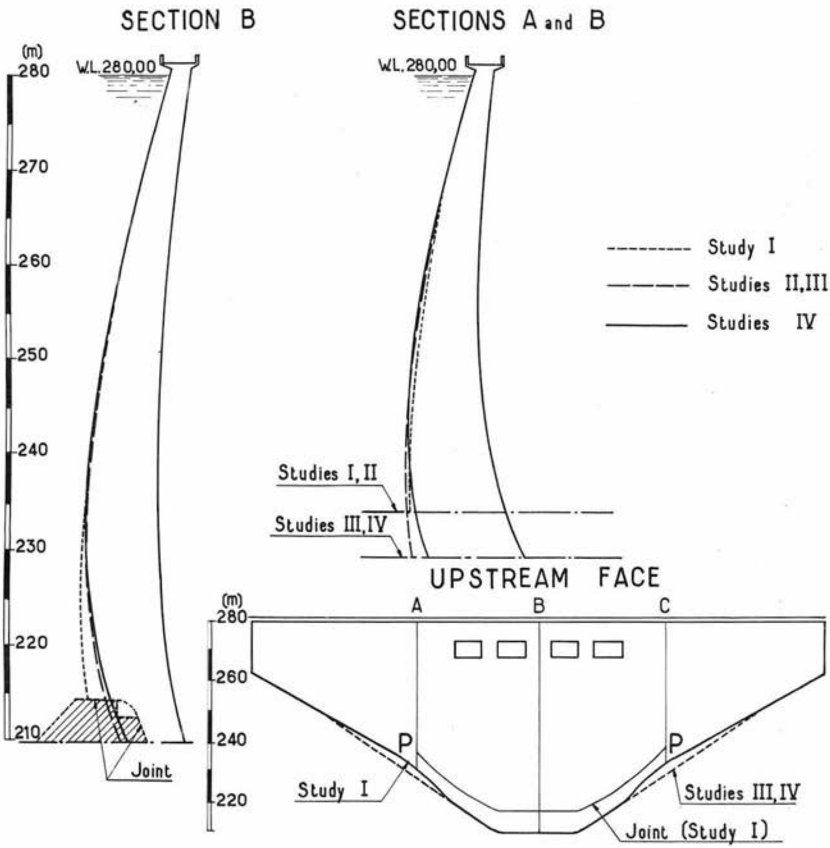


FIG. 10 — SALAMONDE DAM. Designs tested.

foreseen (Fig. 10). The joint between the socket and the dam would be closed by copper sheets and a gallery would be left inside for inspecting the joint. The idea was to give great flexibility to the base of the crown cantilever as it was supposed that the tensile stresses which would develop at the heel would be neutralised by the compression due to the weight of the dam, which would increase very much when the joint opened. This conception was based on a simplified calculation with radial adjustments of the crown cantilever and seven arches.

In the subsequent studies the joint was eliminated and the lower part of the cantilevers was given a greater overhang at the upstream face. The shape of the downstream face remained the same. In Studies II and III the shape of the dam was the same, but in Study III the protuberance of the foundation profile was removed thus making it more regular and non-convex. A further alteration was introduced in Study IV to the upstream face so as to decrease the rigidity of the cantilevers at the base.

In Study I (Fig. 11) a vertical tensile stress of + 32 $kgcm^{-2}$ was developed in the heel at point A, whilst the calculation indicated a stress of + 200 $kgcm^{-2}$ at this point which would be compensated for by the weight of the dam when the joint opened. However, in view of the small value of the stresses measured it was concluded that the joint would not open but would rest on the socket. On the other hand in view of the small compression value observed in the model at the toe, point B, (11 $kgcm^{-2}$) the weight of the dam when the joint opened would produce very high tensile stresses at this point.

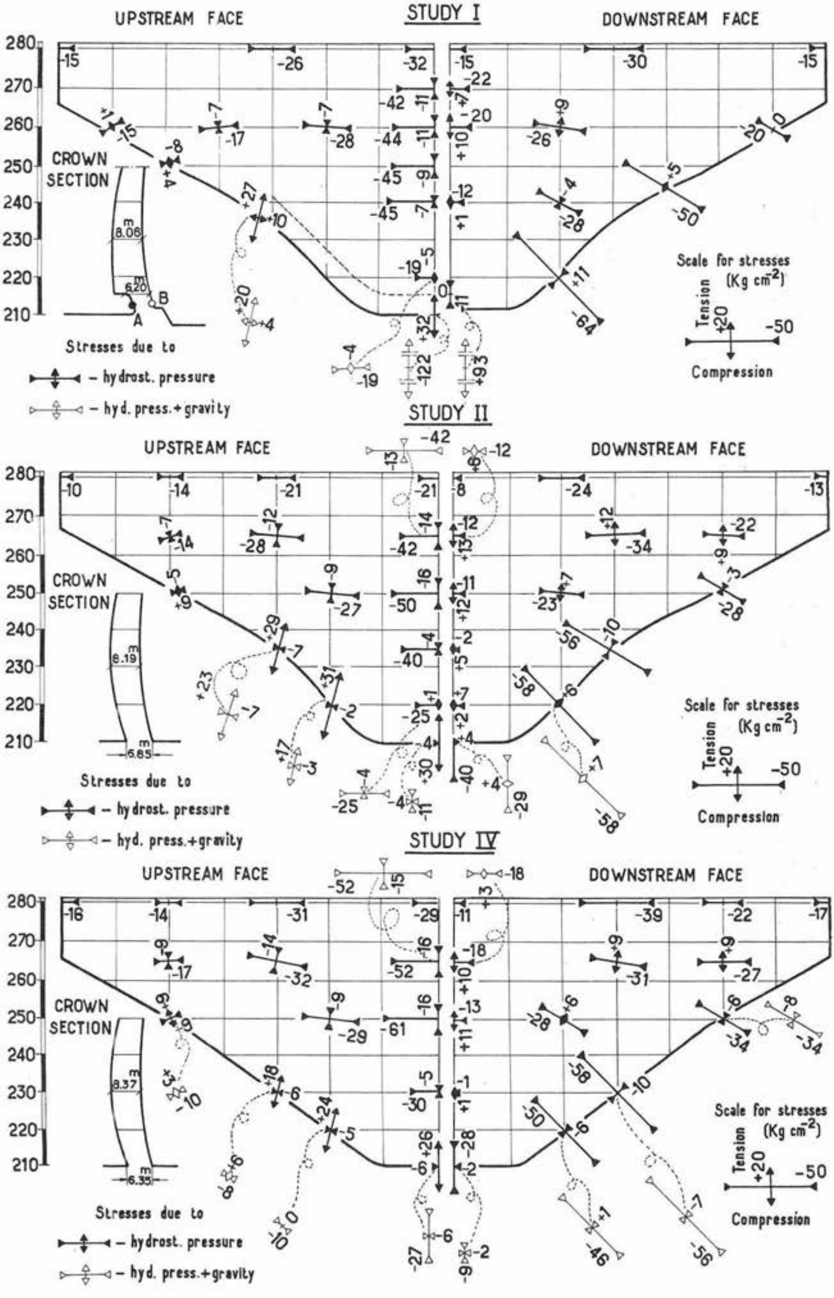
Study II showed also that there were still high tensile stresses under the combined action of the weight of the dam and hydrostatic pressure (Fig. 11), above all in the region where the foundation profile was convex.

In Study IV in which that convexity was eliminated the combined action of hydrostatic pressure and weight did not produce tensile stresses greater than 6 $kgcm^{-2}$, which was considered to be very satisfactory. The maximum compression in this case is 61 $kgcm^{-2}$ and is produced upstream at the crown of the arches.

As can be seen in Fig. 11 (Studies II and IV) the vertical tensile stresses produced by hydrostatic pressure in the downstream face are also very reduced by the weight of the dam. For Study IV a "trial load" calculation was made by adjusting radial displacements of five arches and seven cantilevers. As in the previous case wide divergence of vertical stresses in the base of the crown cantilever was recorded, the calculated stresses being greater than those given by the model. The maximum horizontal compressive stresses in the arches attain values little higher than those given by the models. These results confirm the insufficiency, already referred to, of the abridged "trial load" method for determining the stresses in the base of the dam, though it gives approximate values of the compressive stresses in the arches.

In this case, too, the considerable influence of the convexity of the foundation surface on the behaviour of the arch dams was verified. This study also showed how small alterations of thickness, easily reproducible in the models, can make an important contribution to the reduction of tensile stresses in the base.

In Fig. 12 the curves are plotted of radial and tangential displacements of five cantilevers of Salamonde dam obtained by the goedetic method on five different dates of the years 1953, 1954 and 1955. As this dam is very thin, it had already reached thermic equilibrium when the filling of the reservoir was begun. Thus the differences between the curves of displacements



Hydrostatic load at elevation 280,00 m

FIG. 11 –SALAMONDE DAM. Stresses from model studies.

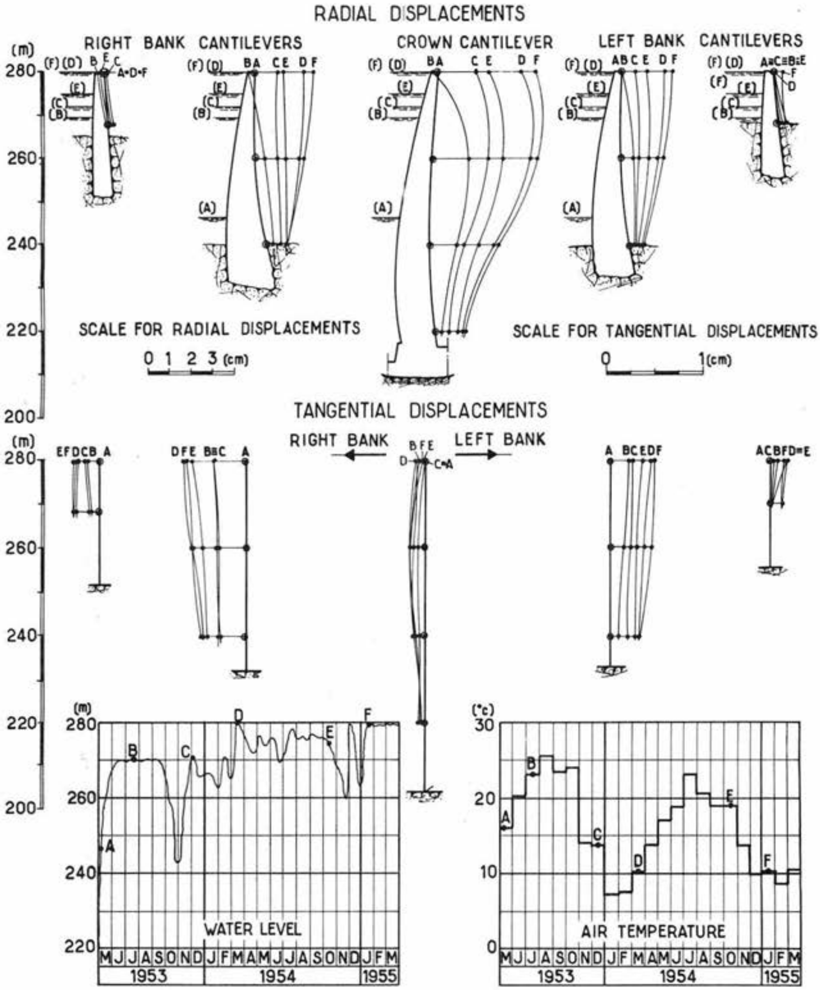


FIG. 12 — SALAMONDE DAM. Horizontal displacements of the cantilevers observed by the Geodetic Method.

corresponding to the same reservoir level are due to air and water temperature variations.

An analysis of the radial displacement diagrams of the dam (Fig. 12) shows that with the first water rise, dates A to B, the dam underwent downstream displacements except for the points in the crest. These points moved upstream because of the rise of air temperature which has an immediate effect, as the crest is only two meters wide. On dates B and C with the water practically at the same level there was a large fall in temperature which caused a considerable downstream displacement. When the water level reached its maximum (dates D and F) the dam experienced its greatest downstream displacements, though the points of date F were slightly below those of date D. As the concrete temperatures observed on dates D and F are equal, the displacement between these dates is probably due to foundation movement.

The diagrams of tangential displacements show that between dates A and B, that is, during the first loading of the dam, there was a displacement toward the right bank probably due to the settling of this bank. Subsequently the behaviour of the dam has been almost symmetrical.

In Fig. 13 the diagrams are plotted of the concrete length changes registered by three no-stress strain meters in the dam as a function of temperature. The three no-stress meters serve three groups of strain meters placed in the base of the crown cantilever, one at one meter from the upstream face (1u), another at a point half way through the dam (1) and the third at one meter from the downstream face (1d). The concrete in the region where the no-stress meters were placed was laid at about 20°C and it reached a maximum temperature of 43°C at the centre of the block. When the temperature reached 36°C the measurement of length changes in the concrete was begun. Once the temperature began to fall the length decreased at the rate shown in the longer branch of the diagram. This branch has a different angular coefficient from the first one, which corresponds to the temperature rise. The minimum temperature of about 8° C was reached and so too the minimum length. There were two further temperature cycles, which are represented in the two further branches of the diagrams.

The coefficients of thermal expansion corresponding to the branches representing the greatest temperature falls were calculated and the values obtained are very close to each other and quite acceptable (Fig. 13).

Cabril

Cabril dam (Figs. 1 and 14), the tallest Portuguese dam, is a thin arch structure, symmetrical and resting on very homogeneous granite foundations. The arches have variable thicknesses and the downstream face has fillets. After Study I, a foundation socket was designed which thickens the dam, makes the shape of the valley more regular and diminishes the stresses on the ground.

In the models of Study I it was found that although there were no tensions due to the combined action of hydrostatic pressure and weight, the compressions were relatively high (Fig. 15). In view of this, Study II had a slightly thicker base and the socket already mentioned. In the actual socket the compression stresses did not reach 30 $kgcm^{-2}$, though it was not very thick (Fig. 14). The increase in the volume of concrete between Studies I and II was only from 348,000 to 354,000 m^3.

In Study II the compressive stresses (Fig. 15) were only 53 $kgcm^{-2}$ and

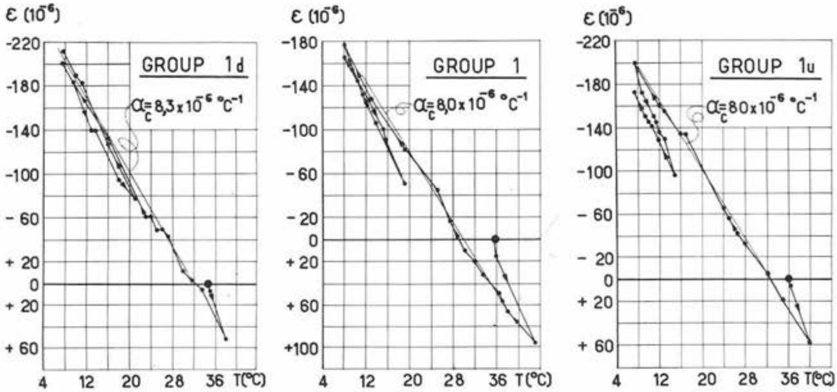


FIG. 13—SALAMONDE DAM. Strains versus temperatures measured by means of no-stress strain meters.

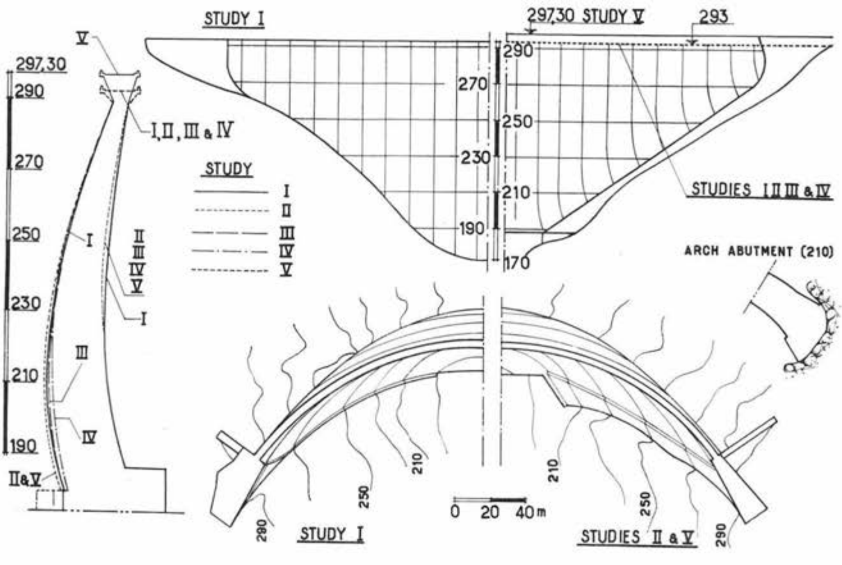


FIG. 14 —CABRIL DAM. Studies tested.

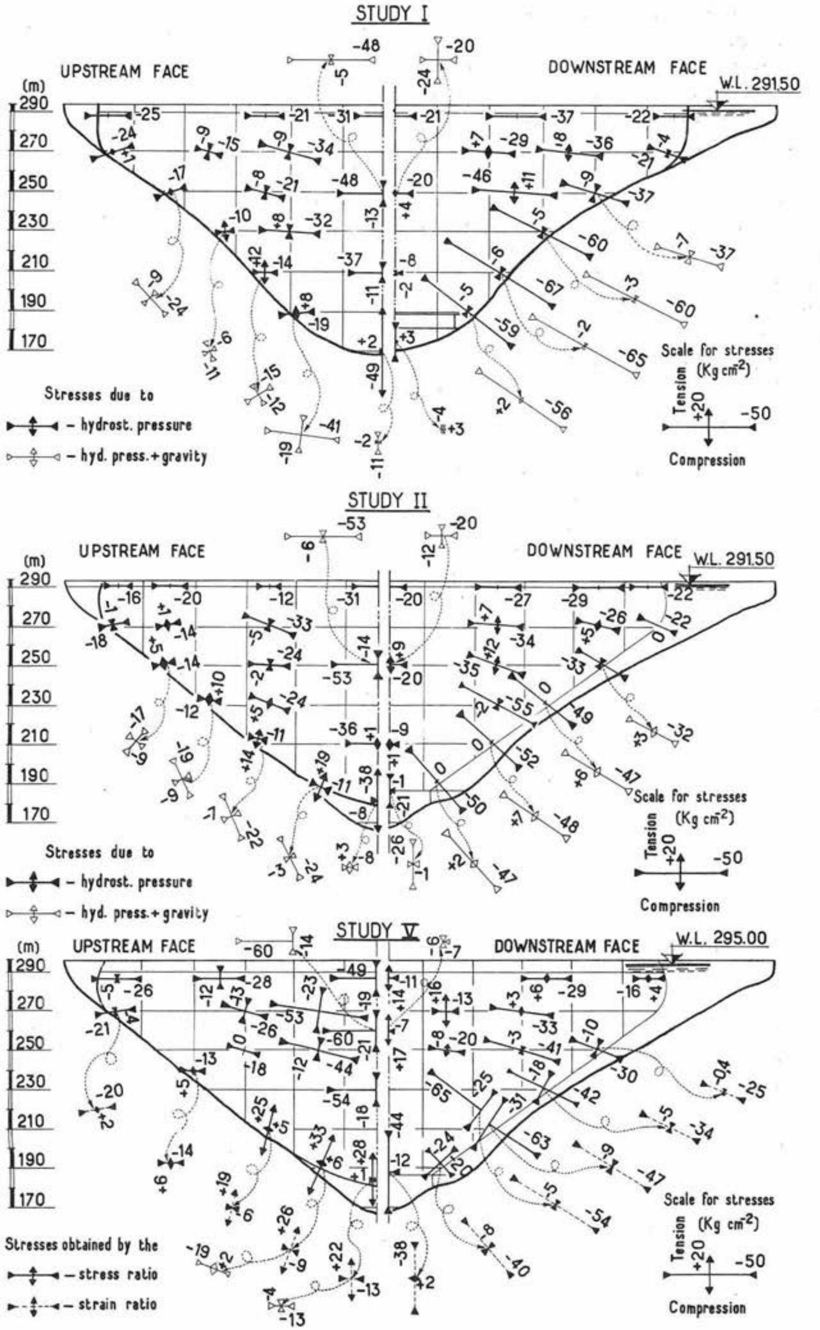


FIG.15– CABRIL DAM. Stresses from model studies.

hence did not reach the maximum allowed of 65 $kgcm^{-2}$. This led to two further studies on the same models (Studies III and IV) in which the thickness of the dam was reduced by cutting away material at the upstream face (Fig. 14). However, at this stage of the tests it was considered advisable to raise the level of the crest of the dam from 293.00 m to 297.30 m and maintain the shape of Study II below the level 290.m (Study V).

The models used for Study II and IV were built with a mixture of plaster of Paris and diatomite, for which Poisson's ratio is approximately that of the concrete, that is, about 0.20. Study V was carried out on an "alkathene" model, a plastic with good characteristics for building models[3,4] but whose Poisson's ratio is 0.5, i.e., considerably greater than that of concrete. In order to analyse the influence in the stresses of a high value Poisson's ratio, an "alkathene" model equal to that of Study II was first built in order to compare the results given by both materials. A comparison of the results showed that the values of the stresses were in agreement except along the abutment. For points there, satisfactory agreement was obtained by transposing the strains observed in the model to the prototype instead of the stresses.

After concluding these tests the crest of the "alkathene" model was raised to the level corresponding to Study V. Fig. 15 gives the values of the stresses obtained by transposing the stresses and, in the vicinity of the foundation, those obtained by transposing the strains and calculating the stresses in the prototype by means of its elastic constants $\underline{E}$ and $\underline{\nu}$. The maximum compressions did not exceed 65 $kgcm^{-2}$ and there were practically no tensile stresses from the combined action of hydrostatic pressure and weight, both in the heel of the dam and in the downstream upper part of the crown cantilever. This is due to the upstream overhang in the lower region and downstream overhang in the upper.

The radial and tangential displacements as well as the rotations were also measured in the models. For Study I, besides making one calculation with only radial adjustments for six arches and nine cantilevers, a complete "trial load" analysis was made, that is with radial, tangential and twist adjustments, for three arches and five cantilevers. The results of the first calculation gave maximum compressions in the arches similar to those given by the model, and stresses in the bases of the cantilevers and radial displacements greater than those given by the model. The complete calculation gave tangential displacements and rotations practically equal to those of the model[4] but the radial displacements and stresses were a little lower than those of the model.

For the final design (Study V) a calculation was made simply with radial adjustments of six arches and nine cantilevers. For the hydrostatic pressure the compressive stresses in the crown of the arches for this analysis were 60 $kgcm^{-2}$ at level of 230 m where the model gave 54 $kgcm^{-2}$, and the tensile stresses at the base of the crown cantilever were 30 $kgcm^{-2}$ where the model gave 22 $kgcm^{-2}$ based on the strain relation (Fig. 15).

The Cabril dam studies once again showed that the "trial load" calculation, only with radial adjustments, gives compressions in the arches which approximate to those of the models and stresses at the base of the cantilevers greater than those of the models. Note that this dam does not have any irregularities in shape.

The vertical displacement diagrams of the dam (Fig. 16) measured by precision levelling show that, during the first loading, the crest rose (dates $\underline{A}$ to $\underline{C}$). This rise is in good agreement with that found in the model; between

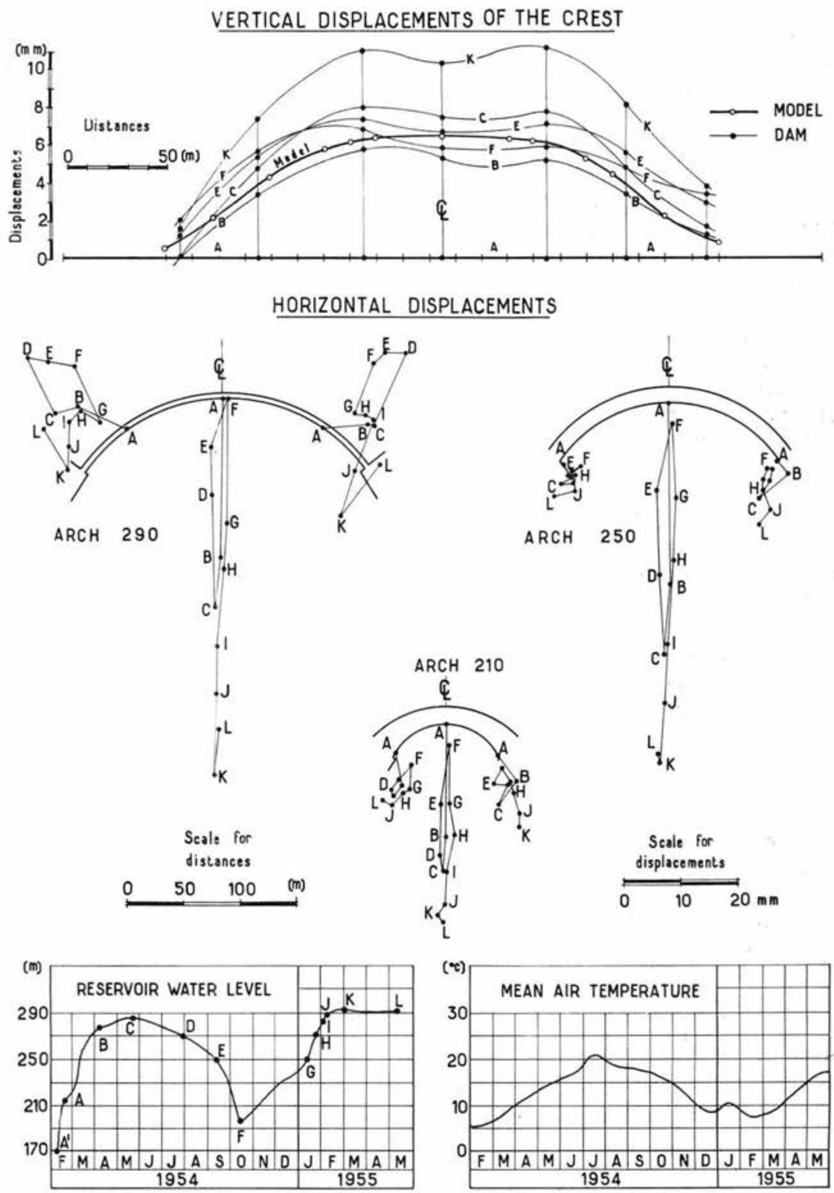


FIG. 16 – CABRIL DAM. Displacements observed by Geodetic Methods.

those two dates the effect of temperature variations was not apparent, as the lower 100 meters of the dam had a mean temperature drop of 1°C and the upper 30 meters a mean rise of 4°C (Fig. 17). The drop of water level between dates C and F did not cause the crest to return to its initial position even when taking into consideration the temperature variations and the rise of the abutments revealed by the end points of the diagrams. The rise of the water level between dates F and K caused a further rise of the crest. As however there was a mean temperature drop in the dam of 5°C the curve corresponding to date K should not be far from that corresponding to date F. It was therefore concluded from the observations of the vertical displacements that there must be another cause for the rise of the crest besides the hydrostatic pressure and temperature.

The analysis of the horizontal displacements (Figs. 16 and 17) shows that the dam has an almost reversible and symmetrical behaviour. Note, however, that from date A to date F, in spite of there having been a large temperature rise above level 230, the points observed in the crown cantilever did not go upstream of the positions occupied on date A. This is probably due to a displacement of the foundation.

The maximum reservoir water level was attained on the second filling (dates K and L), the greatest displacements being observed on date K at points at level 290 and 250 and on date L at points at level 210. This fact is due to the mean temperature of the concrete having risen by 10°C in the arch at level 290 between dates K and L, and having fallen by 1°C in the arch at level 250 and 2°C in that at level 210 (Fig. 17).

The lateral points of the crest of the dam (arch 290) underwent displacements strongly influenced by temperature variations. Between dates C and D, when the mean temperature of the crest rose by 10°C, these points were displaced upstream by the same amount as between dates A and C. With the fall of the mean temperature of the crest following date D till date K (Fig. 17) these points moved continually downstream in spite of important variations in hydrostatic pressure.

The lateral points of the arches 250 and 210, whose paths are similar to the paths of displacements observed on points of the rock near the dam,[(6)] show movements similar to those of the crown of the arches at the various levels.

In Fig. 17 diagrams are plotted for the displacements of the crown cantilever observed by the geodetic method against the reservoir water level. The curves of the mean temperatures of the concrete and the mean monthly air temperature also, against the reservoir level, are drawn in the same figure. The displacement diagrams, when taking the temperatures into consideration, reveal the general reversibility of the dam. Besides the irreversibility between dates A and F already mentioned, these diagrams also show an anomaly in the behaviour during the second filling. In fact on date H the points of the crown of the arches 250 and 210 were situated upstream to the positions occupied in date D, even though for these two dates the reservoir levels were equal and the concrete temperatures were lower in date H than in date D. This is probably explained by the final grouting of the joints which was carried out on date F, when they opened in the lower part.

Fig. 18 shows the diagrams of the temperatures measured in four cross sections of Cabril Dam at 1 meter from the two faces and half way through, from 1952 till 1955. In the same figure are traced the diagrams of the calculated temperatures for the intermediate points which agree very closely with the temperatures observed at these points.

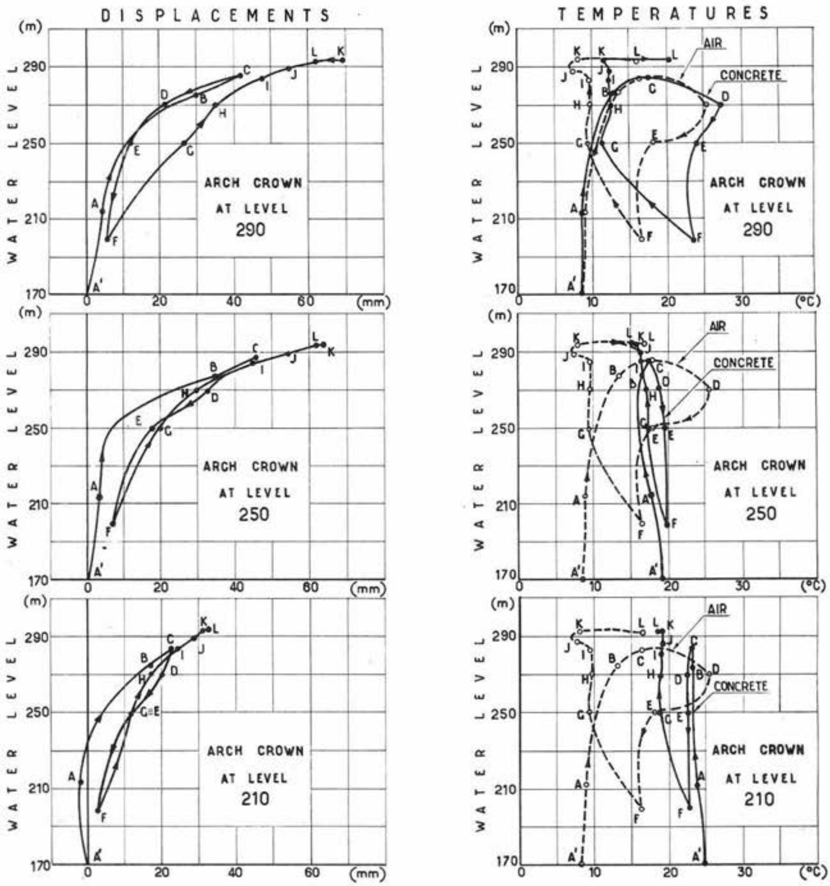
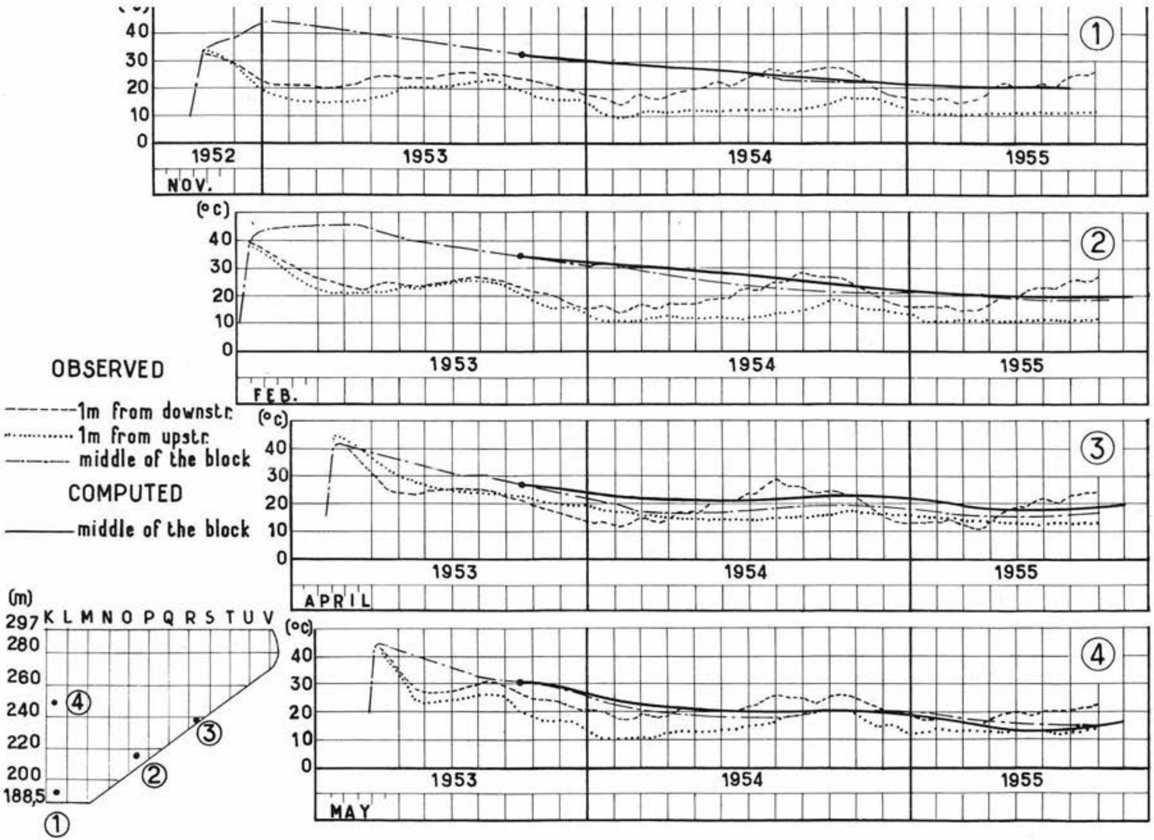


FIG. 17 - CABRIL DAM. Observed radial displacements and mean concrete temperatures of the crown cantilever.



OBSERVED
1m from downstr.
1m from upstr.
middle of the block
COMPUTED
middle of the block
(°c)
0
10
20
30
40
1952
1953
1954
1955
NOV.
FEB.
APRIL
MAY
①
②
③
④
(m)
297
280
260
240
220
200
188,5
K L M N O P Q R S T U V

In the lower part of the dam a low heat type of cement was used which developes the heat very slowly. The concrete of the dam had 250 kg of cement per cubic meter and 1.5 m lifts were poured at intervals of 4 days. As diagrams 1 and 2 show, the maximum temperature was only reached three months after laying. The upper half of the dam was built with normal Portland type cement and also with 250 kg per m^3. The maximum temperature was reached three days after laying and the heat of hydration was dissipated much more rapidly than in the lower part.

Fig. 19, 20 and 21 show the variations of the principal stresses at various points at 1 m from the faces between various periods. The temperature variations of the concrete at 1 m from the faces and at the mean surface are also plotted. The stresses were calculated from the strains using values for the modulus of elasticity and Poisson's ratio obtained in rapid tests on specimens of concrete from the structure. Ignoring the creep does not lead, in this case, to serious errors as the filling of the reservoir began when the greater part of the concrete was over one year old, and the loading and unloading cycles lasted a maximum of five months.

The comparison of the stresses measured between dates A' and C with those measured between dates C and F, taking into consideration the reversal of the loading, show large differences. Between A' and C (Fig. 19) the fall in temperature inside the dam, together with the rise in temperature of the faces caused large compressions in the superficial regions of the dam, except at the crest where probably there were not any large temperature gradients. This skin effect probably developed hydrostatic compressions at 1 m from the surface whose maximum value, taking into consideration the temperature variations measured along the thickness and the modulus of elasticity of the concrete, was probably around 15 $kgcm^{-2}$ at 1 m from the upstream face and 30 $kgcm^{-2}$ at 1 m from the downstream face.

Between dates C and F (Fig. 20) the decompressions are much lower than the values of the compressions previously measured between A' and C, as the temperature variations continue to be in the same direction and of approximately the same value, especially in the region below arch 260.

During the second filling, between dates F and K (Fig. 21), large falls in temperature took place in the faces and relatively small ones inside. These variations caused maximum hydrostatic tensile stress of about 30 $kgcm^{-2}$ at 1 m from the downstream face and 10 $kgcm^{-2}$ at 1 m from the upstream face which relieved the compressions at the faces produced by the water pressure.

Fig. 19, 20 and 21 also have the diagrams of the horizontal stress variations along the thickness in the crown cantilever at level 250, and the diagrams of the vertical stress variations at the base of the same cantilever. The distribution of the stresses is far from the linear, which must be due to the temperature variations.

It should be noted that the agreement between the directions of the principal stresses observed in the various dates and those given by the models shows that the skin effect produced by the temperature variations is a state of hydrostatic stress.

A comparison of the maximum stresses given by the models with those observed on the structure gives a very good agreement when the measured values of the elastic constants and the differences between the mean temperatures of the concrete (Fig. 17) and the temperature at the point are taken in full consideration. For example the maximum compressive stress (point 6u) given by the model, for the water at level 290, is 50 $kgcm^{-2}$, whilst on the

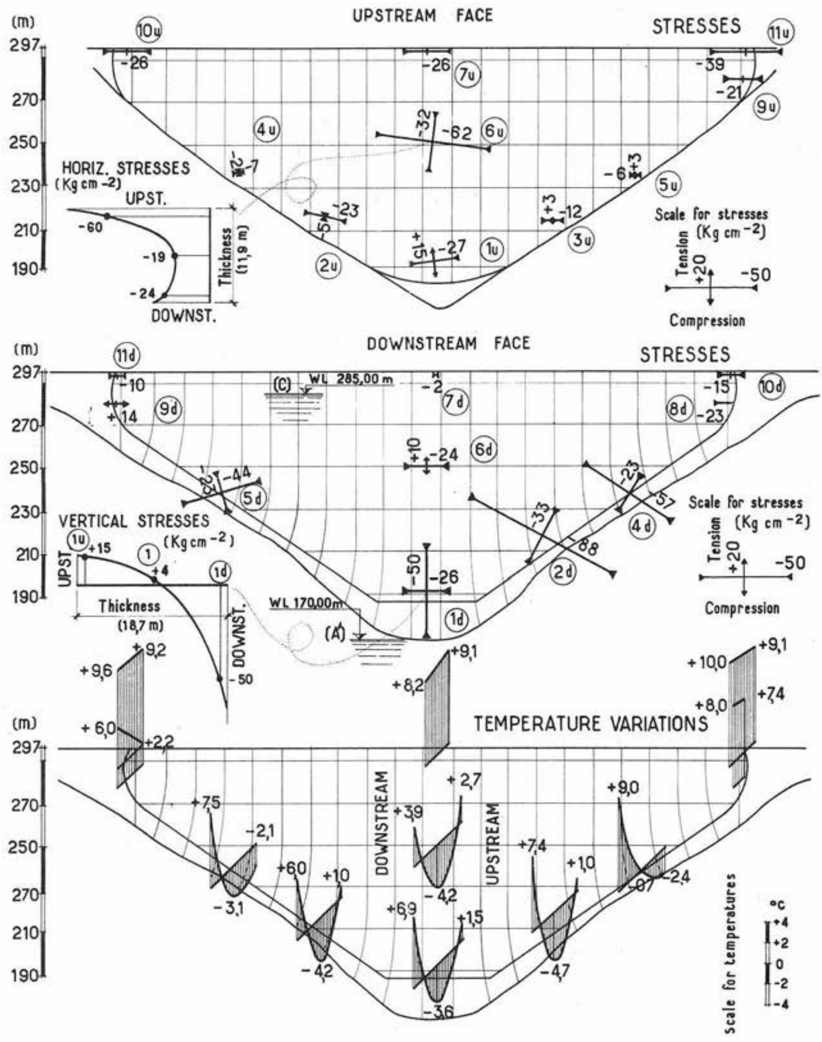


FIG. 19 – CABRIL DAM. Observed stresses and temperatures during the first loading (Date A' to C).

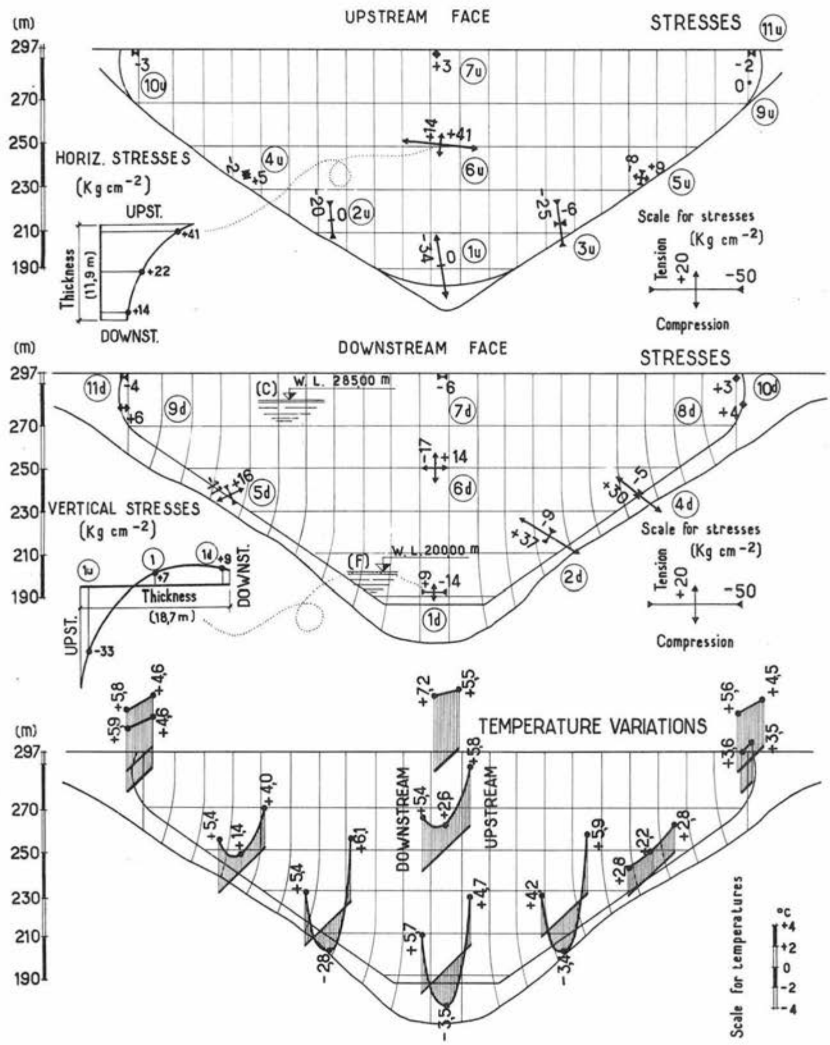


FIG. 20—CABRIL DAM. Observed stresses and temperatures during the first unloading (Date C to F).

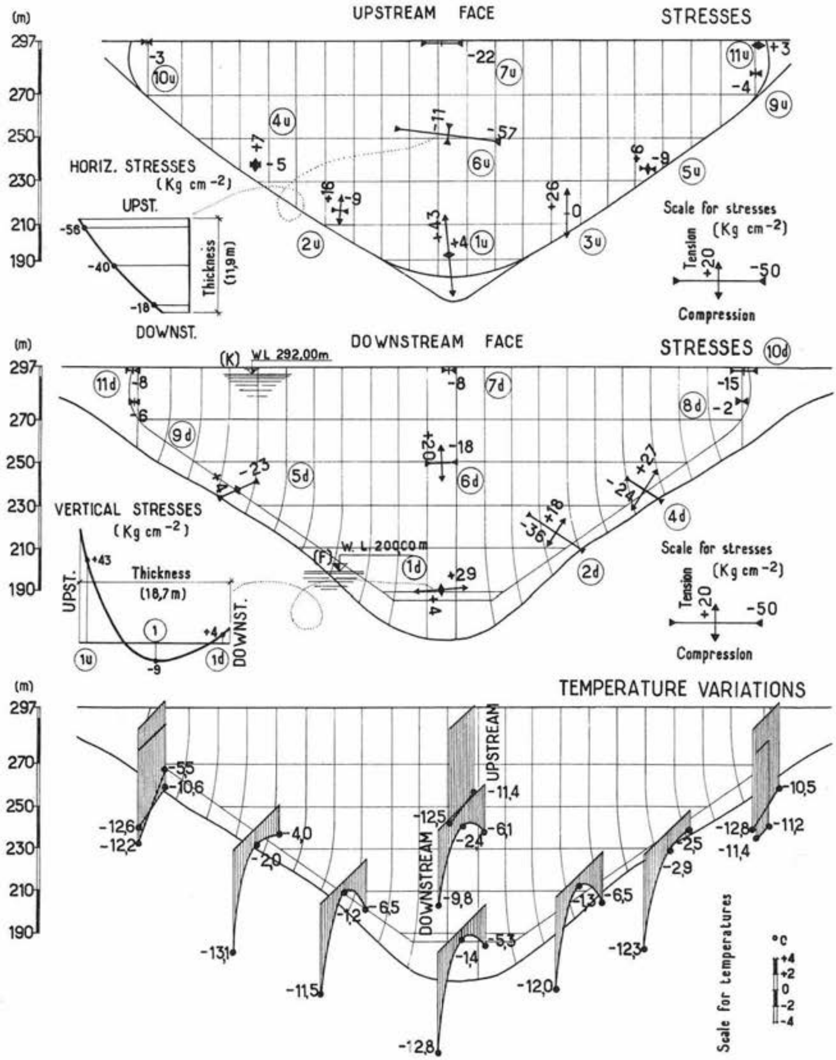


FIG. 21 —CABRIL DAM. Observed stresses and temperatures during the second loading (Date F to K).

prototype between dates A' and C the hydrostatic pressure produced a stress of 62 - 12 $kgcm^{-2}$ = 50 $kgcm^{-2}$; between dates C and F 41 + 9 = 50 $kgcm^{-2}$ and between dates F and K 57 - 6 = 51 $kgcm^{-2}$. The maximum downstream compression at level 220 (point 2d) given by the model is 60 $kgcm^{-2}$ whilst the prototype gives the values 88 - 14 = 64 $kgcm^{-2}$, 37 + 18 = 55 $kgcm^{-2}$ and 36 + 27 = 63 $kgcm^{-2}$.

Caniçada

Caniçada Dam (Fig. 1) has a similar shape to Salamonde Dam and its design was greatly helped by the studies made on the latter. After the first model studies it was considered advisable to increase the thickness at the abutments of the lower arches. The final model study led to the conclusion that the shapes tested were satisfactory.

Fig. 22 shows one of the models of this dam with the isostatic lines on the two faces obtained by a brittle lacquer, and Fig. 23 shows the arrangement of the electrical strain gauges on the downstream face and the rubber sack in which the mercury rises so as to reproduce the hydrostatic pressure.

The comparisons between the stresses calculated by the abridged "trial load" method and those from the models also led to conclusions analogous to those already mentioned.

The diagrams of radial displacements in the dam (Fig. 24) obtained by the pendulum and the geodetic method agree satisfactorily, the difference between the two diagrams being due to the displacement of the reference point P of the pendulum. The displacements given by the model agree well with those observed on the prototype between dates C and D when there was little temperature variation. The values of the displacements given by calculation agree well with those given by the model, only diverging in the upper part.

Fig. 25 gives diagrams of the stresses measured upstream, halfway through and downstream at the base of a cross section of the Caniçada dam by means of Carlson stress meters. These meters were placed at 45° so as to receive the forces normal to the foundation surface. Stress meter 2 shows an initial compression of about 13 $kgcm^{-2}$ due to the rise in concrete temperature as a result of the development of the heat of hydration, whilst stressmeters 1 and 3 registered slight tensile stresses. With the fall in temperature and growth of the block, meter 1 started to register compressive stresses and meter 3 to register tension due to the weight of the concrete, as the block leans upstream. However the tension in meter 3 is counterbalanced by the compression due to the cooling of the inside of the dam. This cooling tends to cause tension as detected by stress meter 2, which however is counterbalanced by the compression from the concrete weight. When the filling of the reservoir began, meter 1, which was already registering decompression (August 1954) due to temperature fall on the faces of the dam in relation to the inside, showed increased decompression due to the effect of rise in water level. Meter 3 shows compression due to this rise in water level very clearly and meter 2 has intermediary values. At the beginning of May 1955 the temperature of the concrete at the faces began to rise and immediately the compression at meters 1 and 3 did so too, whilst meter 2 registered decompression for the same reason. With decrease in water pressure, which took place in October 1955, meter 3 showed decompression. This effect was accentuated by a fall in temperature of the downstream face which occured at the same time. Meter 1 registered an increase in compression due to fall in

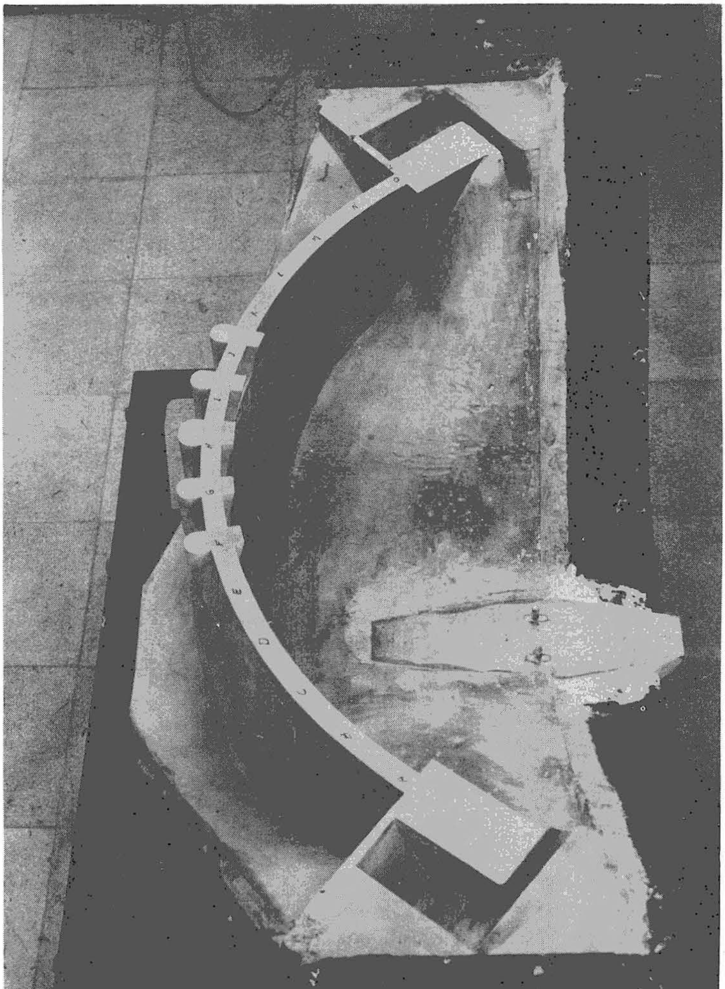


FIG.22-CANIÇADA DAM. Isostatic lines at the faces of the model

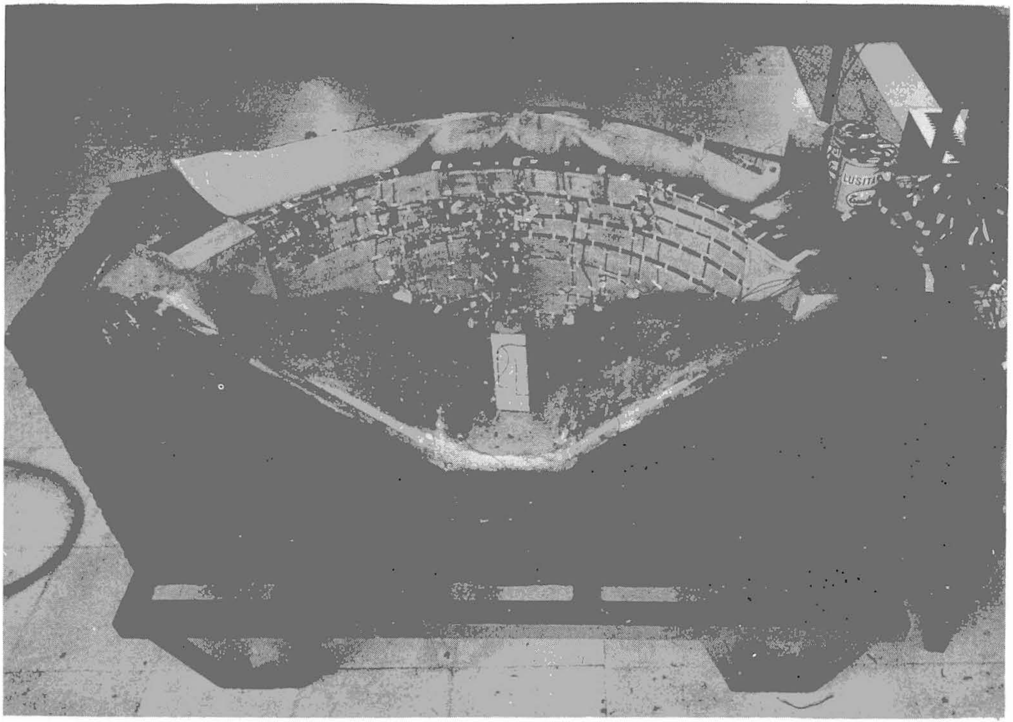


FIG. 23-CANIÇADA DAM. Strain ganges on the downstream face

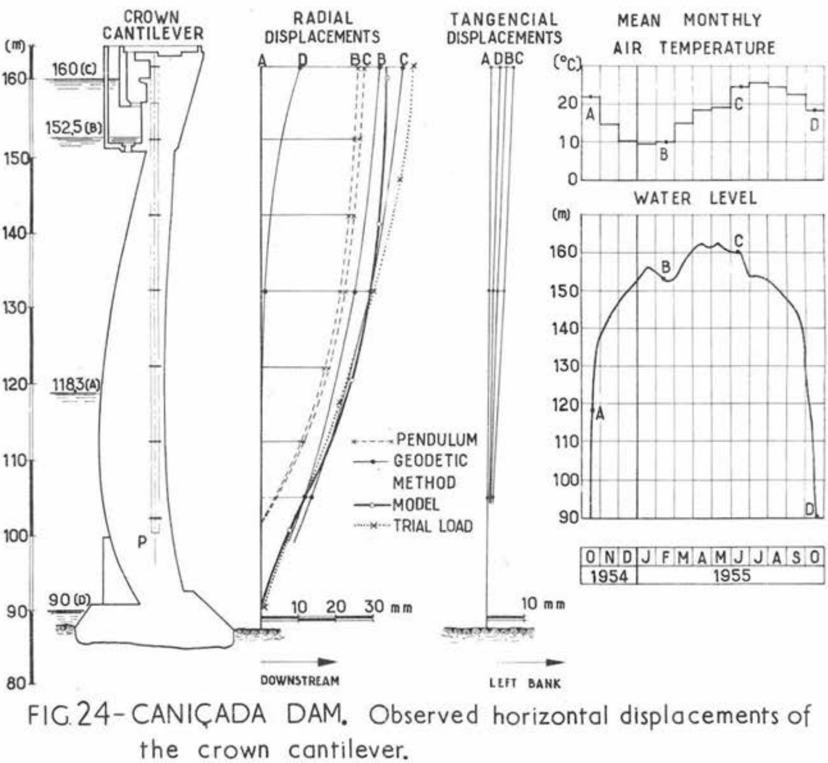


FIG. 24 - CANIÇADA DAM. Observed horizontal displacements of the crown cantilever.

water level. When the water rose again, meter 3 registered further compression and meter 1 decompression. Meter 2 also registered compression which was mainly due to the cooling of the faces rather than to hydrostatic pressure.

The model tests for this dam gave - 50 $kgcm^{-2}$ and + 18 $kgcm^{-2}$, for downstream and upstream respectively, at the same points and direction as the stress meters. Such stresses are much greater than those given by the stress meters, which were - 17 $kgcm^{-2}$ and + 10 $kgcm^{-2}$, between October 1954 and April 1955 when the reservoir was filled. The very much lower values given by the stress-meters can, in part but not completely, be explained by local thermal effects.

Before beginning the construction of the dam a study was made to estimate temperatures as it was wanted to accelerate the construction by laying 2 m lifts at intervals of four days. The proportion of cement used was 250 kg per m^3. This cement at the end of 3 days developed 63 calories per gram, at 7 days 71 and at 28 days 84 calories. Fig. 26 shows diagrams of recorded and calculated temperature for two cross sections of the dam which agree satisfactorily.

Bouçã

The design of Bouçã dam (Fig. 1), like the previous ones was studied in detail by models. It is a dam with accentuated double curvature and which can discharge a flow of about 2300 m^3 per sec. over the crest. The arches have variable thickness and have no fillets. There is a socket on the downstream side, as in Cabril Dam. Fig. 27 shows one of the models being worked.

The shape of the dam is so favourable that if the thickness was reduced to obtain a maximum stress of 65 $kgcm^{-2}$ it would be too thin for mass concrete construction.

Fig. 28 shows the isostatics for the two faces due to hydrostatic pressure, obtained by "stress-coat" and the values of the stresses due to this loading. For some of the most important points the values of the stresses due to combined effect of hydrostatic pressure and weight of the concrete are given.

The dam which is being observed for its first loading has not been grouted yet. Although a flood of 1500 m^3 sec^{-1} has flowed over the structure, when the upstream level reached 179.00 m, no anomaly has been observed in its behaviour.

Fig. 29 gives the stresses observed for the first filling at level 174. These stresses were obtained exactly as in the Cabril Dam. A certain asymmetry was recorded in its behaviour which was also verified in the displacements. This asymmetry appears to be caused by the different exposure to the sun of the two banks.

CONCLUSIONS

The experience obtained in Portugal leads to the conclusion that the thin arch dams, in relation to all other types, constitute the most economic and safe solution whenever the foundation characteristics permit it and the valley is not excessively wide. This last limitation is not, however, so common as is supposed. With regard to the foundations, too, the requirements are not

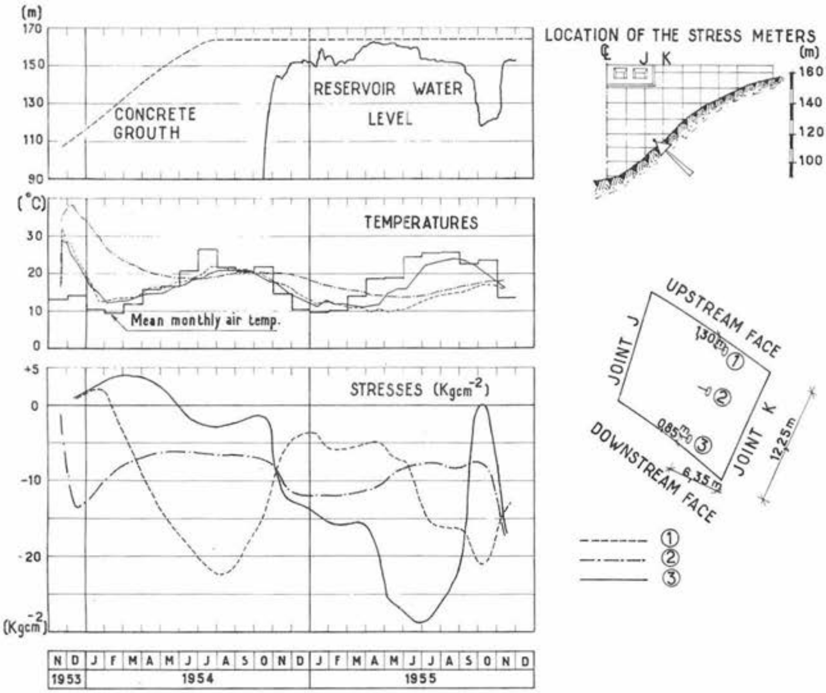


FIG. 25—CANIÇADA DAM. Stresses measured by means of Carlson stress meters.

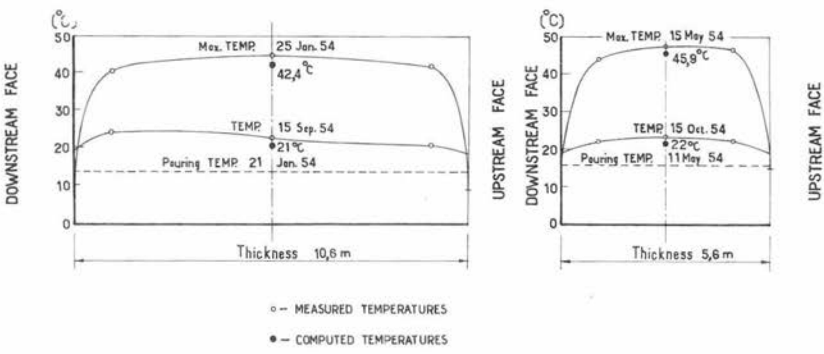


FIG. 26—CANIÇADA DAM. Measured and computed temperatures at two sections of the dam.

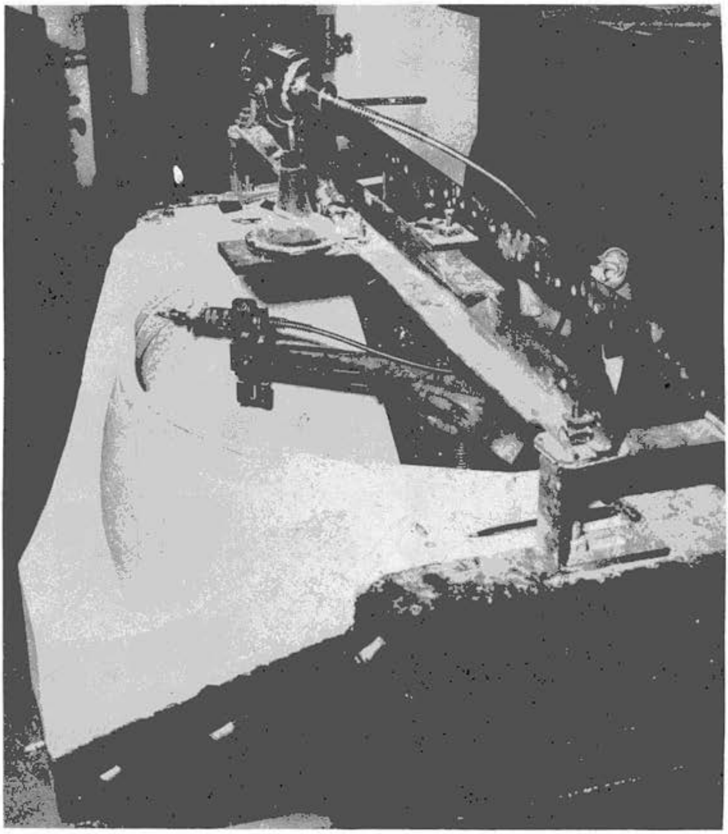


FIG. 27 – BOUÇÃ DAM. Construction of the model

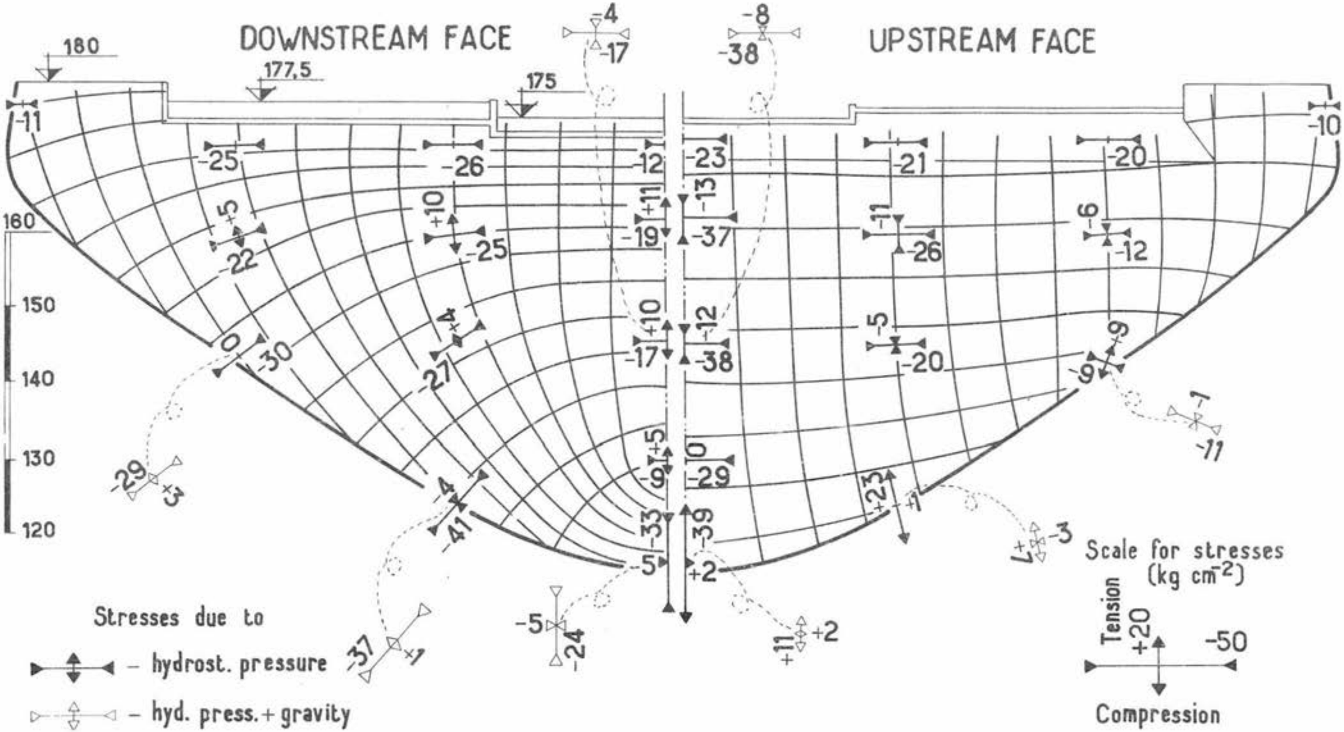


FIG. 28 – BOUÇĂ DAM. Stresses from model studies (Mercury load at elev. 177,5).

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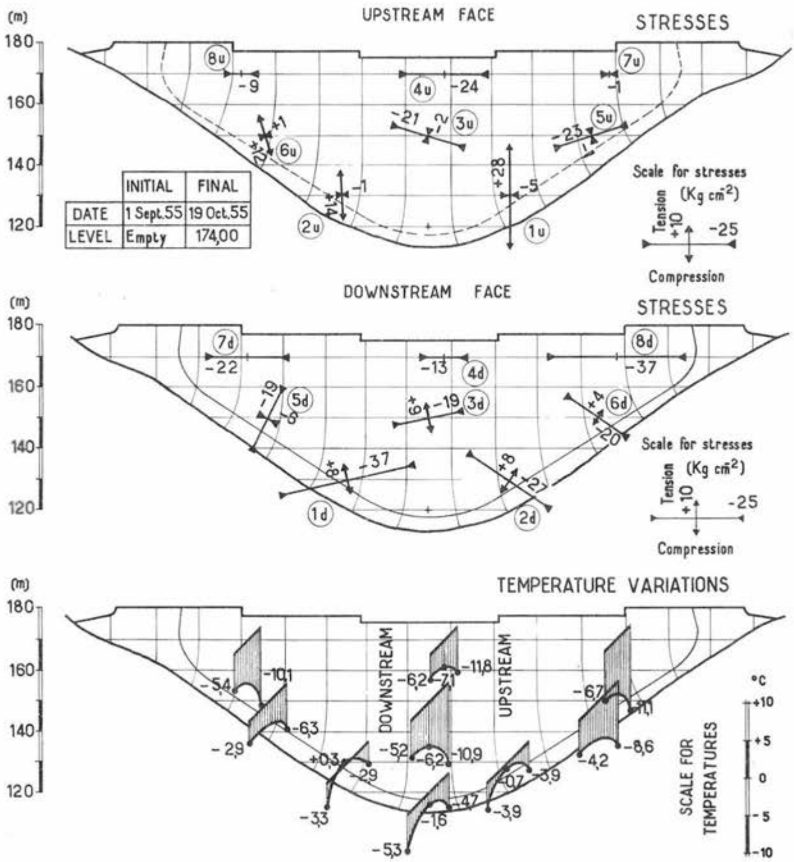

FIG. 29 –BOUÇÃ DAM. Observed stresses and temperatures during the first loading.

very exacting. In fact thin arch dams can be built on very deformable rocks. Only tests carried out on site over large volumes of rock can give information on the deformability of the foundations.

When the shapes, which depend on the site conditions, are carefully studied the arch dams can be very thin, veritable shells in fact. However, it is not advisable for the thickness to be less than certain limits both for reasons of construction and for ensuring the watertightness of the dam. In Portugal 2 m is being considered as a minimum thickness.

With the progress made in model analysis it has been possible to study by means of models the stresses due to hydrostatic pressure so as to obtain, rapidly, accurately and economically, the most suitable shapes for each case. The materials used in the models should obey conditions of mechanical similarity so that the stresses may be known with the necessary accuracy. The influence of foundation deformability, of local thickening, possible development of cracks or any other singularity can be studied by means of models.

The complete observation of the behaviour of the Portuguese dams has made it possible to control their safety and, above all, to check the value of the analytical and experimental methods used in their design. Furthermore it makes it possible to understand the influence of certain factors better, such as temperature variations, and to investigate others, such as movements of the banks. A better understanding of the properties of the materials and foundations is also obtainable by observation. It is recognised that a large number of different observations taken regularly is indispensable to be able to interpret the measurements. The most important of these measurements are dam and ground displacements, strains and stresses, temperatures, joint openings, variations in concrete volume and uplift.

As a general rule it can be said, that whenever it has been possible to separate the effects of the various loadings, the agreement between the results of model studies and the observations on the structure has been extremely good. This is not the case with the results obtained from calculations which at times are considerably different from those of the observation of the structure. The "trial load" method when applied with radial adjustments only, and in simple cases, that is, when there are no singularities, supplies values of compressive stresses in the arches which are near those given by the models, but the tensile stresses at the bases of the cantilevers, given by that method, are exaggerated.

As a result of the studies and observations of the various dams a number of conclusions about the behaviour of arch dams can be presented:

The foundation movements of the dams can result from other causes than the forces applied by the dam to the foundation, as is the case of the upper part of the right bank of the "Cantelo do Bode" dam. These movements can have very great effect on the behaviour of the structure.

Singularities in the foundations should be avoided. Normally convexities can increase the stresses both tensile and compressive. It is therefore always advisable to investigate the influence of singularities and to decide to what degree they can be permitted, by means of model studies.

Special attention should be paid to the stresses in the faces of openings in arch dams as important tensile stresses can develop there.

The thermal phenomena of the dams are complex but can be calculated. The dissipation of the heat of hydration causes considerable compression at the faces. If such compressions are not excessive they can contribute to the watertightness of the structure. However, once the heat dissipation causes joint openings, the best means for obtaining suitable conditions for grouting in a short time is by artificial cooling of the concrete.

The displacements due to loss of heat and external temperature variations are very important. In the upper parts of the thinner dams they may reach values comparable to those due to hydrostatic pressure.

The external temperature variations produce a skin effect which consists of a hydrostatic state of stress parallel to the faces of the dam. The values of these stresses at any moment can be estimated by taking the difference between the temperature for the point in question and the mean temperature along the thickness on which the point lies. The agreement between the results of the stresses obtained on models and those on the actual dams, after making this correction for stresses of thermal origin, is remarkable.

ACKNOWLEDGMENTS

The various investigations described in this paper were possible due to the collaboration of the technical staff of various Portuguese organizations among which we would like to mention "Direcção Geral dos Serviços Hidráulicos," "Comissão de Fiscalização das Obras dos Grandes Aproveitamentos Hidro-Eléctricos," "Hidro-Eléctrica do Zêzere" and "Hidro-Eléctrica do Cávado."

Engineers of the Dams Studies Section of the Laboratory who contributed to these studies are O. V. Rodrigues, A. F. de Lemos, J. M. R. Neto, J. L. Fialho, J. P. Rodrigues, M. Q. Guerreiro and Maria Emília C. e Matos.

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LIST OF FIGURES

Discussion of
"ARCH DAMS: DESIGN AND OBSERVATION OF
ARCH DAMS IN PORTUGAL"

by M. Rocha, J. Laginha Serafim, and A. F. da Silveira
(Proc. Paper 997)

CORRECTIONS.—On page 997-37, on line 3, the equation 57 - 6 = 51 $kgcm^{-2}$ should be changed to 57 + 6 = 63 $kgcm^{-2}$. On this same page, on line 5, the equation 88 - 14 = 64 $kgcm^{-2}$ should be changed to 88 - 24 = 64 $kgcm^{-2}$. In Fig. 7, Studies I and II, Upstream Face, at El. 640 on the left side the value -23 should be changed to +23. In Fig. 11, Study I, Upstream Face, at El. 210 the stress -122 should have the direction of the arrow reversed. In Fig. 15, Study I, Upstream Face, Lateral Cantilever, at El. 230 the term +8 should be changed to -8. In Fig. 15, Study II, Upstream Face, Crown Cantilever, at El. 180 the term -38 should be changed to +38. In Fig. 15, Study V, Downstream Face, Lateral Cantilever, at El. 250 the term -8 should be changed to +8. In Fig. 20, Stress at Downstream Face, Crown Cantilever, at El. 250 the term -17 should be changed to +17. In Fig. 20, Temperature Variations, Crown Cantilever, at El. 250 the term +2, 6 should be changed to +1, 6.

ROBERT E. GLOVER,[1] M. ASCE.—In this paper the authors present a valuable series of correlations among the results of analytical studies and model studies for arch dams and observations of the prototype structures under load. They find a very favorable comparison between the model test results and prototype stresses after the prototype observations have been corrected for stresses, due to thermal changes, which were not present in the models. Correlations with incomplete trial load analyses, based upon a radial adjustment only, showed arch stresses from ten to twenty percent higher than those obtained from the models and gave cantilever tensile stresses at the base which were much higher than those found in the models. A complete trial load study for the important Cabril Dam appears to have given a much better correlation.

These relationships are as they should be since a radial adjustment alone does not fulfill the basic requirements for a proper solution of the stress and strain distribution in a structure. These requirements are (1) that every element of the structure should be in equilibrium under the stresses and forces which act upon it., (2) that every element of the structure must deform in such a way that it continues to fit with its neighbors on all sides as the structure passes from the unstrained state to the strained state and (3) that the appropriate boundary conditions must be met. The Kirchhoff uniqueness theorem[2] contains a proof that there is only one stress distribution capable of meeting these requirements. A radial adjustment alone meets

1. Research Engr., U. S. Bureau of Reclamation, Denver, Colo.
2. The Mathematical Theory of Elasticity, by A. E. H. Love., Fourth Edition-Cambridge University Press-1927-Paragraph 118.

2. In the cantilever elements there is a tendency toward increased compression at the upstream face and a corresponding decrease of stresses at the downstream face, especially in the lower elevations of a dam where the effects of twist are invariably greater.

3. In the central part of a structure, reduction of both radial deflection and stresses is general. The changes of stresses at and near the crown point of arch elements are generally small in comparison with greater reductions at the abutments. At the abutments the tendency is toward an increase in compression at the extrados and a decrease in compression at the intrados while at the crowns of the arches near midheight the opposite effects are usual.

4. In dams where the abutments have been sharply thickened by the addition of fillets, the stresses in the arch abutments will increase considerably due to tangential shear effects. It is impossible to compute these shear effects by means of a radial adjustment of deflections only, but they can be estimated by comparison with the computed effects from other comparable studies.

The Bureau has been able to save much time, effort, and expense in design costs by the judicious application of the above generalities to obtain final or close to final design dimensions. The conjugate points of the arches and cantilever elements representing the structure of a particular design are adjusted into radial deflection agreement by trial load process. The initial and succeeding design dimensions are altered to yield approximate desired stresses in accordance with the above characteristics applied to the results of analyses made to account for only the radial adjustment of deflections in the arches and cantilevers. The final analysis is made to complete the stress study and is usually made with a simultaneous adjustment of radial, tangential, and twist effects on representative elements of the structure.

The cost of making the analyses of stresses sometimes amounts to a large share of the total cost of designing an arch dam. The costs of analyses vary with the complexity of the method used, the number, time, and pay scale of the people employed, and to a lesser degree with the cost of materials, equipment, or other services involved. Contrary to the apparent experience of the Laboratoria Nacional de Engenharia Civil, Portugal, the Bureau of Reclamation has found that design of arch dams by trial load methods yields the most reliable results in less time and cost than by model tests. The question of reliability of results has heretofore been involved in the Bureau's experience because the matter of model testing has not been as conclusive by comparison with computed and observed results as has Portuguese and other European experience along these lines. In verifying the fundamental accuracy of new and existing theories, the use of models took an important part in the solution of many hydraulic and structural problems in the designs of Hoover, Grand Coulee, and other Bureau dams. In contrast to the hydraulic models, which provide direct empirical data, the principal function of the structural models of these dams was to furnish a check on analytical methods of design. Although considerable information which could not be readily obtained by analytical methods was derived and used in design from the arch dam model tests, this was only incidental to their use in determining the adequacy of the trial load method of analysis. From this viewpoint, the method of applying results of structural model tests of dams by the Bureau of Reclamation differed somewhat from the use of other types of structural models.

Now, however, the authors have contributed valuable evidence to the profession which demonstrates that with careful simulation of all known dimensions, loading conditions, mechanical and physical properties of the dam and foundation, and by the skillful employment of testing techniques, model tests may be used as a reliable basis for designing an arch dam.

It is not the purpose of this discussion to unfavorably compare or advocate the use of one means of analysis over another for arch dam design. The writer believes that whenever model tests are to be made the basis of determining stresses or movements for design, that they should always be checked by proven analytical methods so far as possible.

The writer is also interested in exchanging ideas regarding the time and expense involved in designing by use of the trial load method and by the use of model tests as described in Paper No. 997.

With an experienced team of engineers, the Bureau of Reclamation expends an effort averaging 120 man-days to analyze one loading condition on an ordinary arch dam design with variable thickness elements by adjusting to account for radial movements only. By ordinary arch design is meant that the layout is considered sufficiently symmetrical and that no unusual loading, structural, or abutment conditions exist which would prohibit the structure from being analyzed by symmetrical conditions. If the analysis is carried to completion, including the total effects of tangential shear and twist, an additional 280 man-days, average, is required for this further effort. It costs approximately 40 percent more than the above figures to analyze each additional loading condition for the same layout.

Discussion of
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OF ARCH DAMS IN PORTUGAL"

by M. Rocha, J. Laginha Serafim, and A. F. da Silveira
(Proc. Paper 997)

M. ROCHA,* J. LAGINHA SERAFIM** M. ASCE, and A. F. DA SILVEIRA.***—The writers thank Robert E. Glover and Fred A. Houck for the attention and appreciation shown their paper.

In the discussion of Robert E. Glover and M. D. Copen's paper "Trial Load Studies for Hungry Horse Dam" (Proc. Paper 960) and in the closing discussion of the paper "Model Tests, Analytical Computation and Observation of an Arch Dam" (Proc. Paper 696) presented by the writers, an opportunity was presented for giving their opinion on the advantages and setbacks of the trial-load method. It is not intended that we should repeat the reasons already given; the trial-load method can supply the same results as a model test, when the dam is not complex, that is, when both shape irregularities and heterogeneity in the properties of the materials are absent. However, as the designer has to take into account in the majority of cases, irregularities and heterogeneities chiefly in the foundation rock, he has no tool to preview the behaviour of the structure unless he follows Mr. Glover's suggestion at the end of his discussion, obtaining an overdesigned structure. The advantage of experimental methods is that they can be applied both to simple and complex cases, always with the same accuracy and at a cost lower than the completetrial-load calculation, provided that adequate techniques are used. At present, with the modern high precision testing techniques using small models, a model test costs less than a calculation by the complete trial-load method, as the writers could verify based on the numbers given by Mr. Houck. Note, for instance, that a complete adjustment supplies information on one reservoir level only, whilst informations on the different reservoir levels—what sometimes is of very considerable interest--can be obtained without any change whatsoever in the arrangement of the measuring apparatus. Additionally, it is possible to study various solutions on the same model, if care is taken to begin by the thickest shape. The writers believe that not only the model tests carried out at the Bureau of Reclamation were the first of their kind to be made but also that the measuring techniques were then insufficiently developed, from which resulted a great waste of time and money and a lack of reliance in the results. At present, in Portugal, the preliminary design justified by analytical calculations supplies the first dam shapes to be tested

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on model, whilst the final shapes, that is the design proper, are based, exclusively on the model tests.

The writers are glad to congratulate Mr. Glover on the interesting synthesis presented with regard to the need to make all the adjustments in the calculations by the trial-load method and the influence of the tangential and twist adjustments on the stress distribution.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: DEVELOPMENT IN ITALY

Carlo Semenza,* M. ASCE
(Proc. Paper 1017)

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams (April, 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be printed from the Symposium. So far, eleven papers have been approved: "Arch Dams: Their Philosophy," by Andre Coyne (Proc. Paper 959); "Arch Dams: Trial Load Studies for Hungry Horse Dam," by Robert E. Glover and Merlin D. Copen (Proc. Paper 960); "Arch Dams: Portuguese Experience with Overflow Arch Dams," by A. C. Xerez (Proc. Paper 990); "Arch Dams: Theory, Methods, and Details of Joint Grouting," by A. Warren Simonds (Proc. Paper 991); "Arch Dams: Santa Giustina Single-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 992); "Arch Dams: Measurements and Studies on Santa Giustina Dam," by Claudio Marcello (Proc. Paper 993); "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 994); "Arch Dams: Isolato Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 995); "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs," by Claudio Marcello (Proc. Paper 996); "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira (Proc. Paper 997); and "Arch Dams: Development in Italy," by Carlo Semenza (Proc. Paper 1017).

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

Note: Discussion open until November 1, 1956. Paper 1017 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 3, June, 1956.

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SYNOPSIS

The reasons for extensive development of arch dams in Italy are listed, and the historical stages in that development are traced, with numerous examples.

The methods of analysis are briefly reviewed and special features of construction procedure are brought out. A final section is devoted to an explanation of the benefits of the peripheral point which has been successfully used by the author in several large arch dams.

INTRODUCTION

The rather remarkable development of arch dam construction in Italy is due to several factors which are partly interconnected:

a) The geological characteristics of the country. In the Alps, which constitute the principal hydraulic power area, and still more in part of the Appennines, there is a prevalence of relatively recent formations, where fairly narrow gorges are quite common, for which the arch-dam is the most natural solution.

b) The cost of skilled and specialised labor in the construction industry is still relatively low and hence its factor in total cost is small. Materials, on the contrary, particularly cement and steel, are an important factor. The volume of concrete therefore constitutes the major element of cost.

c) It is recognised in Italy as in any other country, that given the same degree of safety, the choice of the type of dams should depend only on the economic factor. Italian engineers for this reason have been driven to prefer increasingly refined structures in order to reduce the volume to a minimum.

d) The peculiar characteristics of the Italian mentality, which is fairly individualistic, and which therefore tends to examine every problem on its own merits, and free from any preconceived set of ideas.

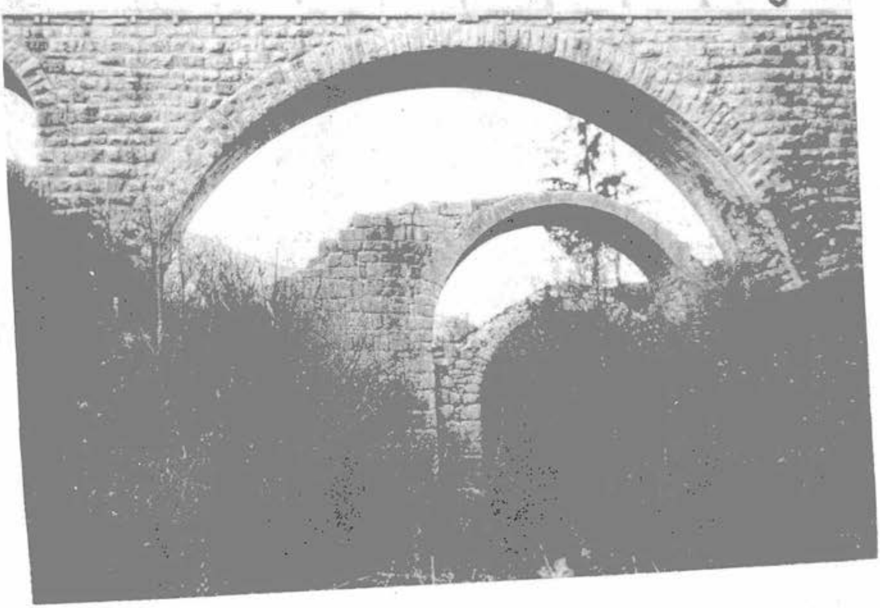
As a result of this attitude of mind, the principle, valid for any country, that each dam constitutes a problem in itself to be solved according to criteria free from any pre-conceived idea, has found in Italy an ideal atmosphere for its full development, sometimes indeed beyond common limits. Hence the widespread and elastic application of the most varied structural forms.

e) This general tendency, which I will call mental, has been reinforced to some extent, both by tradition and artistic environment, since the arch, from the time of ancient Rome, has been a common architectural element: thus it was both logical and natural that modern designers of hydraulic structures should use it.

f) The realization of the exceptional resistance of the arch has grown through centuries of experience in the Italian building workers whose craft has ancient traditions and deep intuition. Even for modest structures in house-building, small and slender brick arches have been used for centuries, as, for example, in Romagna and Tuscany. (Fig. 1.)

From a technical point of view, it is now considered, in short, that the overall safety of an arch structure is far higher than that of a gravity

Fig. 1



structure, as has indeed been proved in model tests to failure. In these it was seen that the collapse of dams takes place at loads which vary between 5 and 11 times the normal, the lower values mostly corresponding to failure of abutments and not of the dam itself.

In two of his contributions to the 1955 Icold Congress in Paris, the author cited some examples which illustrate the peculiar possibilities of arch structures.

In the dome structure which enjoys considerable favor in Italy, an attempt is made to realize a "sail" structure following the funicular polygon of the hydraulic loads and working in every direction with simple compressive stresses and with as uniform a value as possible: ideal limits which cannot be reached in practice, but which correspond to maximum rationality and economy.

In the arch-gravity dams—which are designed in accordance with a more complex conception because here the effect of weight is considered—there is a tendency towards structures in which the maximum compressive stresses are as far as possible of the same order whether with empty or full reservoir, and spread as evenly as possible over the whole structure. (Fig. 2.)

The Development of Curved Dams in Italy

It is not easy to summarize the history of arch dams in Italy in a few lines.

The oldest are probably the many barrages built by Government engineers for flood control purposes, or to stop debris, in the narrow mountainous valleys or for irrigation or to drive flour-mills. Perhaps the oldest of all is that of Ponte Alto in the neighbourhood of Trento. The construction of this barrage was begun in 1537 and continued with arched heightenings in the years 1611, 1748 and 1883. The height of the barrage thus reached 46.80 m. Even though the gorge is exceptionally narrow, such a height, considering the period is worthy of note. (Fig. 3.)

Naturally, these modest dams did not claim to utilize the masonry to the full; they were proportioned roughly on the cylinder formula and almost always built of stone masonry, often of cyclopean type.

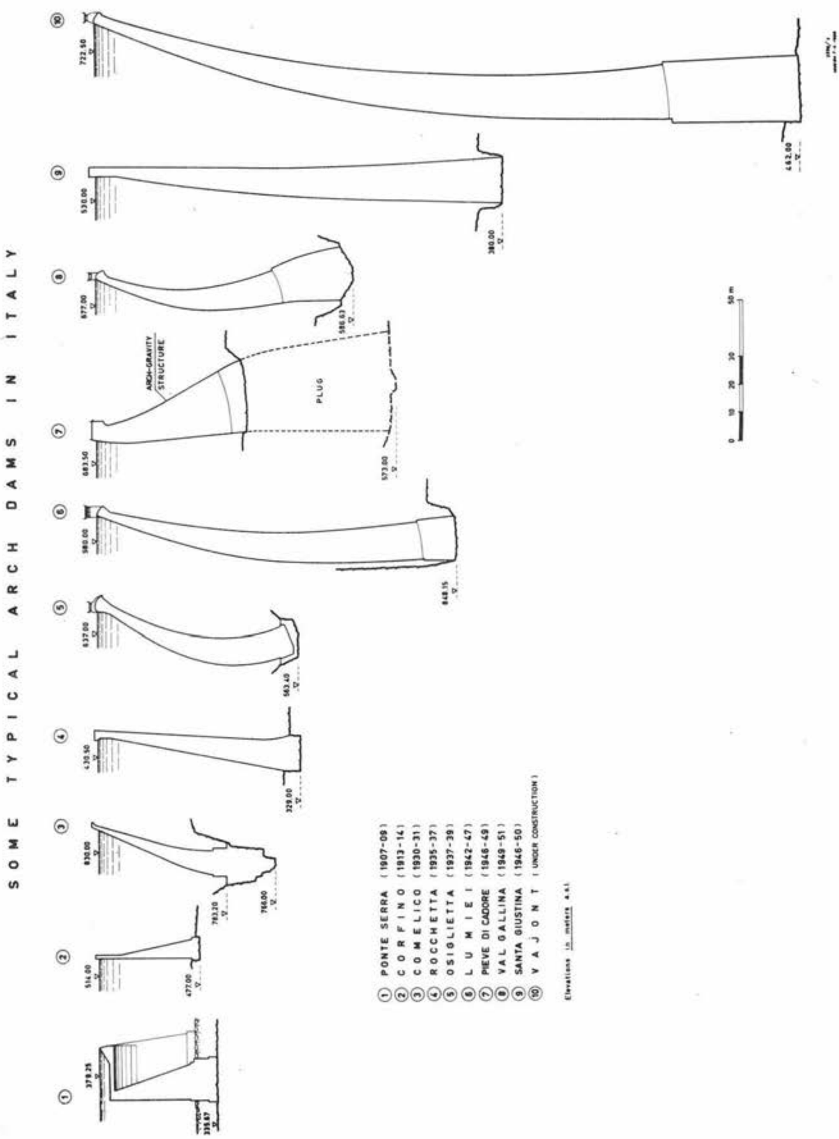
Of course, the great development of arch dams has its true beginning in more recent time. A table of the arch dams existing in Italy is attached.

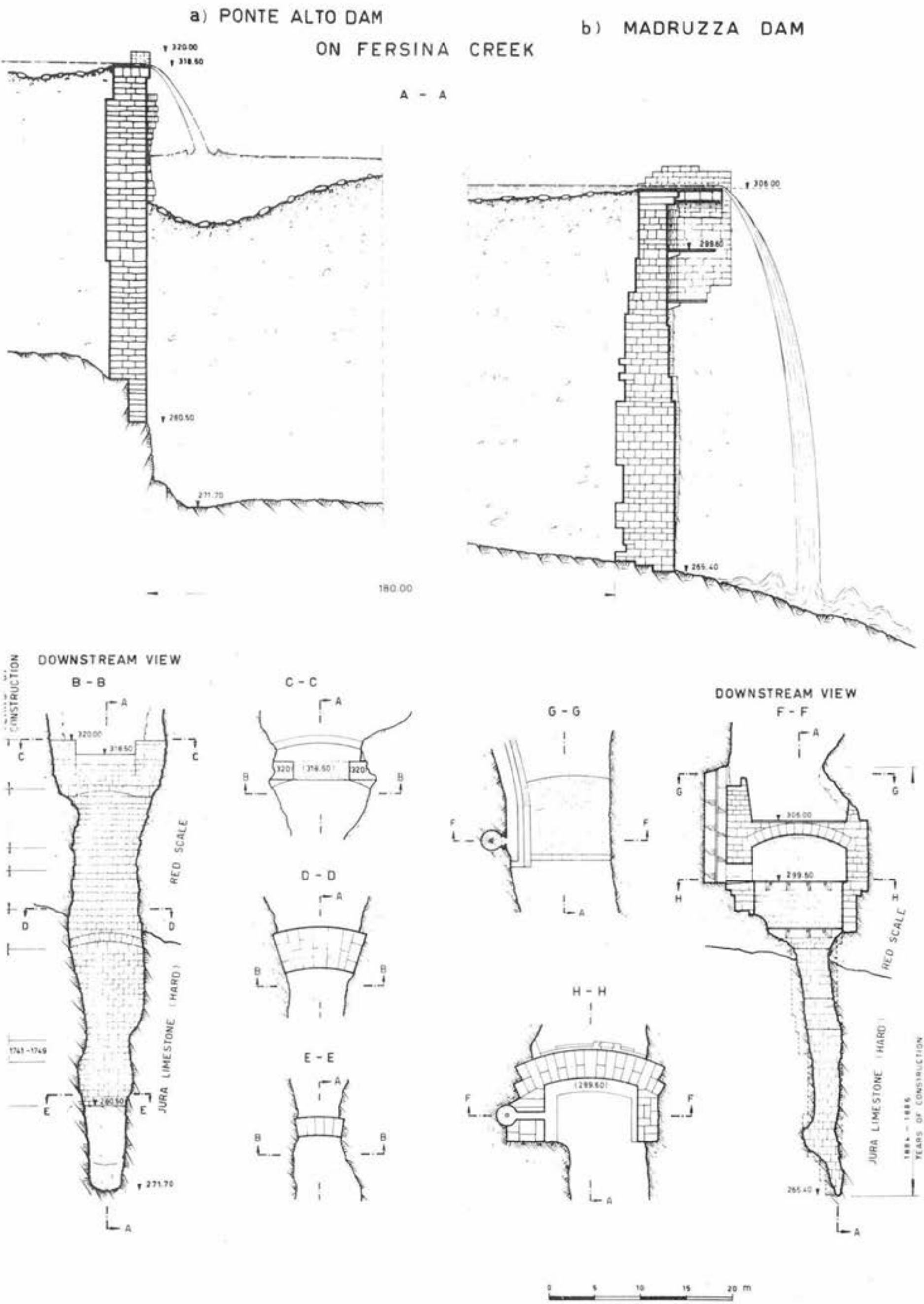
Generally speaking, it can be said that the struggle to economize has, from the very beginning, been one of the foremost idea in our constructional work and has pervaded its whole subsequent development.

In this development one can distinguish certain fundamental stages which of course cannot be exactly defined in the progress of time: that is, they are superimposed one above the other. In general, and to avoid any misinterpretation, I would point out that in my rapid summary, I shall limit myself to mentioning those constructions which have marked important steps in conception and realization: exclusion, therefore, casts no aspersion on the importance of any given structure, but is simply due to the fact that in my view that particular dam does not constitute an essential step forward with respect both to preceding and following structures.

The First Stage

The first stage, from the beginnings up to about 1930, is characterised by


SOME TYPICAL ARCH DAMS IN ITALY
378.25
335.87
514.00
477.00
830.00
783.20
766.00
430.50
329.00
637.00
563.40
980.00
848.15
683.50
ARCH-GRAVITY STRUCTURE
PLUG
573.00
677.00
590.63
530.00
380.00
722.50
462.00
0
10
20
30
50 m
① PONTE SERRA (1907-09)
② CORFINO (1913-14)
③ COMELICO (1930-31)
④ ROCCHETTA (1935-37)
⑤ OSIGLIETTA (1937-39)
⑥ LUMIEI (1942-47)
⑦ PIEVE DI CADORE (1946-49)
⑧ VAL GALLINA (1949-51)
⑨ SANTA GIUSTINA (1946-50)
⑩ VAJONT (UNDER CONSTRUCTION)
Elevations in meters a.s.l.


a) PONTE ALTO DAM
ON FERSINA CREEK
b) MADRUZZA DAM
A - A
320.00
318.50
280.50
271.70
180.00
306.00
299.60
265.40
DOWNSTREAM VIEW
B - B
C - C
D - D
E - E
G - G
H - H
DOWNSTREAM VIEW
F - F
320
(318.50)
(299.60)
RED SCALE
JURA LIMESTONE (HARD)
1741 - 1749
CONSTRUCTION
1884 - 1886
YEARS OF CONSTRUCTION
0
5
10
15
20 m

the construction of arch dams with generally cylindrical upstream face and vertical axis.

The dam over the Cismon at Ponte della Serra in the province of Belluno (1906-1909), which on account of its thickness could properly be called an arch gravity dam belongs to this period. However, here the cantilever effect was not analysed and, therefore, not relied upon. The whole of the dam is somewhat original inasmuch as downstream from the arch and in contact with it a thick vertical pier was constructed on the center line. On this pier and on the two rocky sides of the valley rest two heavy arches whose extrados constitutes a flood-spillway. Upstream, this structure is also supported by the crest of the arch.

The dam, which was the work of the late Prof. Angelo Forti, is built partly of masonry and partly of concrete without construction joints between the various structural elements.

The cross-section of the arch is a massive triangle; the structure was analysed by the cylinder formula without taking in account temperature variations.

Rather daring for those times may be considered the idea of discharging from the crest high floods up to 750 cu.m./sec. into the river without any protecting structure downstream.

This dam has given excellent results from every point of view. The Author who since 1929 was responsible for its maintenance, had only to arrange for partial downstream reinforcement at the base of the central pier in 1929.

A little later than the Cismon dam comes that of Corfino on the Serchio (1913-1914), the first example in our country of a thin arch dam. With a height of 35 m and a radius of 28 m, it has a base thickness of 5.50 m. The dam has stood up to a violent earthquake without the slightest damage. Other dams of this period are those of Muro Lucano, Turrite di Gallicano, planned by the later Mr. Angelo Omodeo, and that of Furlo (1919-1921). The so-called Ritter method was used for the calculation of the last-named.

The planning of a large dam (the Sottosella dam) on the Isonzo built by the Author in 1937-1939 in territory now passed to Jugoslavia was carried on conservative lines: it does not represent however a return to the idea of massive dams with cylindrical upstream face. The solution chosen is due entirely to the peculiar requisites of the site and of construction; the main problem of the dam was indeed to carry out the foundations in a narrow rocky river-bed subject to considerable and very frequent floods. The structure of the dam had therefore to be adapted to that of the foundations which consisted of one single large compressed air caisson built like a bridge on the two rocky sides of the gorge and sunk right down to the rocky bottom.

It is of some interest to mention that in a previous design of this dam, as I explained in a publication of 1940, a flood crest spillway (up to 2000 cu.m./sec.) was designed with a 12 meter water head and with a large flume connected to the dam, but statically independent of it. The structure of the flume did not have to rest on the rocky bottom, but on a series of great arches spanning the rocky banks.

The Second Stage

The last ten years of what I called the first stage, that is to say, the years after 1920, witnessed the adoption of the method of analysis based on the theory of horizontal arch elements with encastrement at the abutments which were held to be "rigid." Prof. Guidi was the first exponent of this in Italy. At the same time the principle of the variability of radii was applied tending towards the type with a constant or nearly constant central angle of the arches.

The characteristics of the arches had to be developed rapidly: the vertical cross sections pass from triangular forms more or less thick at the base to curved profiles, sometimes with the upper arches considerably overhanging downstream at the crest. In other words, the simple arch begins to tend towards the double curvature structure. The plan consequently becomes more complex and tends towards a shell shape which later on will be characteristic of dome dams.

There is also a trend towards the principle of the variability of thickness in the individual arches with appropriate thickening from the crown to the abutments.

The first important example of this type of structure is the Comelico dam on the Piave downstream from S. Stefano di Cadore (1930-1931), which, like the dam at Fortezza of which I shall speak later, was the work of the late Mr. Nicolai, who left a notable mark on the history of arch dams in Italy.

The dam is 66 m high at the deepest point of the foundation, 1.20 m thick at the crest, 8.66 m at the base. By applying the principle of the constancy of the central angle of the arches, a cross section was obtained with an upstream face which overhangs 17 m downstream, and a downstream face which overhangs 10 m. The result was a construction of some elegance. The dam is built of concrete with light steel reinforcement for a better distribution of the stresses and a heavier reinforcement at the abutments, to put into practice the theoretical encastrement conditions.

Practically contemporary (1931) is the Ceppo Morelli dam on the Anza, which was the work of a great Italian engineer, Mr. Vincenzo Ferniani.

In the Rocchetta dam, of slightly later date (1935-1937) even the central angles of the arches vary (from 100° to 120°); the extrados radii decrease from 71.90 m at the crest to 33.90 m at the base. The upstream face has an overhang of about 17% and the downstream one of 5%. The arches thicken slightly from the crown to the abutments. Particular attention was given to the surface of support of the structure in order to render it practically continuous according to the principle of the perimetral joint adopted later (see Osiglietta dam). This dam has also undergone the test of some earthquakes without suffering any damage whatsoever. (Fig. 4.)

The Fortezza dam on the Isarco (1938-40), i.e. slightly later than the Osiglietta dam of which I shall speak further on and which marks the beginning of a definite tendency towards the dome structure, has a steep downstream overhang; radii, central angles and thickness of the arches vary continually. The arch (with overhang lip) has no encastrement and rests both at the abutments and at the base on a sort of continuous cradle, as in the case of Osiglietta.

The Third Stage

All these dams may perhaps be considered as transitional structures leading towards the more refined type, now widely used in Italy, of the dome dam. The first, and remarkable example, is that of Osiglietta in the Appennines of

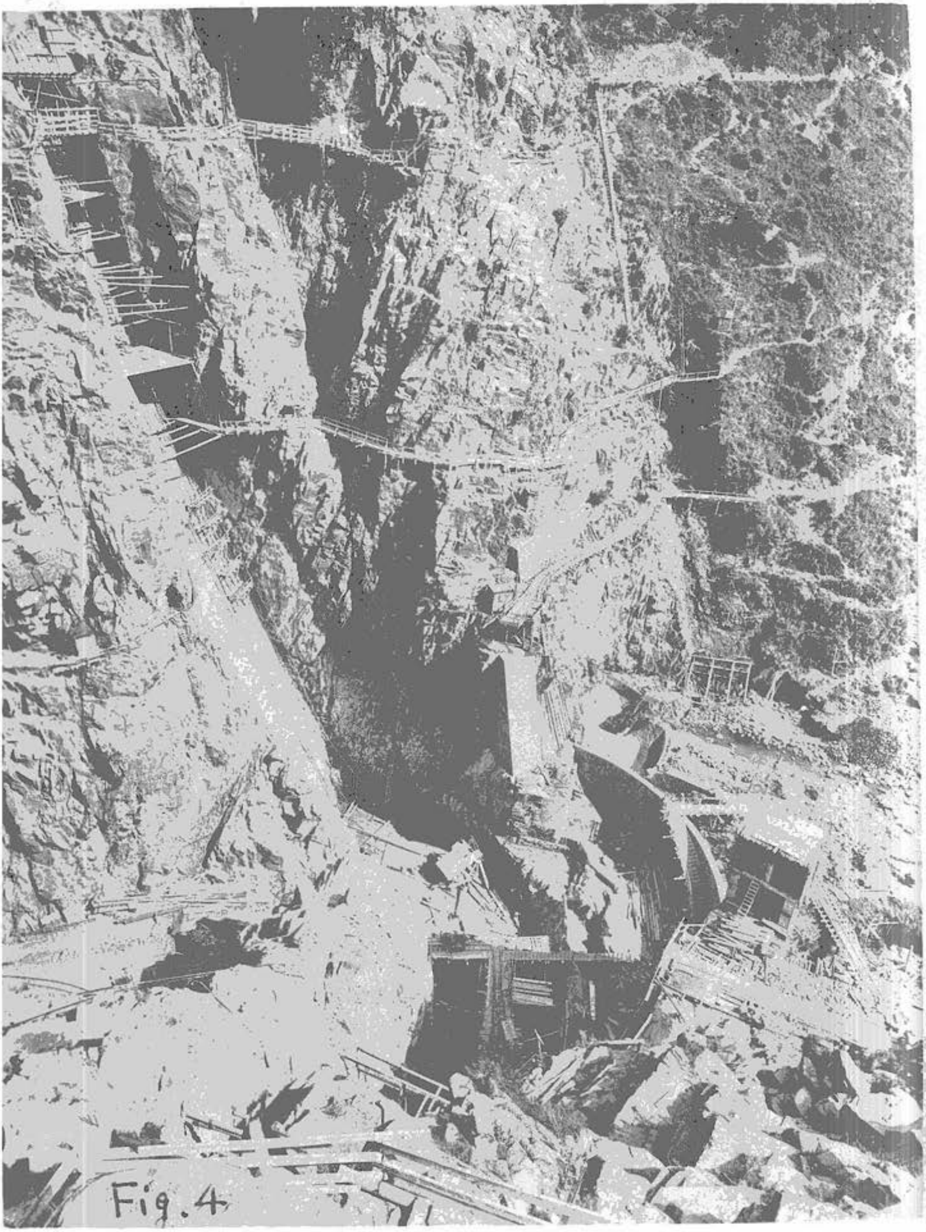


Fig. 4

Liguria (1937-39). About 67 m high with a crest length of 224 m and a volume of 75,000 cu.m., it has many characteristics which are still being discussed in this field. It consists of horizontal elements of varying thickness with downstream face radii of 21 m at the base to 101 m at the crest and with arch central angles of from 79° to 129°. The crown cross-section has an upstream overhang at the base of 2.40 m and a downstream overhang at the crest of 15.45 m; both faces are strongly curved. The dam is perfectly symmetrical and its abutments have practically a continuous curvature. (Fig. 5.)

Another noteworthy characteristic is provided by the suppression of the lateral encastrement: the dam simply rests on a sort of continuous concrete saddle which follows the rocky surface of support. It constitutes the first example of this device, of which I shall speak fully in the last part of this Paper.

The Osiglietta dam was followed almost immediately, i.e. already during the second world war, by the excavations and the first concrete pouring for the Lumiei dam which was planned in 1939-40 and completed in 1946-47; and after the war by a considerable number of dome dams of great height. These dams were built in sites, and therefore with characteristics, very different one from another, but nevertheless designed and carried out along fairly similar lines. In the region of the Tre Venezie alone there are a score of structures of this type.

The Lumiei dam (1942-47) is doubtless the most important of this group: 136.15 m high with a crest length of 138 m, volume 100,320 cu.m., it is one of the thinnest structures of its kind. (Figs. 6 & 7.)

It was for some years the highest arch dam in the world and was the first dam in Italy whose analysis began to take account of the effect of rock deformation.

Another dam built in that period is especially remarkable for some details of its abutment-saddle of which I shall speak in the last part of this paper: that of Val Gallina (1949-1951). (Figs. 8 & 9.)

Outstanding for its peculiar site conditions, characteristics and height, is the dam of S. Giustina on the Noce (h=152.50 m, V=120.000 cu.m.) which in its day, like that of Lumiei, held the "blue ribbon" for arch dams in the world, an honour now held by Tignes.

The characteristics of the gorge at S. Giustina, with practically vertical sides, obviously required a solution in which the small variation in the central angles and the almost vertical upstream face made it resemble the cylindrical type very closely. (Fig. 10.)

The dam of Rio Freddo in the Western Alps near Cuneo is peculiar because it consists of a thin arch supported by two large concrete shoulders (height 45 m) which sustain the pressure of the arch. From the two artificial abutments two short gravity walls keep the water upstream.

The construction of the large Vajont dam in a very narrow and striking limestone gorge in the Dolomites has started this year. With its height of 262,50 m, a chord of 160 m, V = about 335.000 cu.m., it will rank amongst the highest dams in the world.

The Fourth Stage

In the last fifteen years the problem of arch-gravity dams has been squarely faced in Italy. Many dams built in the past have actually utilized,

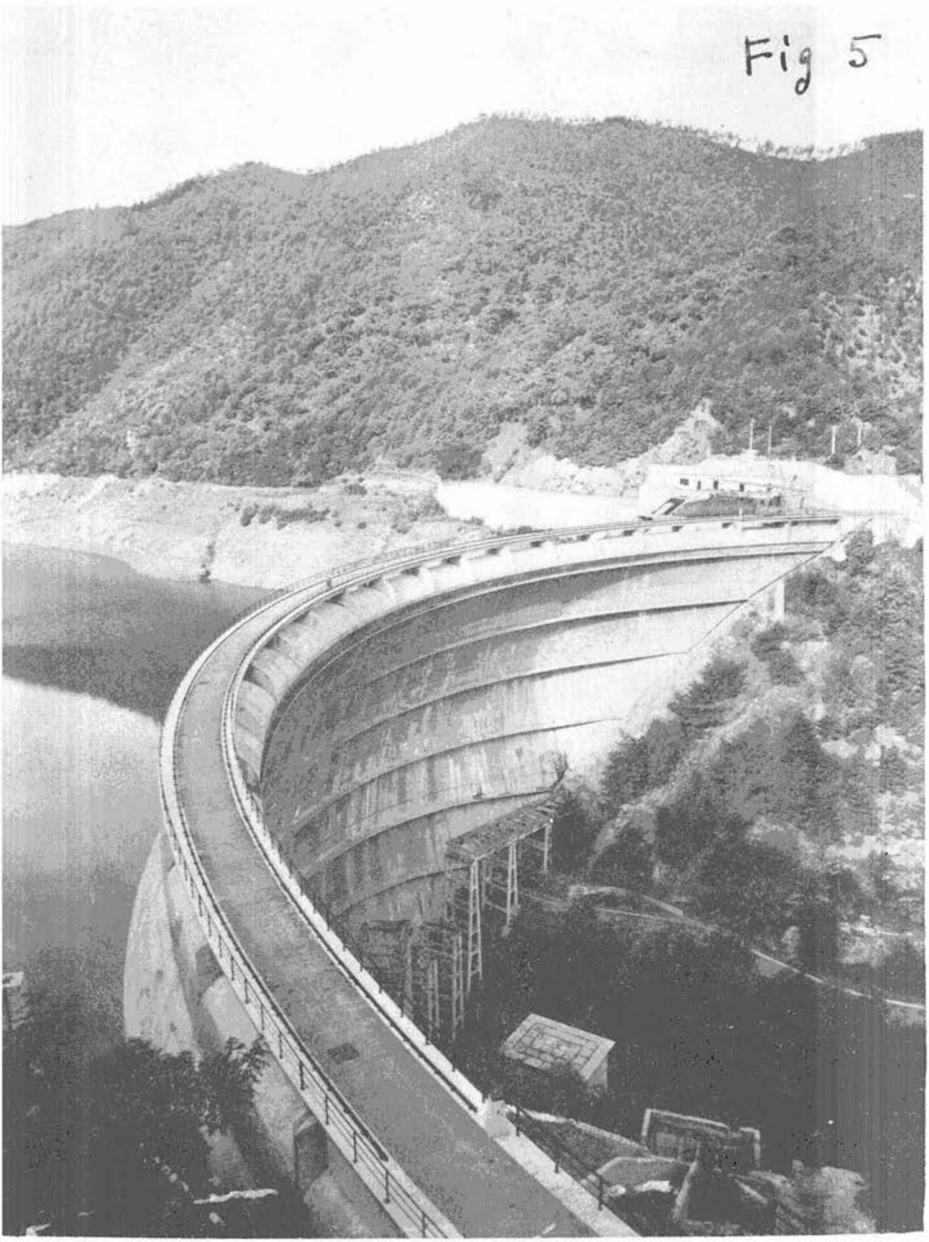


Fig 5

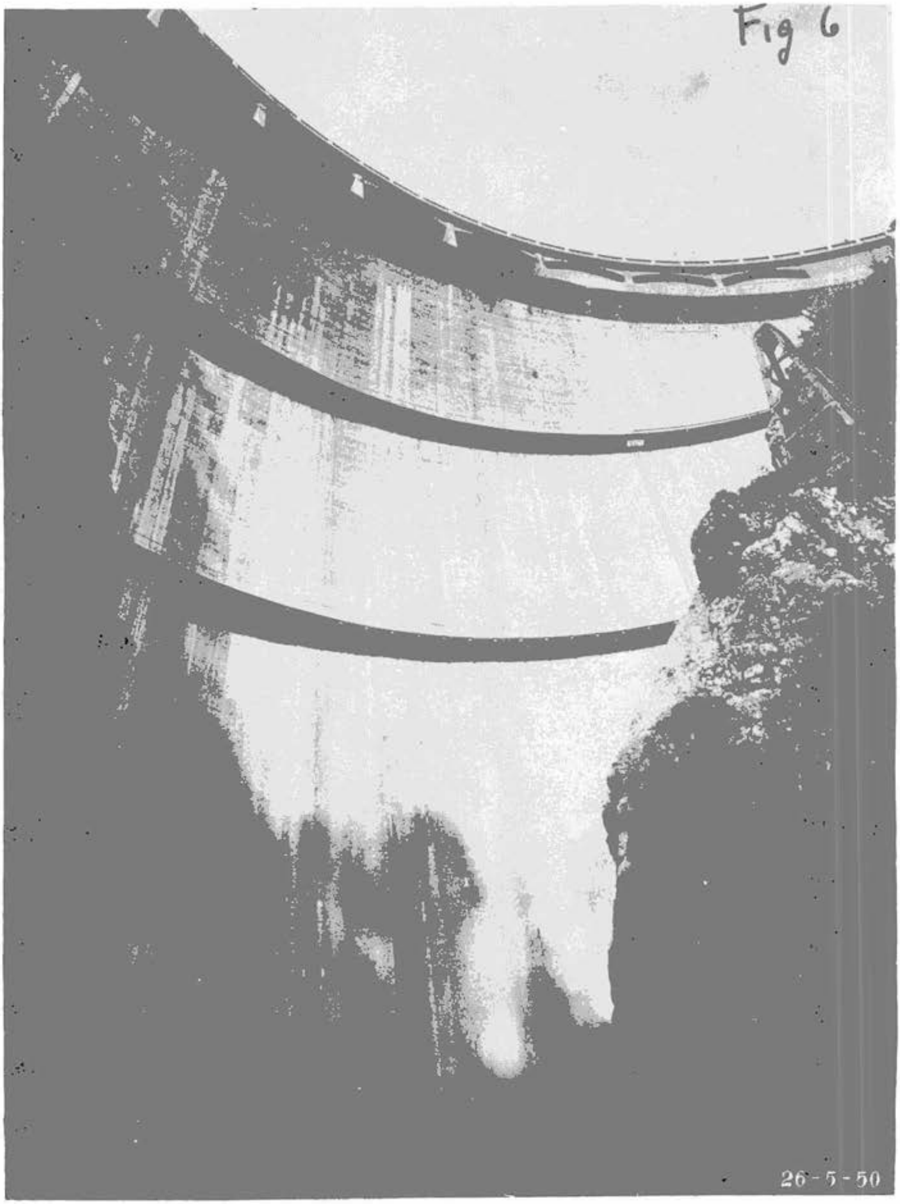


Fig 6
26-5-50

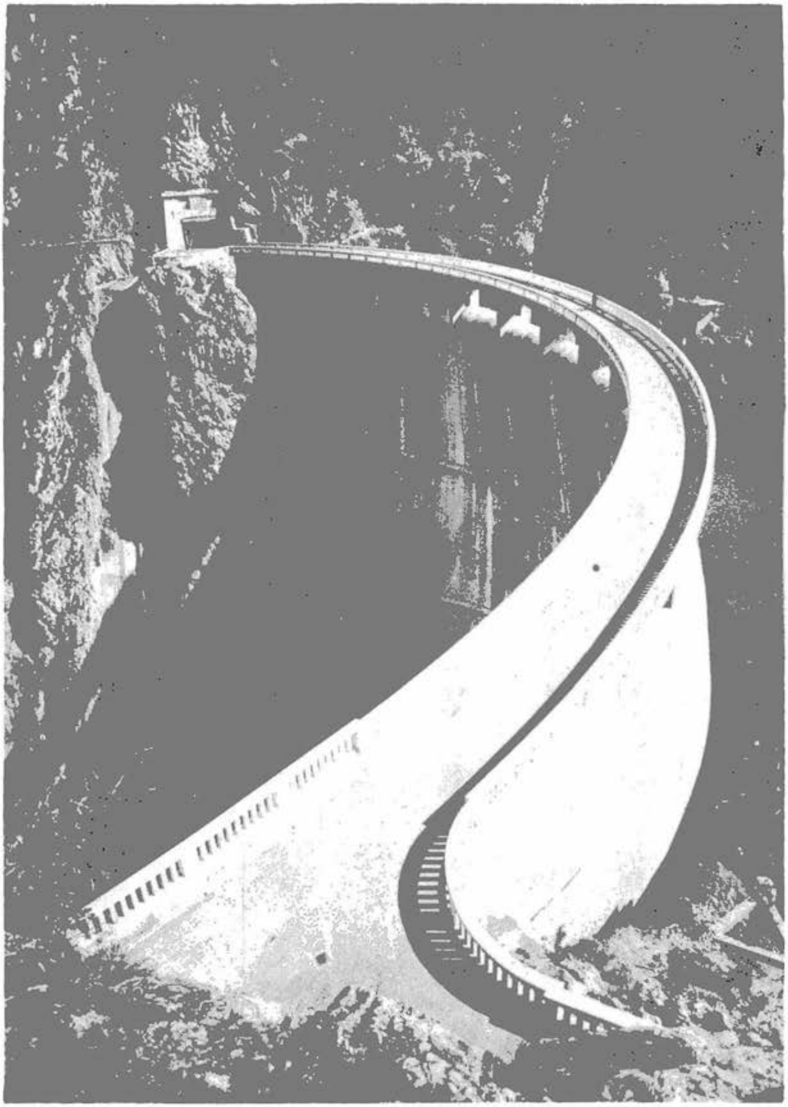


Fig. 7

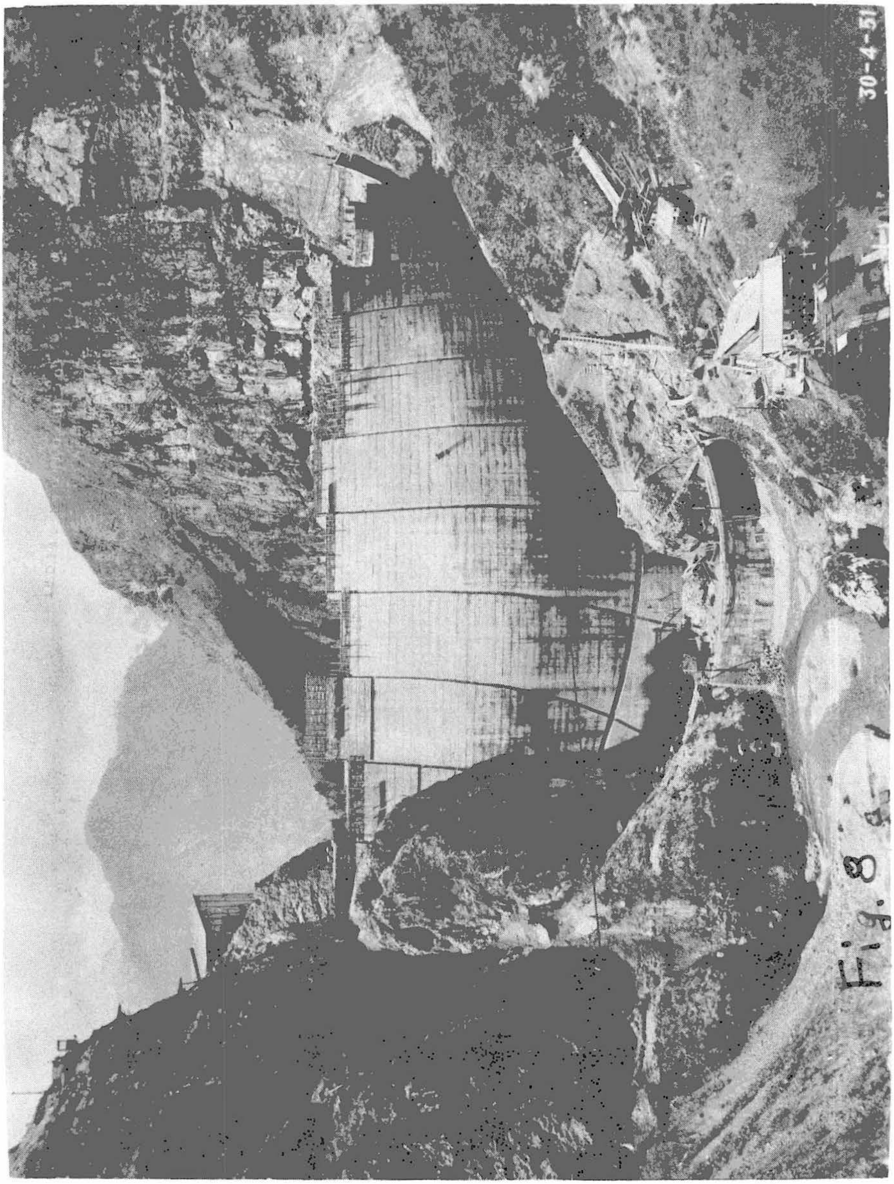


Fig. 8
30-4-51

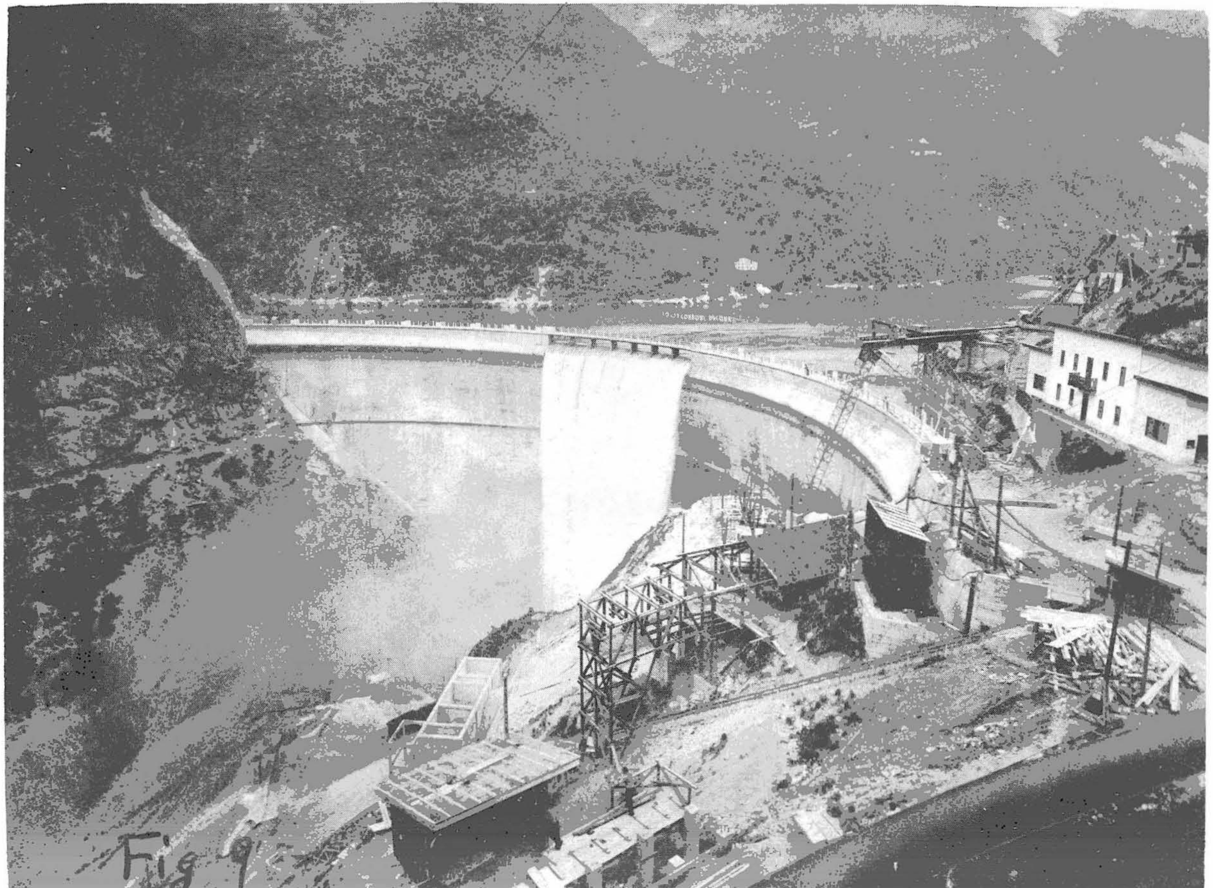


Fig 9

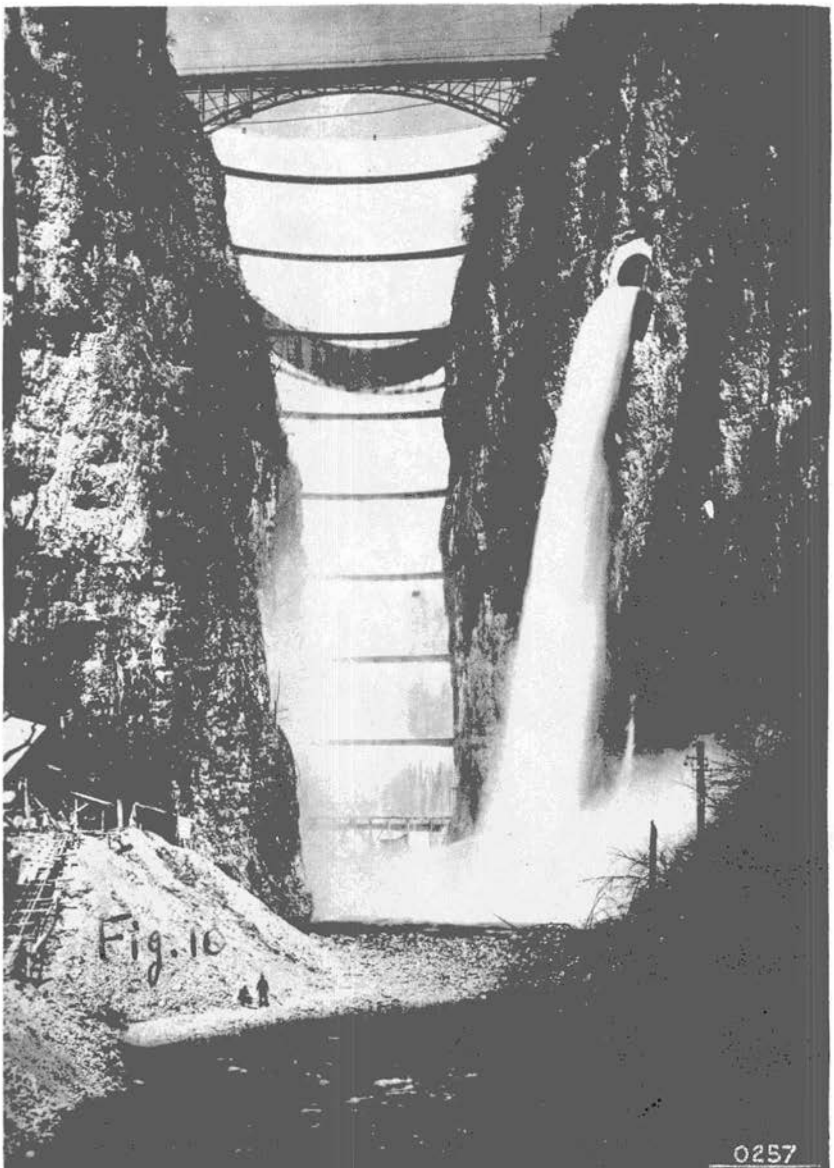


Fig. 10
0257

because of their characteristics, the double way of working of the structure, but the conception in which also the cantilevers—that is the weight—come to be considered as a determining static element, was adopted, I believe I am right in saying, for the first time in Italy in the Pieve di Cadore dam. (Figs. 11 & 12.)

As regards the behavior of the dam as a dome, in this particular case, owing to the features of the site, it is of secondary importance; on the contrary the other great arch-gravity dams built or in construction since 1945 tend to exploit the dome function to the utmost.

The Pieve di Cadore dam is really a unique example, both for its dimensional ratios and for the fact that its arched structure (of constant height on the perimetral joint) rests for the most part on a rocky plateau, while the remainder is supported by a concrete plug which closes a deep narrow gorge lying alongside the plateau.

The dam has a maximum height (from the base of the plug) of 111.50 m; the average height of the arch gravity structure on the general foundation line (elev. 634 m) is 55 m, the crest length 410 m. The total volume is 377,000 cu.m. The thickness varies from 6 m at the crest to 26 m at the base of the arch and 36 m at the base of the lateral foundation plug. The arches up to elevation 640 m are fundamentally circular with a constant radius for the upstream face, whereas the downstream face is polycentric; above 640 m the axis gradually becomes polycentric with more and more substantial differences between the radii. Hence the cross sections vary continuously from the crown to the abutments; they bend downstream at the crown and straighten out at the abutments as in dome dams.

The Pieve di Cadore dam is also remarkable for the number of measuring devices installed.

Following the construction of the Pieve di Cadore dam, other noteworthy arch-gravity structures were built. Among these we will cite, especially, the dams of Publino in Valtellina (h = 34 m, chord length 173 m, volume = 34.000 cu.m.), Travignolo (h = 110 m, volume = 260.000 cu.m.), Mucone (h = 55 m, volume = 62.500 m), Fiastrone (h = 87 m, volume 106.000 cu.m.; moreover, those at present under construction at Beauregard in Val d'Aosta (h = 132 m, volume = 440.000 cu.m.), Cancano in Valtellina (h=173 m, volume = 1.100.000 cu.m.), Mulgargia (h = 99 m, volume = 240.000 cu.m.) Flumendosa (h = 119 m, volume = 305.000 cu.m.) in Sardinia, Frera in Valtellina (h=138 m, volume = 420.000 cu.m.). (Figs. 13 to 18.)

Several of the dams built in the last ten years are particularly interesting because of the peculiar problems solved in connection with construction on not altogether reliable rock. Among those I have already quoted is the Val Gallina dam, the foundation of which had to a great extent to be substantially reinforced. I still remember, in connection with the Beauregard dam already mentioned, that at the foot of the left abutment of this large dam there was a sort of enormous pocket of alluvial sand about 30 m high (which was discovered only after the main part of the excavation was carried out), the filling up of which required considerable concreting.

I have cited these problems not so much because they involve particular solutions from the point of view of construction but to show how in Italy in many cases there is no hesitation in adopting the arch structure even in not altogether favorable geological conditions, always of course provided the appropriate measures are taken. The study of the measures and of the special precautions to be taken in these cases has reached a pitch of development in my country which I consider worthy of interest.

Fig. 11

PIEVE DI CADORE DAM
ON THE RIVER PIAVE

DEVELOPED ELEVATION
ALONG THE MIDDLE FIBRE OF THE ARCHES

FROM DOWNSTREAM

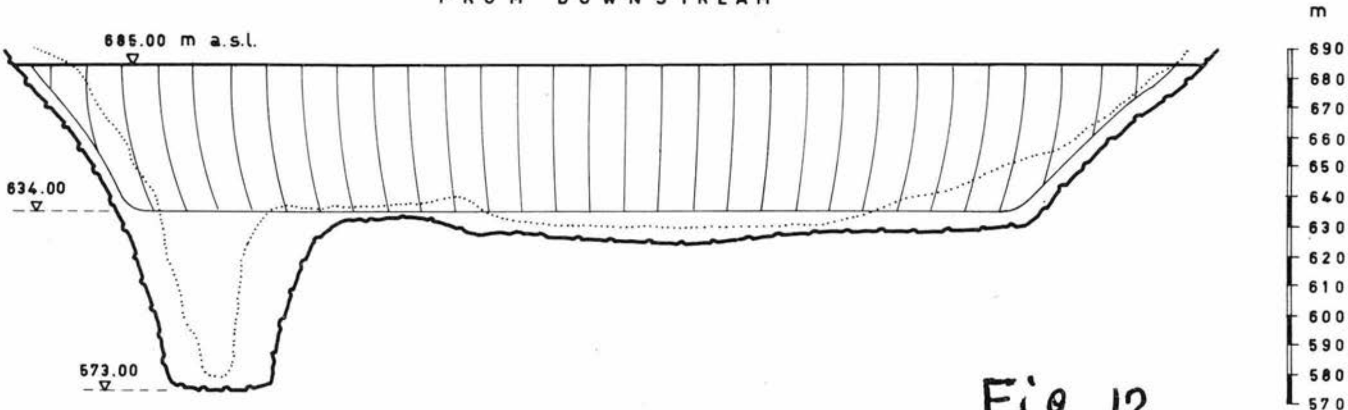


Fig. 12

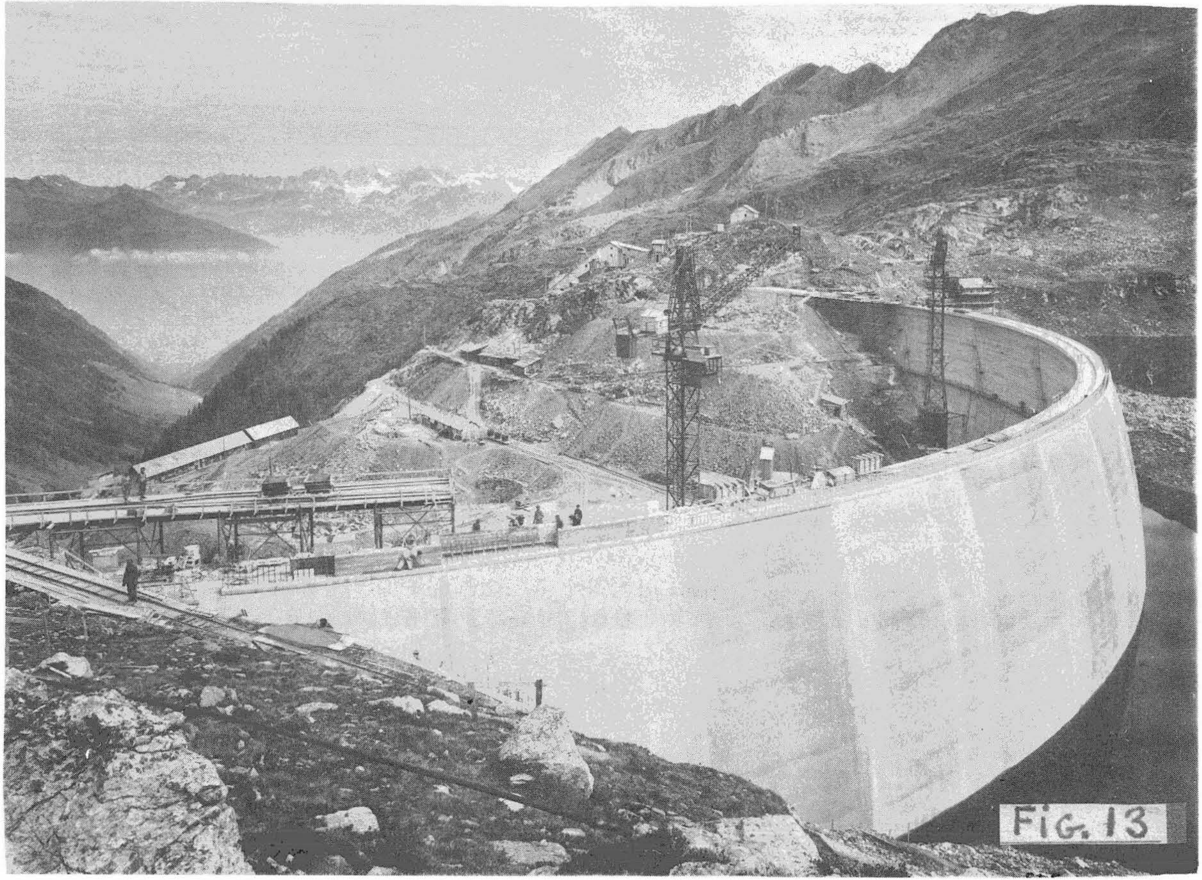


Fig. 13

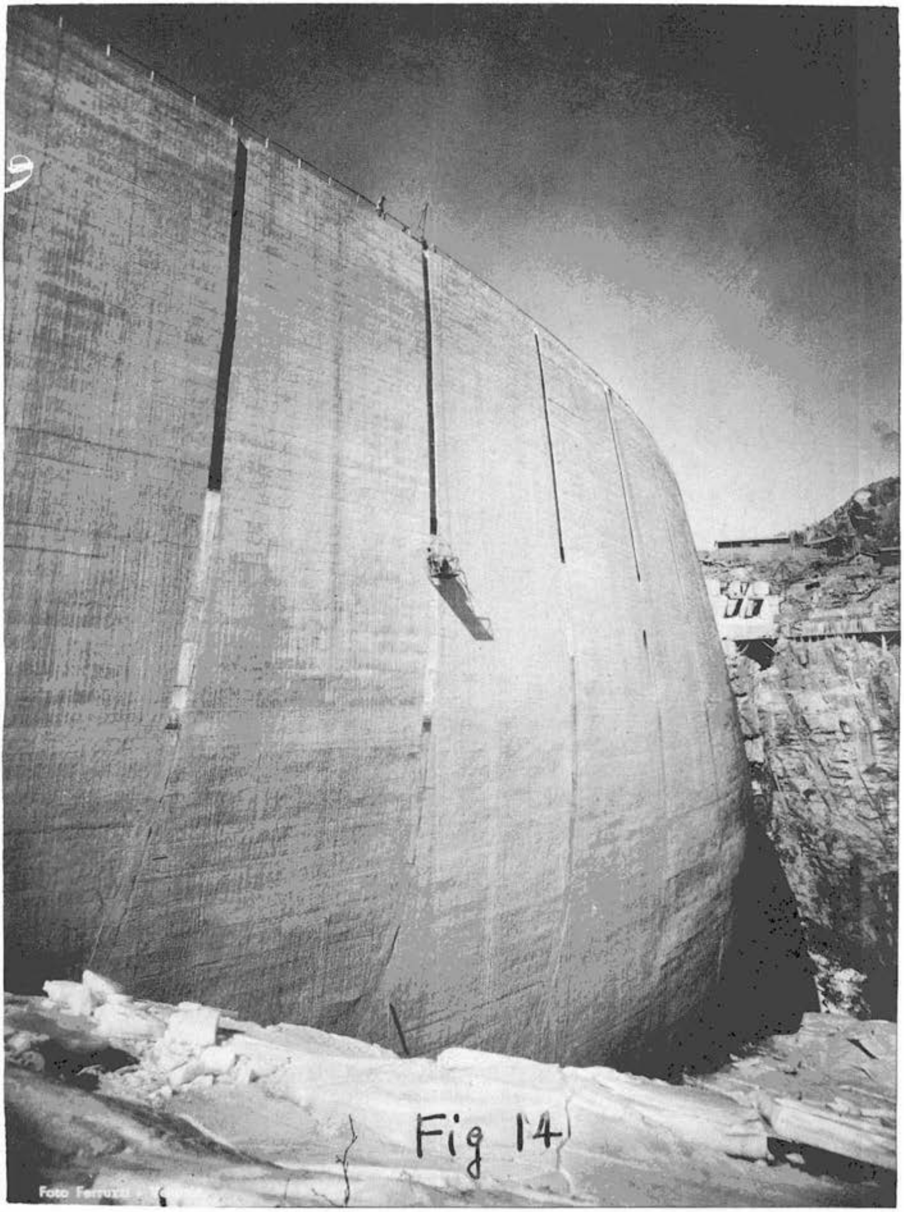


Fig 14

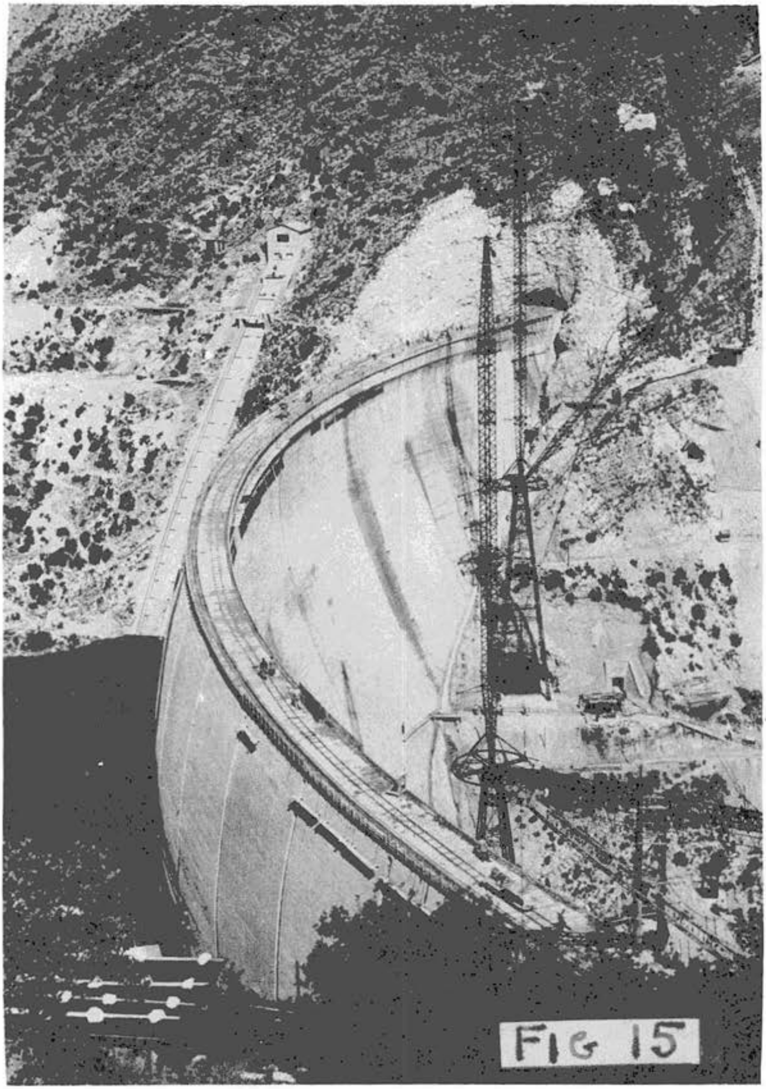


FIG 15

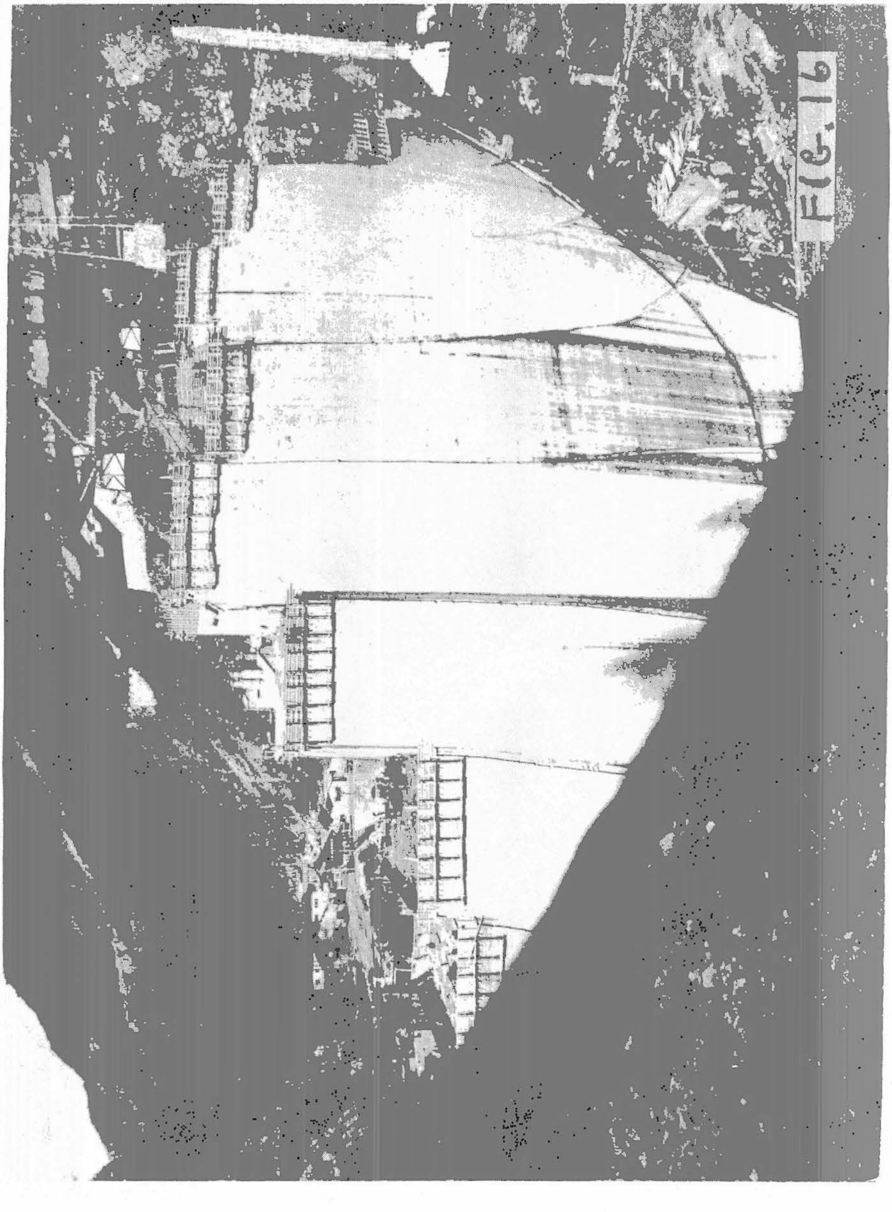


FIG. 16

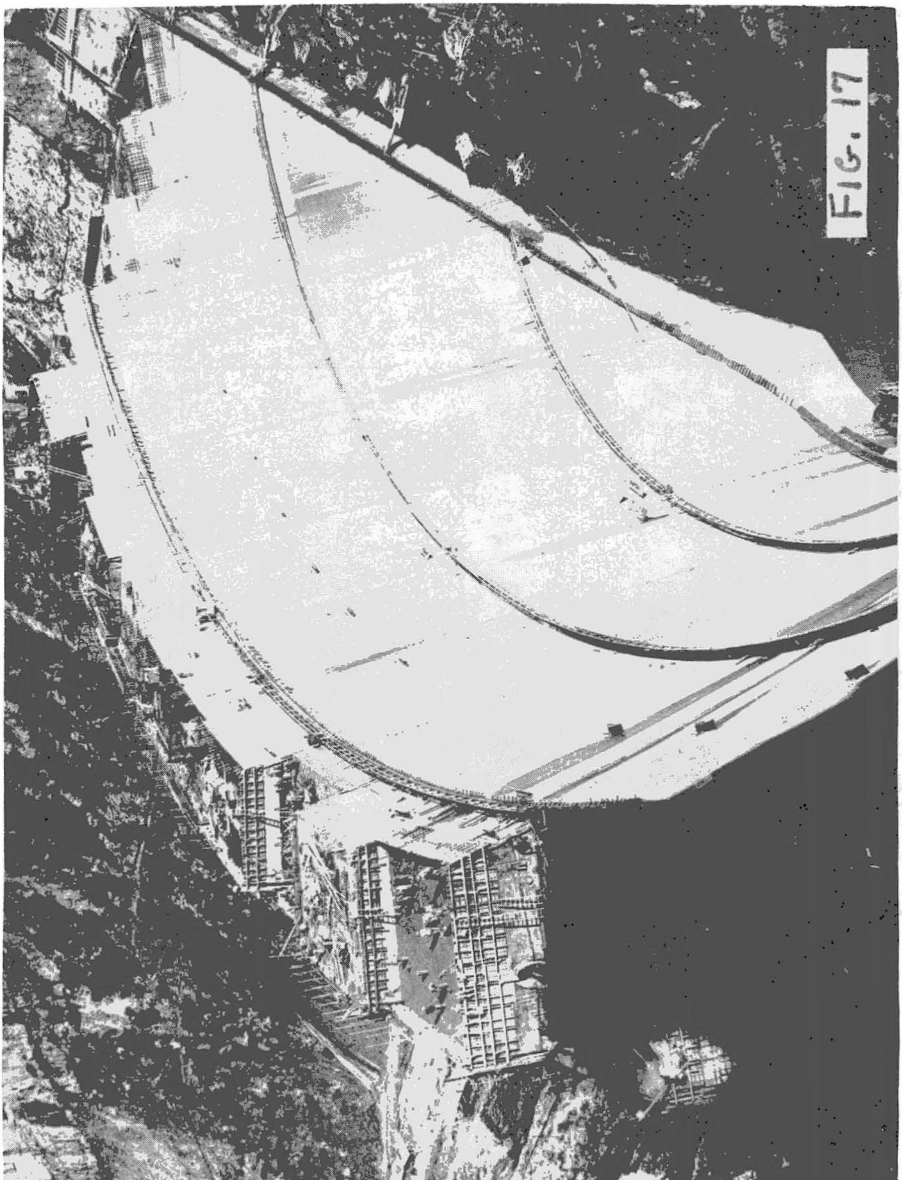


FIG. 17

FIG 18

Methods of Analysis

As I have mentioned incidentally in preceding paragraphs, the principles underlying the analysis for arch dams in Italy have followed more or less the same evolution as in other countries. The basic calculation, which even today gives the very first indications for a preliminary sizing-up is that based on the cylinder formula; assuming correlatively rather low stresses.

The Italian "Regolamento Dighe" itself (Dams Regulation) also allows the application of the cylinder formula for the lower parts of the structures, where the thickness of the arches is too great in relation to the radius to permit the use of elastic arch analysis. For all the other parts of the structure the use of this last type of analysis is fundamental. As I have already said, it was introduced by Prof. Guidi with subsequent applications by means of the ellipse of elasticity. The Italian regulations sanction this practice: the "Regolamento Dighe" in force accept in the main the elastic arch analysis for arch and dome dams which can be reasonably considered only from that point of view.

In structures where the ratios between the dimensions cause the cantilevers to have an actual static function (but not so remarkable as to make it necessary to consider them genuine arch-gravity structures), an ordinary analytical check of the cantilevers for the effect of their own weight is required. In some cases the study is limited to the central cantilever, following the Ritter and Stucky systems, in other cases extending the calculation to other cantilevers.

Furthermore, in some cases, the behaviour of the upper parts of the structure is checked considering the inclined arches (the "arches plongeants" of the French engineers): however, this control does not always give satisfactory results because the choice of the arches is often difficult to define. In other cases, the analysis with the "membrane type" calculation is adopted, i.e. an extension to thin revolution arches of the classical method by Schwedler, reported also by Krall.

The foregoing refers of course to the true arch or dome dams, relatively slender, in which the resistance of the cantilevers can be totally or partially ignored. When this conception does not apply, recourse is made to the calculation of arch-gravity structures according to the classic methods of Tölke and the Trial Load.

Special applications of these methods have been carried out by Italian specialists. Professors Danusso and Oberti have availed themselves of the Tölke method, adding some original ideas of their own. Prof. Arredi, in his turn, for the planning of the Piave dam, studied special methods (inspired partly by some ideas of Smith) which were later generalised and amplified and applied to the analysis of many other structures. Prof. Tonini used the Trial Load Method repeatedly and introduced simplifications and additions of considerable practical interest, substituting research by systems of equations for that by trials.

As a consequence of the improved knowledge of the distribution of stresses in arches, due to theoretical studies and model tests, the monocircular arch form and the constant thickness have now been completely abandoned. Nowadays the normal form for the arch has increasing thickness and increasing curvature towards the abutments (the fillet arches), sometimes with local points of variation as in the case of polycentric curves, and sometimes gradually as in the case of special curves.

The hypothesis of foundation deformability has also been applied for the first time in Italy by Oberti in 1938 according to widely known principles and formulas to be found also in American technical literature.

A great deal of attention has been paid to the problem of the stresses due to temperature variations and there are important studies on this subject by Professors Guidi, Puppini, Ippolito and Arredi.

The widespread use of model tests, which are genuine tools for three-dimensional calculation of structures, has been a great help in the study of arch dams in Italy. Prof. Oberti will deal with this. I will therefore confine myself to mentioning the fact that model tests permit the analysis of even very special circumstances of great interest for the study and adoption of arch structures, such as peculiar conditions of the rock of the abutments, the influence of assymmetry and now also the behaviour of the structure in case of earthquakes of all kinds and degrees.

With regard to the maximum stresses which can be used for concrete there is a tendency to accept values equal to one fifth of the 90 day strength the regulation in force still prescribes 1/5 at 28 days and 1/7 at 90 days but it will probably soon be modified. In Italy the normal values reached for arch and arch-gravity dams are of the order of 60 to 70 kg/sq.cm. for the compressive stresses and 8 $\div$ 10 kg/sq.cm. for tensile stresses.

To conclude this short chapter on the principles of calculation I would like to place the objective view point of the Servizio Dighe (Dams Department) of the Italian Ministry of Public Works which, without ever departing from its safety standards, has allowed every rational step in our field.

Construction Methods

I do not think that the principles of construction in Italy differ greatly from those practised in other countries. We may consider before all the question of the time required for the construction. In particular, one may say that in Italy the construction of an arch-dam is not held to constitute, from that point of view, a more serious or worrying problem than that for another type of concrete dam.

In other words, it is held that a dam, simply because it has to be built as an arch dam instead of a straight one, cannot make the whole construction a longer business.

The difficulties here might arise exclusively from the climatic conditions which in many Alpine districts are peculiarly severe. Hence the arch solution, with its lower volume, may with suitable organisation involve less time. From a monthly pouring of 44,150 cu.m. for many consecutive months on the Piave dam in the years 1948 and 1949, the figure of 54,700 cu.m. was reached in 1955 for the Cancano dam in the Upper Valtelline, and those figures were almost reached on Pian Telessio and Beauregard dams.

It is obvious that, in case of necessity, for the arch-gravity dams of great thickness, the daily pouring could be increased so as to reach quantities comparable with those of a gravity dam at least in proportion to their respective thicknesses. The arrangement of radial blocks, normally 10 to 15 m thick in Italy, corresponds exactly to the pouring for blocks in gravity dams. Obviously the difficulties of bigger concrete pourings do not lie so much in the lesser thickness as in the height of the layer that can be poured daily, and this is true in general for any type of dam. There may be further limitations

TABLE 1. SIMPLE CURVATURE ARCH DAMS

a) with mostly constant radius

b) with mostly constant angle

Name of the Dam	Type of the Dam	River Basin	Dam: Maximum Height m	Dam: Volum[e] m^3 x 1[…]
1. Crosis	a)	Isonzo	38,20	
2. Ponte della Serra	a)	Brenta	44,40	19,7
3. Corfino	a)	Serchio	37,50	2,2
4. Muro Lucano	a)	Sele	53,25	10
5. Turrite di Gallicano	a)	Serchio	42,00	9,0
6. Lago Sucotto	a)	Serio	15,20	0,4
7. Ogna Supriore	a)	Serio-Adda-Po	26,60	
8. Ogna Inferiore	a)	Serio-Adda-Po	11,50	
9. Gurzia	b)	Dora Baltea	50,00	8,0
10. Val Molinara	a)	Pioverna-Adda-Po	17,00	
11. Valla (L.Scuro)	a)	Bormida	47,00	23
12. Lago Campelli	a)	Serio	20,00	1,2
13. Rimasco	a)	Sesia-Po	30,50	
14. Tul	a)	Tagliamento	40,10	1,9
15. Orichella	a)	Neto	36,20	9,5
16. San Colombano	a)	Avisio-Adige	19,00	
17. Ceppo Morelli	b)	Toce (Ticino)	46,00	10,5
18. Comelico	b)	Piave	66,50	31,5
19. Moledana	b)	Adda Alpina	42,60	12,8
20. Malciaussia	b)	Stura di Viù	30,50	7,3
21. Rocchetta	b)	Magra	76,00	49
22. Sottosella	a)		57,75	24,3
23. Giaredo	a)	Magra	27,50	1,8
24. Panigai	b)	Adda	41,50	5
25. Provvidenza	b)	Vomano	52,20	70,8
26. Novarza	a)	Tagliamento	38,75	1,3
27. Santa Giustina	b)	Adige	152,50	112
28. Ganda	b)	Adda Alpina	30,00	6,0
29. Zolezzi	a)	Sturla-Entella	19,00	

Reservoir		Year built	Bodies to which the Dam belongs	
Storage capacity $m^3 \times 10^6$	Max. Level m.a.s.l.			
0,150	259,15	1901	Filatura Cascami Seta	
4,00	379,25	1907-1909	Sade	Overflowing
0,87	514,00	1913-1914	Selt-Valdarno	
5,78	567,00	1914-1917	Soc. Lucana I.I.	
0,89	298,00	1915-1916	Selt-Valdarno	Overflowing
0,45	1.863,50	1919-1923	Az.El.Crespi	
0,150	-	1922	Villa Carlo e F.lli	
0,150	-	1922	Villa Carlo e F.lli	
1,249	427,50	1922-1925	Ovesticino	In reinforced concrete-overflowing
0,006	-	1923	Soc.An.Orobia	
2,89	280,00	1923-1925	A.F.L. Falck	Overflowing
0,47	2.044,15	1924	Az.El.Crespi	
0,200	387,50	1925	Soc.Idr.Valsesia	
0,80	268,10	1925-1928 1947-1949	Soc.El. del Tul	Overflowing
0,20	795,90	1926-1928	Meridionale	In reinforced concrete-overflowing
0,600	-	1928	Soc.E.Jacob	
0,47	780,75	1929	Montecatini	
2,08	830,00	1930-1931	Sade	
0,11	909,00	1930-1931	A.F.L. Falck	Overflowing
1,15	1.805,00	1932-1933	Ovesticino	In reinforced concrete-overflowing
5,00	403,50	1935-1937	A.F.L. Falck	
9,60	153,00	1937-1939	Sade	In reinforced concrete-overflowing
0,12	362,00	1940-1941	A.F.L. Falck	Overflowing
0,12	704,00	1941	Orobia	Overflowing
2,00	1.060,00	1941-1943 1946-1947	Terni	Overflowing
-	997,65	1946-1947	Sade	Overflowing
182,81	530,00	1946-1950	Edison	
0,071	913,00	1947	A.F.L. Falck	Overflowing
0,061	352,00	1923	Cons.Idr.Monte Aiona	

TABLE 2. DOUBLE-CURVATURE (DOME) DAMS

Name of the Dam	Type of the Dam	River Basin	Dam	
			Maximum Height m	Volume $m^3 \times 10$
1. Osiglietta	D.-C. O.	Bormida	76,80	75,00
2. Fortezza	D.-C. O.	Isarco (Adige)	63,50	15,00
3. La Maina di Sauris	D.-C. O.	Tagliamento	136,15	100,32
4. Bau Mandara	D.-C. O.	Alto Flumendosa	20,30	1,30
5. Monte Zovo (Avandiga)		Piave	28,40	1,43
6. Ponte Racli	D.-C. O.	Livenza	75,35	18,00
7. Valle di Cadore	D.-C. O.	Piave	61,25	4,607
8. Val Gallina	D.-C. O.	Piave	92,37	99,10
9. Pezzè di Moena	D.-C. O.	Avisio	28,30	1,50
10. Barrea	D.-C. O.	Sangro	62,75	4,5
11. Carboi		Carboi	47,00	32,00
12. Isolato	D.-C. O.	Mera	38,50	6,50
13. Senaiga	D.-C. O.	Senaiga-Cismon--Brenta	68,00	-
14. Corlo	D.-C. O.	Cismon-Brenta	71,00	-
15. Barcis	D.-C. O.	Cellina-Meduna--Livenza	50,15	9,00
16. Grotta Campanaro	D.-C. O.	Melfa-Liri-Garigliano	49,00	5,40
17. Rio Freddo	D.-C.	Stura di D-Rio Freddo	40,5	32,2
18. Forra dei Cami ni(Stramentizzo)	D.-C.	Avisio	63,00	24,0

Reservoir		Year built	Bodies to which the Dam belongs	
Storage Capacity $m^3 \times 10^6$	Max.Level m.a.s.l.			
13	637,00	1937-1939	A.F.L. Falck	
3,35	722,50	1938-1940	Ferrovie Stato	
73	980,00	1942-1947	Sade	
0,31	803,30	1942-1948	Soc.El. Sarda	
-	-	1946-1949	Sade	
25	313,00	1948-1951	Snia Viscosa	
4,90	706,50	1949-1950	Sade	
6,40	677,00	1949-1951	Sade	
0,46	1.197,00	1950	Trentina di El.	
24,30	973,00	1950-1951	Com.Sangro-Sme-Terni	
35,5	179,00	1950-1951	Ente Rif.Agr.Sic.	
1,76	1.246,80	1951-1952	Edison	
5,750	404,00	1954	S.I.I.A.	
42,600	268,00	1954	S.I.I.A.	
21,990	402,00	1954	Sade	
0,394	783,00	1954	Soc.Alto Liri	
0,325	1.202,50	1954-1955	Edison	
10,000	789,50	1954-1956	Soc.Ind.Trentina	

LEGEND : D.-C. = Double-curvature.
O. = Overflowing

TABLE 3. ARCH-GRAVITY DAMS

Name of the Dam	Type of the Dam	River Basin	Dam	
			Maximum Height m	Volume $m^3 \times 10^3$
1. Pontebba	A.-G.	Tagliamento	23,50	1,55
2. Fusino	A.-G.	Adda	59,00	42,00
3. Furlo	A.-G.	Metauro	62,22	-
4. Pieve di Cadore	A.-G.	Piave	112,00	377,00
5. Cecita	A.-G.	Crati	55,00	60,00
6. Val d'Auna	A.-G.	Talvera (Adige)	53,00	54,00
7. Prà da Stua	A.-G.	Aviana - Sorne (Adige)	42,87	30,50
8. Publino	A.-G.	Adda Alpina	42,00	34,00
9. Forte Buso (Travignolo)	A.-G.	Avisio	110,00	260,00
10. Lago di Valsoera	A.-G.	Orco (Po)	54,00	37,00
11. Beauregard	A.-G.	Dora Baltea	132,00	430,00
12. Fiastra	A.-G.	Fiastrone-Chienti	87,00	160,00
13. Pian Telessio	A.-G.	Rio Piantonetto-Orco - Po	80,00	380,00
14. Piaganini	A.-G.	Vomano	45,50	26,00
15. Monte su Rei	A.-G.	R.Mulargia-Flumendosa	100,00	-
16. Bellicai	A.-G.	R.Bellicai-R.Corongiu	30,00	-
17. Cancano II	A.-G.	Adda { 1° f. / 2° f.	132,00 / 172,00	490,00 / 586,00

Reservoir		Year built	Bodies to which the Dam belongs	
Storage capacity $m^3 \times 10^6$	Max.Level m.a.s.l.			
0,01	616,00	1901-1902	Friulana di El.	
0,20	1.153,50	1920-1924	A.E.M. - Milano	
1,800	174,28	1921-1952	Unes	
68,50	683,50	1946-1949	Sade	
108,22	1.142,25	1949-1951	Meridionale	
0,412	916,00	1950-1951	Trentina di El.	
1,50	1.041,50	1950-1951	S.A. El.Valeggio sul Mincio	
5,00	2.134,40	1950-1951	A.F.L. Falck	
32,10	1.458,00	1950-1952	Smirrel	
8,40	2.412,00	1950-1953	A.E.M. - Torino	
72,00	1.770,00	1951	SIP	Under construction
20,400	640,00	1955	Unes	
24,000	1.917,00	1955	A.E.M. - Torino	
0,550	400,00	1955	Terni	
304,00	-		Ente Autonom.Flumendosa	Under construction
1,02			Soc. Monteponi	
115,00 240,00	1.898,00 1.938,00		A.E.M. - Milano	Under construction
				LEGEND : A.-G. = Arch-gravity dams.

to the height of the layer in the case of arch dams with considerable overhang.

As I said at the beginning, the greater skill necessary for the concrete for the more refined structures is no great handicap in Italy, since there is a high standard of skill in our country and many contractors who have specialised in the construction of dams have reached a high level in the utilization of labor. The same is true for the layout; many of our technicians are rather clever in this work.

Of great help to the smooth running of the concrete placement and to its constant good quality are the preliminary studies of mix design which in Italy are generally carried out in the specialised and well-equipped laboratories of University Foundations and of private concerns.

Now with regard to the cost. As to the single elements of the cost of concrete in itself (i.e. apart from the influence of the cost of installations), undoubtedly some of them are a little higher for arch dams than for gravity dams if only for the fact that there must be a higher unit strength. However, this extra cost reduces itself in practice to the higher percentage of cement, which does not normally exceed a plus-difference of 50 kg, per cubic meter and some slight extra cost for formwork. There is another almost negligible difference due to the heavy vibration rendered inevitable by the Italian practice of reducing the water-cement ratio to as low a value as possible. With regard to concrete mixtures I would like to draw attention here to what I also pointed out at the recent Icold Paris conference: Italian mixtures of pozzolanic cement (now commonly used in nearly all dams) include also the pozzolana calculated in the weight as cement, so that the values involved are not comparable with analogous American values. For example, the mass concrete used in the construction of Hungry Horse Dam was made up in the proportion of 111 kg of cement and 56 of fly ash, total 167 kg, as against 200 kg of the ferrico-pozzolanic cement used in the Pieve di Cadore dam.

It must be born in mind that in Italy dams are built very often in high mountain areas or at any rate in very cold regions where the problem of freezing is of first importance. The consequence of this is that even for a gravity dam there is need for special mixtures of concrete and special care in pouring, at least in the neighbourhood of the faces.

Some difficulties in the construction of arch dams may be due to form, on account of the curvature of the surfaces; and to the fact that the curve of the axis in some cases renders some forms of transport and concrete-laying inappropriate. This last difficulty, however, can easily be overcome by widening the operating range of the cable-way equipment or, as was done in the great Piave dam, in that of Travignolo and others, with the use of movable distribution towers on curved rails which follow the ground plan of the structure. Moreover, it should be remembered that gravity dams more often demand longitudinal joints and artificial cooling of the concrete, both of which are only a few cases necessary for arch dams and less rarely also for arch-gravity dams. Furthermore, the diagram of the pouring as a function of height is often more favorable to good utilization of the installations in the case of curved dams than in that of a gravity dam, on account of the fact that the maximum of surface comes in the middle of the period of construction.

At any rate, to conclude the question of costs of many arch and dome dams recently built in Italy, in comparison with gravity structures, we may state that the difference in cost is, in practice, non existent for arch-gravity dams: in short, it is reduced to the influence of the lower concrete volume in the incidence of the building installation. In other words, the fundamental factor in the unit cost is the volume of concrete, as already made clear by others (see Biadene, Icold report). The unit cost rises, on the other hand, of course, for thin arch and dome dams, but even for these, fundamentally, in inverse ratio to the volume.

In our country, however, the total cost for the arch solution as against the gravity solution cannot, generally speaking, work out higher, because the saving in volume is generally, in percentage terms, far higher than the percentage increase in the unit cost, so that the total cost shows always an actual and substantial saving.

Some Particular Considerations

I think that the above summary is enough to make clear the conceptions and the development of designing and construction of arch dams in their various types in Italy. The general ideas explained above are naturally applied to the single cases which present themselves in practice with some variations according to the ideas of the designers and builders.

Indeed, while in a certain sense one can speak of uniformity of trends in the field of analysis, one cannot say that any such uniformity exists in particular applications, although a certain tendency to standardisation is discernible.

I would like here to give some explanation of a few principles on which many Italian designers and builders have worked and which, as it appears from my experience, are often rather difficult for foreign technicians to rightly understand.

I think, therefore, that it would be of some use to mention these principles, again making it clear that they do not however, constitute normal Italian practice and still less Italian compulsory regulations: they merely reflect a group of ideas and the fruit or the practical experience of some designers.

1) It is often held that Italian engineers tend to adopt symmetrical structures for arch dams: this affirmation does not perhaps express the matter accurately. It is merely a question of suitability which seems to us rational.

The arch and the dome, even if calculated as being composed of a series of independent elements, must always be considered as continuously curved three-dimensional plates as they actually are.

Assymmetry causes a less uniform distribution of stresses with higher maximum and minimum values, than in the symmetrical structure having the same mean stresses with the maximum values (usually fairly high) proper to arch dams. It may therefore be that an assymmetrical solution, while having a smaller surface development than a symmetrical one, might involve greater volume, because, should the stresses be logically kept within the same limits as in a symmetrical solution, often the thicknesses would be substantially increased in certain parts of the dam.

Moreover, it should not be forgotten that the analysis of an assymmetrical structure is more difficult and more delicate in conception, and therefore the design must depend to a great extent on model tests, unless more conservative stresses are adopted.

In conclusion, it is not a question of taking up an absolute position in favour of symmetry, but of choosing between solutions which may both be technically sound but differ in cost: the choice must be based on economic criteria.

At any rate, we must also bear in mind the fact that this geometrical symmetry applied to the arch, would remain a mere desire from a functional point of view, as generally the characteristics of the rock and the thermal conditions are not equal on both sides of the location; therefore, the symmetry attained in the shape would not be reflected in the static performance.

2) If the conception of symmetry has a relative importance, we give a greater importance to that of geometrical continuity of the structure. The conception of geometrical continuity, however, must be applied, in my view, not only to the arch itself (whose horizontal and vertical sections must vary gradually, as all the geometrical parameters of the structure must do) but also, at least to some extent, to the supporting surfaces.

Our point of departure is that every more or less abrupt variation in the perimeter of the curved plate gives rise to considerable concentrations in the stresses, with static repercussions also on part of the adjacent areas, resulting in marked static difference in the behavior of the structure.

Moreover, this principle of a relative continuity of the surface of the abutments may cause serious increases in the excavations as well as in the volume of the arch, when there are abrupt local variations of the natural profile. For example, a deep recess in the abutment, should a completely continuous perimeter be sought, would render necessary considerable increases of volume also in the adjoining zones. In such cases we are led to eliminate the discontinuity by means of a sort of plug which fills in the recess mentioned, the plug being separated from the curved plate itself by a construction joint; and the same holds good for all the parts which, owing to the irregularity of the rock surface, do not follow a reasonably continuous line for the surface of the supports. (Fig. 19.)

I may here mention that, on the basis of this idea of the continuity of the abutment surface, many Italian designers have eliminated the stepping of the rock in arch dam abutments, achieving excavation lines as far as possible continuous, without abrupt changes of relief.

3) Further, it is recognised that the perfect encastrement on which the analysis of arches is often based is only a hypothesis which very rarely corresponds to reality. For a perfect encastrement the perimetral part of the plate should be locked in a trench of solid rock whose side walls should encase the structure to a sufficient depth; and further, the part to be encased and the adjacent one should be strongly reinforced. We can otherwise admit that the need to realize the perfect encastrement would arise only when the curve of pressure comes out of the middle third of the abutment.

In any case, we think it is only fair to admit that, in general, the abutment of the structure tends to a simple support. It certainly is so when the plate rests on a plug.

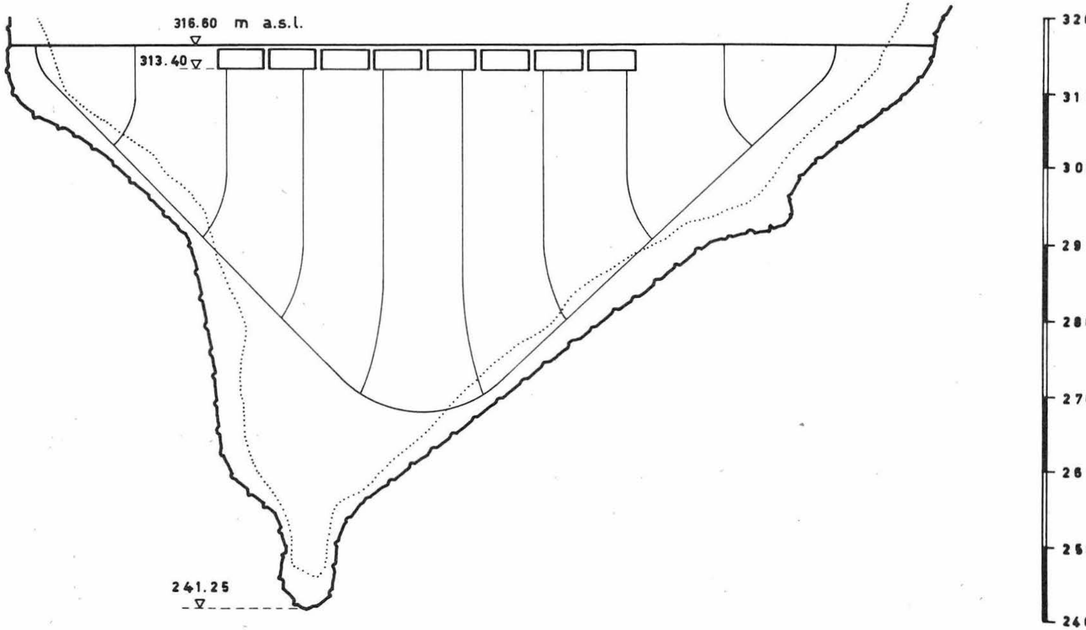
Furthermore, the adoption of the simple support permits greater freedom and ease in carrying out the abutments, which in many cases results in a saving of cost. Considering this fact and given the conception of the continuity of the surface of the supports, owing to which in certain points, where the surface is irregular, the part against the rock is separated from the structure by means of a joint, we have thought that it was logical to extend the joint itself to the whole perimeter: that is, to place the arch or dome structure, designed with a continuous perimetral line, on a kind of distribution saddle or cradle running all along the surface of supports. This saddle is separated from the structure by a construction joint, the "perimetral joint."

The curved plate thus simply rests on the rock by means of this

PONTE RACLI DAM
ON THE RIVER MEDUNA

DEVELOPED ELEVATION
ALONG THE MIDDLE FIBRE OF THE ARCHES

FROM DOWNSTREAM



distribution structure on saddle which we call, through an analogy from architecture, the "pulvino" or cushion. The "pulvino" makes a clear break between the dam properly speaking and the rock, of which it actually becomes to all intents and purposes a part. (Fig. 20)

4) What in our view are the advantages of this arrangement? I will list them briefly:

a) From the static point of view, there is a reduction of the tensile stresses which in certain conditions (for instance: thermal variations, shrinkage, earthquakes) come into play near the abutments in the upstream face of the arches, especially in the lower, thicker ones, and at the foot of the central cantilevers; indeed, in the neighbourhood of the joint one can consider that the tensile stresses are completely suppressed because the section is free and works only in compression.

b) In this way the danger of cracking on the upstream face near the abutments becomes more remote: in short, we ourselves pre-determine the zone where we prefer eventual cracks to come up, and where we can face them with adequate protection from infiltrations which might give rise to abnormal stresses in the body of the dam and, in any case, prove detrimental to its conservation.

c) Similarly, any tendency to the separation of the plate from the rock in the upstream part subject to tensile stresses, is limited.

d) The construction of a joint at the abutment at the very point of contact with the rock would not be easy and, in any case, would require the construction, rather unreliable, of a small concrete block adhering to the rock in order to realize the joint between two concrete surfaces. The perimetral joint is instead resolutely planned and rationally constructed at a short distance from the rock.

e) The constitution of the perimetral joint allows us to give the surface between the plate and the supporting structure a transverse shape which seems to us naturally appropriate to the elastic functioning of the plate. We generally prefer a cradle-shaped surface, or at any rate a broken surface following approximately a cradle surface and crossing perpendicularly the upstream and downstream faces — One might even think of something like a continuous hinge along the whole perimetral joint. (Fig. 21.)

f) From a practical point of view, it appears also evident from the above mentioned considerations that the "pulvino" facilitates a good geometric definition of the curved plate and a good connection of it with the surface of abutments which often can be defined only when the excavations are complete. Otherwise the final geometric characteristics of the plate could only be fixed then.

g) From the constructional point of view, one can cast the "pulvino" in stages, days or weeks or even months in advance of the dam blocks next to it. This seems to us to be a particulary rational and practical system; furthermore, it can simplify the carrying out of the grouting. In addition, the presence of the "pulvino" facilitates the operations of joining through grouting to the rock.

6) For all the reasons explained in the foregoing paragraphs we have been led to adopt the perimetral joint in several Italian dams.

I already spoke of the original concept and the first application of this device to the Osiglietta dam, owing to the converging ideas of many experts; I

A SKETCH OF THE PROFILE
OF A PERIMETRAL JOINT

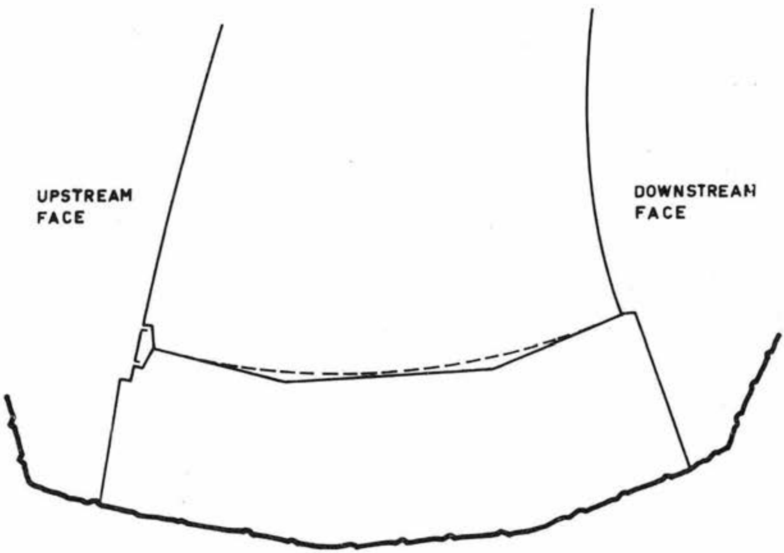
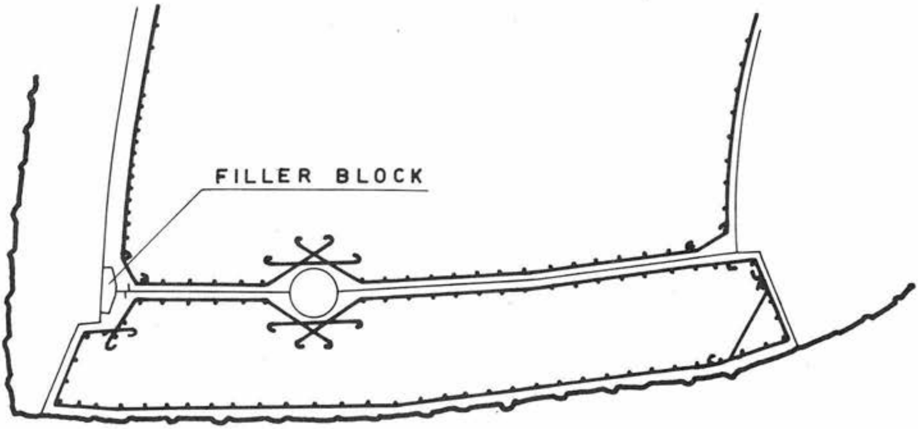


FIG 20

LUMIEI DAM
DETAILS OF THE PERIMETRAL JOINT



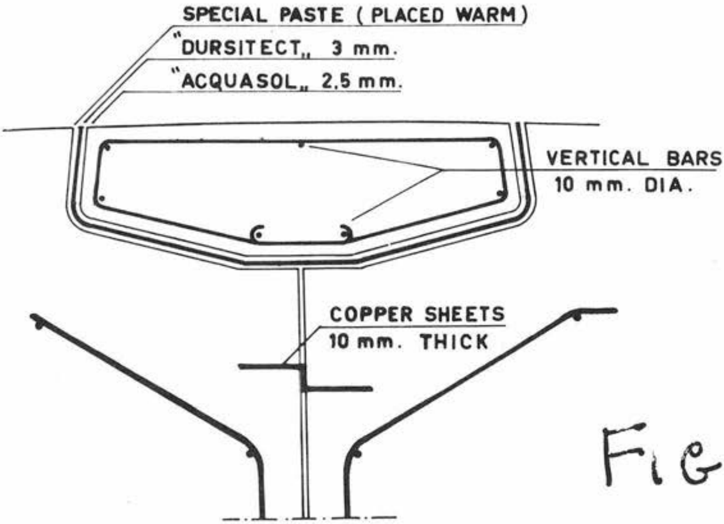


Fig 2

must add that a previous proposal by Mr. Scalabrini for the design of Rocchetta dam had to be abandoned.

After Osiglietta, the joint was applied to many other Italian dams, principally those of the Veneto area and various others among those already placed on record. The most of those in the Vento, built mainly by the Società Adriatica di Elettricità, to which the author belongs, were worked out in collaboration with Prof. Oberti. The others were built by various public bodies and are the work of other designers, including especially Prof. Arredi.

We have been asked many times for news of the practical results of the perimetral joint. Of course, in order to say the last word on these practical results, it would be necessary to build the same dam both with and without the joint so as to be able to compare the behaviour of the two, which is obviously impossible. We therefore had to confine ourselves to comparing the dam models carried out on the two principles.

We can say that a comparison of the Lumiei dam models, as reported by Prof. Oberti, proved that the joint provided remarkable, if not decisive, advantages: the stresses in the structure were lower and better distributed. This result has been borne out—even though the evidence given by the same construction without the joint does not exist—by the excellent static behaviour of the actual structure.

All the other dams with perimetral joints have also given very good results.

This does not mean that without the joint the results would not have been good. The conclusion is clearly positive without excluding other possibilities.

7) Another question which is frequently put to us is whether there is not the possibility, which ought to be feared, of a slipping movement of the dam in a downstream direction on the surface of the joint. According to our conception, there should be no such possibility. The joint is designed "spatially" so as to obviate any chance of movement in the downstream direction; it is roughly normal to the line of pressures and could not be therefore a joint allowing sliding movements. It could exercise its function only on the surfaces tangential to the faces of the dam, thus facilitating the structure's "respiration" i.e. the tiny rotations of the curved plate in respect to the pulvino.

There was one particular case, that of the Val Gallina dam, in which an early model, built without taking into due account the structure weight, had indicated the possibility of sliding downstream; we limited in that case the joint in the base zone of the dam to only the upstream third of the section.

8) I may add that, from the functional point of view, the "pulvino" could be built of concrete with those elastic characteristics which render it able to act as an intermediary between those of the concrete of the arch and those of the rock, so as to increase still further its capacity to work as a transition structure between the arch and the rock: a function already implicit in its conception from the structural and position points of view. One of the functions of the "pulvino" is precisely that of distributing and reducing the stresses in the rock: its form and its dimensions can indeed be adapted (with appropriate widening and re-inforcement) to the characteristics of the rock. Inversely, it is capable of reducing the repercussion on the curved plate of the discontinuities and irregularities in those elastic characteristics along the surface of support.

In particular the "pulvino" can be thickened, widened and, if necessary, strengthened by adequate steel re-inforcements, in order that it can work as a "bridge" over the weaker areas of the abutment surface. Admittedly, this bridge-function can also be taken over by the adjacent areas of the curved plate itself but the presence of the "pulvino" facilitates this function: the "pulvino" plays a more defined part which can be more easily adjusted and analysed so that the plate confines itself to absorbing a more limited proportion of the abnormal and additional stresses.

9) Another device we use in the arch dams, which is far from being peculiar to our country, but from the frequency of its use can be called a normal conception in Italian technique is that of the discharge of floods over the crest. Our assumption is that for actual safety it is necessary that among the various discharge provisions, at least one ought to be an open spillway followed by an open conduit. Indeed, the consequence of a closed conduit would be a sudden and uncontrollable raising of the water level whenever the peak-flood exceeded the calculated maximum. A crest-spillway probably constitutes in many cases the most economical solution for the open discharge works and one which we strongly recommend. For the above consideration, in many arch and dome dams in Italy the crest has been utilized for the discharge of floods exceeding those which can be normally absorbed by other discharge devices.

Of course—should there not be any special device provided against scour and erosion—the rock must be capable in these cases of taking the full impact of the discharge, although it generally lasts for only a short period. We are actually convinced that, when it is a question of discharging only exceptional peak flows and where the foundation rock is strong enough, it is in some cases not even necessary to arrange for special downstream protection.

I already mentioned the old Cismon dam and the Sottosella dam (1939). There are several more modern examples. In some cases, where a discharge exceeds the calculated maximum, the excess flow has also been allowed to spill over a crest not expressly designed for that function (for instance, Pieve di Cadore).

All these ideas which I have brought to your knowledge are naturally open to discussion. I thought it my duty to submit them to the most full and frank discussion of our eminent American colleagues: I can only say that our practical experience to date is completely successful.

The author wishes to express acknowledgment and appreciation to Professors. Oberti and Arredi, to Messrs. Sensidoni and Scalabrini, and to his staff, whose important additions and suggestions have been invaluable in the drawing up of this paper.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: DESIGN OF THE KAMISHIIBA ARCH DAM

C. C. Bonin,[1] A.M. ASCE, and H. W. Stuber,[2] A.M. ASCE

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee,

Since the last symposium on masonry dams (April, 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time, it is not known exactly how many papers will be printed from the Symposium. So far, twelve papers have been approved: "Arch Dams: Their Philosophy," by Andre Coyne (Proc. Paper 959); "Arch Dams: Trial Load Studies for Hungry Horse Dam," by R. E. Glover and Merlin D. Copen (Proc. Paper 960); "Arch Dams: Portuguese Experience with Overflow Arch Dams," by A. C. Xerez (Proc. Paper 990); "Arch Dams: Theory, Methods, and Details of Joint Grouting," by A. Warren Simonds (Proc. Paper 991); "Arch Dams: Santa Giustina Single-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 992); "Arch Dams: Measurements and Studies on Santa Giustina Dam," by Claudio Marcello (Proc. Paper 993): "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 994); "Arch Dams: Isolato Double-Curvature Arch Dams," by Claudio Marcello (Proc. Paper 995); "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs," by Claudio Marcello (Proc. Paper 996); "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira (Proc. Paper 997); "Arch Dams: Development in Italy," by Carlo Semenza (Proc. Paper 1017); and "Arch Dams: Design of the Kamishiiba Arch Dam," by C. C. Bonin and H. W. Stuber (Proc. Paper 1018).

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

Note: Discussion open until November 1, 1956. Paper 1018 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 3, June, 1956.

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SYNOPSIS

This paper outlines the design of the first arch dam to be built in Japan. Discussed in detail are geologic conditions at the site, basic assumptions for the design, design criteria, and the allowable stresses used. Also included is a brief discussion of the spillway adopted for this project.

INTRODUCTION

On June 25, 1955, the 90 000 kilowatt capacity Kamishiiba Hydro Electric Development was formally dedicated by the Kyushu Electric Power Company of Japan. This development includes the first arch dam in the Far East and is located on the upper reaches of the Mimi River in the central portion of Kyushu, the southernmost of Japan's four principal islands. Figure 1 is an aerial photograph of the dam. The area is mountainous with ranges up to 1500 meters in elevation cut by deep, steep sides, V-shaped valleys. Forests and dense vegetation cover the entire area. Annual rainfall averages approximately 2 500 mm of which more than one-half occurs during May, June and July. October through January is the dry season during which about one-tenth of the rainfall occurs. Typhoons with torrential rains causing short duration high crested floods in the rivers of the area generally occur in August and September of some years. The average monthly flow in the Mimi River at Kamishiiba varies from about 8 cubic meters per second in mid-winter to about 40 cubic meters per second in mid-summer. Five hydro-electric developments utilizing gravity dams with little or no seasonal storage were previously built on the Mimi River below Kamishiiba. The smallest of these has been by-passed and eliminated by the Kamishiiba Hydro Electric Development.

Kamishiiba Hydro Electric Development

Just prior to World War II, Hassoden (The Japan Electric Generating and Transmission Company) began site investigations and power studies for a development with a 90 meter high gravity dam at Kamishiiba. When it became possible to resume these studies after the war, a 110 meter high gravity dam was considered in an attempt to make the development economically feasible. However, the value of the additional energy made available was offset by the cost of the added mass of concrete in the larger gravity dam, and the development did not appear too attractive.

In 1950, Hassoden retained the American firm Overseas Consultants, Incorporated, to review the engineering practices then being used in the Japanese electric power industry. They felt that their engineering practices had not kept pace with the rest of the world during their self-imposed period of semi-isolation and World War II. While assigned to this work, a group of American engineers visited Kamishiiba and suggested that an arch dam be considered for that site. This prompted the Japanese engineers to form a committee to make a comprehensive study of arch dam design and construction. Up to this time they had, in general, rejected the idea of building arch dams in Japan, owing to apprehension of the ability of such thin structures to withstand the frequent and sometimes severe earthquakes occurring throughout the country.

Early in 1952, Overseas Consultants, Inc., was retained by the newly formed Kyushu Electric Power Company to investigate, analyse and study the height and type of dam to be utilized, and to make recommendations on other engineering matters all related to the Kamishiiba Hydro Electric Development. Five alternative heights of dam were analyzed for both arch and gravity types; various locations were considered. Thorough consideration was given to the geology of the site. Aggregate, cement, formwork, manpower, equipment, layout, type of spillway and earthquakes during and after construction were additional factors considered in the study of the problem. As a result, it was recommended that a 110 meter high constant-angle type arch dam be built at Kamishiiba. A significant factor in the choice of an arch dam over a gravity dam was the fact that one year's time would be saved in construction—very important in a power-short country.

At the damsite the Mimi River has cut through a series of palaoezoic sediments composed of very indurated sandstone and slates. These sediments have a general N 70° E strike and dip steeply to the north, and are part of a palaeozoic zone which spreads NNE to SSW across central Kyushu. At the damsite a very thick zone of graywacke covers nearly the entire arch dam foundation, slate being encountered only on the upper 25 meters of the left abutment. However, the graywacke is interspersed by sparsely located thin layers of slates. These were not considered detrimental. The boundary plane between the graywacke and slate has been faulted, the slate along the fault plane being shattered, crushed and considerably weathered. This zone of crushed slate is interspersed with layers of unsound mylonitic rock, graphite and clay. The height of the dam was limited to 110 meters largely because of this zone of unsound crushed rock. This material was excavated and replaced by a concrete mass to form a thrust block approximately 25 meters high. The recommended location of the dam was established primarily by the geology of the site, secondarily by the surface topography.

The Arch Dam

General Description

In August 1952, Overseas Consultants Inc., was retained by the Kyushu Electric Power Company to supervise the design and construction of the Kamishiiba arch dam. At this time work was begun on the final design for the dam.

Kamishiiba Arch Dam is a constant angle type arch dam with a design base circle radius of 143.50 meters and a top central angle of 119° 47'. It has a design arc length of 300 meters measured along the base circle between the right rock abutment and the concrete thrust block on the left abutment at Elevation 483 meters, the top of the dam. The final actual length is 308 meters. The dam is shown in relation to the construction plant area on Plate No. 1. The maximum design height of the dam was 110 meters assuming Elevation 373.0 as the base of the dam. (The lowest point of the actual base was found to be Elevation 369.69, thus making the actual height 113.31 meters). It has a virtually constant central angle between Elevations 483, and 420, varying by no more than five degrees from 116°. Below Elevation 420 the central angle diminishes progressively as shown in Plate No. 2. The principal dimensions and arch data listed are given in meters: t and r represent the uniform thickness of the arch lamination and the centerline radius of the lamination, respectively. The dam was constructed in blocks 20 meters wide, along the base circle, with the radial construction joints grouted in order to provide as nearly a monolithic structure as possible.

The following tabulation gives the principal water surface levels and other pertinent data:

Maximum design flood with wind and seismic waves	Elev 483.0 m
Normal high reservoir level	Elev 480.0 m
Minimum reservoir level	Elev 435.0 m
Drawdown	45.0 m
Maximum tailwater	Elev 398.0 m
Minimum and normal tailwater	Elev 373.0 m
Maximum silt	Elev 420.0 m
Base of dam (assumed)	Elev 373.0 m

The main body of the dam was founded on the hard compact graywacke at the damsite. However, the foundation rock of the upper left abutment was such that a 25 meter high 30 meter long thrust block was required. See Plate No. 6. At the junction of the arch and the thrust block a concrete gravity cut-off extending to the north to sound rock was adopted. The graywacke on the upper one-third of the right abutment was found to be more closely jointed and deeply weathered than expected, requiring extensive excavation. It was estimated that 225 000 cubic meters of overburden and rock were to be excavated for the dam; actual volume removed was 268 000 cubic meters.

The arch dam contains 323 000 cubic meters of concrete, the thrust block and gravity cut-off 30 000 cubic meters, the spillway chutes and walls 28 000 cubic meters. The dam impounds a reservoir with a total storage of 91 500 000 cubic meters, of which 76 000 000 cubic meters is usable for power generation.

Basic Assumptions

For the design of Kamishiiba Arch Dam it was assumed that:

1) The foundation rock at the site would be homogeneous and strong enough to carry the applied loads with stresses well below the elastic limit.

2) The concrete in the dam would be homogeneous and strong enough to carry the applied loads with stresses well below the elastic limit; Hooke's law applied.

3) The dam would be thoroughly embedded in the rock so that the arches and cantilevers may be considered as integral with the foundation.

4) The vertical construction joints would be grouted before the water load is applied so that the dam may be considered to act as a monolith.

5) Effects of creep have been adequately allowed for by using a somewhat smaller value of the modulus of elasticity than would otherwise be adopted.

In addition to these basic assumptions, more specific assumptions for analyzing arch and cantilever elements will be presented later.

Loads

A clear line of demarcation should be drawn between "external loads" and "elastic deformations," although they are intimately related. "External loads" are forces or influences acting on the outside surfaces of the dam. "Foundation deformation" may be considered to satisfy this definition; however, it has been found preferable to list this under "elastic deformations" to be discussed later. Temperature drop which is an "elastic deformation" has been considered to be an "external load." Its counterpart "shrinkage" is an "internal influence"; however, it is difficult to separate the two, and it is not necessary to do so when the vertical joints of the dam are grouted after the concrete has cooled to or below its mean annual temperature and the effect of further shrinkage may be neglected. Therefore, negligible residual shrinkage is considered to be included in temperature drop for the individual laminations in this design. The active "external loads" that were considered are static water pressure, silt pressure, inertia of masonry during earthquake, dynamic water pressure during earthquake, weight of dam, uplift pressure, weight of water on surface of dam and temperature variation from the mean annual value.

Horizontal loads are resisted by both arch and cantilever elements. They were apportioned by trial until coincidence of deflections of both elements was obtained. Formulas used for obtaining total unit loads are:

Static water pressure	$= w_1 x$	(tons / M^2)
Silt pressure	$= C_e W_3 X^1 = 0.66\ X^1$	(tons / M^2)
Inertia of masonry	$= K_1\ W_c\ t \cos\theta$	(tons / M^2)
Inertia of water	$= 7/8\ w_1\ k_1\ hx \cos\theta$	(tons / M^2)

in which

w_1 is the specific gravity of water (1.0)

w_2 is the specific gravity of sand in air (1.8)

w_3 is the specific gravity of sand in water (1.1)

w_c is the specific gravity of concrete (2.3)

C_e is a constant $= 0.6$

x is the depth of water from the surface elevation to the elevation of the point under consideration

h is the maximum depth of water behind the dam

K_1 is the earthquake acceleration = 0.12 reservoir full
= 0.06 reservoir empty

x^1 is the depth of silt measured from Elevation 420 to elevation of point under consideration

t is thickness of arch lamination

θ is the angle between the assumed direction of earthquake motion and a normal to the face of the dam at the point under consideration.

For design purposes the elevation of the water surface, reservoir full, was taken to be the same as the top of the dam or Elevation 483.0 because of the effects of abnormal floods, wind and seismic waves. The formula for silt pressure was taken from the publication, "Design Standard for High Dams for Hydro-Electric Developments" by the Natural Resources Board of Japan. In the factor $C_e w_3 x^1$;

$$w_3 = w_2 + w_1 + vw_1$$

where v equals the percentage of voids, here taken as 30%.

Earthquake forces were assumed acting in a downstream direction with the reservoir full thereby increasing the stresses due to water load and temperature drop. The condition of reservoir full with earthquake forces acting across the valley normal to the thrust block was also investigated. For reservoir at low water level the earthquake forces were assumed to act in upstream direction.

Vertical loads act on the cantilever elements only. The cantilever elements are bounded by radial lines one meter apart on a circumferential plane passing through the center line of the Elevation 483 lamination. The actual weight of the element was computed assuming the specific gravity of the concrete as being 2.3. The weight of the water on the cantilevers was determined at each elevation under consideration and used as a vertical load. Uplift pressure has been divided into two parts; namely, that which acts on the base of the dam at any elevation and that which acts on the overhanging portion of the dam between the heel and the furthermost projection upstream of the heel of the dam. The uplift pressure on the base of any cantilever was computed on the basis of 35% of the pressure at the heel of the dam and varying uniformly to zero at the toe of the dam. Where the tensile stress of the cantilever was higher than the allowable stress a crack was assumed and the uplift over the cracked area was computed for 100% of the head and held constant over the cracked area of the concrete. Over the uncracked area the uplift varied from 35% to zero.

Temperature deflections of the arch elements were added to the water load deflections before adjusting the arch and cantilever movements. The temperature load was applied directly to the arch elements only and not directly to the cantilever elements since temperature changes in vertical sections cause negligible horizontal movements. Increases in temperature in the arch elements work against the water load and were neglected except for the case of minimum water level. Decreases in temperature in the arch elements work with the load and were included in the analysis. The maximum probable decrease in temperature below the temperature existing at the time of grouting the joints is the change which was included in the stress analysis. The vertical joints were to be grouted after the concrete had cooled to the mean ambient temperature of 15.6° C.

Temperature changes vary with the thickness of the arch and were computed from the equation

$$F = 340 / t + 8$$

in which F is given in degrees Fahrenheit and t is the thickness of the arch ring in feet. They were assumed constant over an entire arch element. To prevent the formation of cracks in the dam to the greatest possible extent it was necessary to limit the temperature rise of the concrete as much as possible. To accomplish this, a concrete mix with minimum amounts of moderate heat type cement was used. Air entraining admixture was added to improve the workability of such lean concrete mixes. Other important considerations were the rate of placing concrete and the height of the lifts. Concrete lifts that were in contact with foundation rock were limited to one meter, elsewhere two meter lifts were specified. Before grouting contraction joints the concrete was to be cooled to mean ambient temperature by circulating refrigerated water through systems of cooling coils embedded in the concrete lifts.

"Elastic deformations" were included in analyzing the stresses of Kamishiiba Arch Dam. However, certain elastic deformations counteracting water load stresses have been omitted. Among these are swelling of concrete and temperature rise. Present knowledge indicates that they would be beneficial in most cases, but they have been included only in the case of low water where they would, theoretically, be detrimental. The elastic deformations that were included in the design are:

Moment deformations
Thrust deformations
Shear deformations
Foundation deformations

The external loads shorten the arches and introduce thrusts, shears and moments. Stresses so produced are known as rib shortening stresses, radial shear stresses and bending fiber stresses respectively. The effect of Poisson's Ratio was ignored except when evaluating the shear modulus of elasticity. All the effects of the "elastic deformations" mentioned have been included in computing the stresses for the design cases considered. The following physical constants were used.

Temperature coefficient		0.000010 /^{o}C
Modulus of elasticity of concrete		E_c = 200 000 kg/cm^2
Modulus of elasticity of hard sandstone		E_{R1} = 250 000 kg/cm^2
Modulus of elasticity of slate		E_{R2} = 200 000 kg/cm^2
Poisons Ratio, concrete		0.20
Poisons Ratio, rock		0.20
Shear Factor	k = 1.25 radial	1.00 tangential
Shear modulus ratio	n = 3.00 radial	2.40 tangential

Design Criteria

The design criteria described here establish conditions for which complete trial load analysis were made for the completed dam and the conditions for which the stability of the dam was investigated while it was under construction.

To analyze stability during construction the loads considered as acting on the structure, or portions of it, were: weight of masonry and earthquake acting upstream. One of the objects of this study was to determine the height to which each individual block could be built without benefit of the support of adjacent blocks, and to determine the magnitude and variation of stresses at each corner of the base of each block under consideration as the height varied from the foundation to the top of the dam at Elevation 483. Blocks 11-12-13 on the right abutment were considered to be the most critical. Each of these was considered individually, and they were also investigated as a unit.

The blocks were considered safe against sliding and overturning when the following conditions were satisfied.

1) Coefficient of friction equal to or less than 0.80; i.e.

$$\frac{\text{Summation of horizontal forces}}{\text{Summation of vertical forces}} = 0.80$$

2) Shear-friction factor of safety equal to or greater than 4.0; i.e.

$$S_{s-f} = \frac{f\,(W) + rs_a\,A}{P} = 4.0$$

In which
P = Horizontal forces
W = Vertical forces
f = Coefficient of friction
r = Ratio of average to maximum shear on joint
Sa = Unit shear strength = 200 ton/m^2
S_{s-f} = Shear-friction factor of safety
A = Area of section of base

3) $$\frac{\text{Resisting moment}}{\text{Overturning moment}} = 1.2$$

Load conditions for which complete or partial trial load analyses were made are:

1) High water Elevation 480
2) High water Elevation 483 + silt + temperature drop
3) High water Elevation 483 + silt + temperature drop + earthquake inertia downstream
4) High water Elevation 483 + silt + temperatuer drop + earthquake across valley normal to thrust block
5) Low water Elevation 435 + temperature rise + earthquake inertia upstream, no silt
6) Stability During Construction, no water, earthquake inertia upstream.

It was not considered necessary to obtain a complete "Convergence of Adjustments" for each case. The extent of the convergence was limited to radial adjustment only for case 5, and radial adjustment and shear adjustment for case 4. For the basic design, case 3, a complete convergence of adjustments was made in the order of R_1; S_1; T_1; R_2; where:

R = Radial adjustments
S = Tangential or shear adjustments
T = Twist adjustments

For Case 4 the order of convergence was S_1; R_1. The subscripts indicate the order of the adjustments. R_1 for instance, means the first completed radial adjustment.

Case 1 was analyzed because this was considered to be the most probable condition for long periods of time and the one most likely to provide stress evaluations for comparison with the values to be obtained in the future from the Carlson measuring instruments embedded in the dam.

A complete trial load analysis was required for Case 3 because this case involved both static and dynamic loads and was the most severe condition of loading. The results of this analysis are shown in Plate No. 7. Wind and seismic waves were included in the dynamic loads. The height of wave due to earthquake was computed from the formula.

$$h_1 = 0.75 + \frac{F^{1/2}}{3} - \frac{F^{1/4}}{4}$$

in which h_1 is in meters and F is the distance of reach in kilometers. The height of wave due to earthquake was computed from the formula

$$h_2 = \frac{KT}{2} (gH)^{1/2}$$

in which

K = Equivalent horizontal acceleration = 0.12
T = Frequency of earthquake = 0.1 to 0.3
g = acceleration of gravity = 9.8
H = Depth of water = 107 meters

For convenience of computation the effect of the above loads was added to the normal high water level used to evaluate static water pressure on the dam. This resulted in a maximum water level of Elevation 483. Dynamic earthquake forces were computed for water at Elevation 480. Temperature drop is given as a condition for this case because it made the stresses higher. Actually, high water occurs in the hot months when a temperature rise is more probable.

A complete trial load analysis was required for case 2 in order to provide complete date for comparison of conditions with and without earthquake. This gave a clear picture of the effect of earthquake on Kamishiiba Arch Dam.

Allowable Stresses

It was proposed that the following 91 day concrete strengths be used as a basis for establishing the allowable design stresses:

W/C	91 day compressive strength, Kg/cm^2
.55	265
.50	320
.45	365

It was also proposed that the minimum value of 265 kg/cm^2 could be increased by 10% by the use of an enriched concrete mix where necessary. Therefore, a minimum available compressive strength of 291.5 kg/cm^2 could be assumed. For a Safety Factor of 5 the allowable stresses for static loads and temperature would be:

Final compressive principal stresses
normal mix 53.0 kg/cm^2
enriched mix 58.3 kg/cm^2

Final tensile principal stresses should not exceed 15% of the allowable compressive stresses, or for
normal mix 8.0 kg/cm^2
enriched mix 8.7 kg/cm^2

Concrete production for Kamishiiba Arch Dam was to be under constant quality control, thus, easily meeting this minimum 91 day strength. However, in accordance with Japanese code requirements the allowable stresses were reduced by 10% to allow for possible variations. The final maximum principal stresses for a combination of static load and temperature therefore did not exceed the following values:

	normal mix	enriched mix
Max. compression	47.7 kg/cm^2	52.5 kg/cm^2
Max. tension	7.2 kg/cm^2	7.9 kg/cm^2

and these values were increased by 15% for maximum allowable principal stresses under earthquake conditions.

The final layout of the dam was the result of many trials. Each lamination was considered as a unit, then was inspected in relationship to the entire dam. Fowler's curves were used as a guide to the general adequacy of each arch ring in the preliminary studies. These early stress explorations were regarded as preliminary design. After a satisfactory layout was selected, a more exact determination of the probable load and the induced stresses at the various elevations was made by the procedure outlined in pamphlet R/C 21 of the Portland Cement Association. This determination was basically a design procedure and served as the preliminary step for radial adjustments in the trial load analysis. The calculations required in this procedure were tabulated for ten evenly spaced elevations. In the design, and subsequently in the analysis, only the portion of the dam between the base and elevation 472 was considered effective as an arch.

Spillway

The spillway consists of four bays controlled by Tainter gates 9.0 meters wide and 8.3 meters high, in pairs near each end of the dam. General arrangement is shown in Plate No. 1, and details can be seen in Plates No. 2, 4 and 5. Spillway is capable of discharging 2 160 cubic meters per second when reservoir rises to Elevation 481.75 (1.75 meters above the maximum normal operating level).

In addition to the arrangement adopted, two alternative plans were studied in detail; namely, a free overflow spillway located in the center of the arch, and a tunnel spillway through the left abutment. For economic and other reasons neither of these plans was as attractive as the two chute spillways adopted.

Extensive model studies were made at the Electric Power Central Research Laboratory, Tokyo. Careful analysis was made of the approach conditions to the spillways which resulted in the layout shown in Plate No. 5. The orientation of the chutes was model tested very carefully for the purpose of having the two chute jets impinge upon each other, thereby obtaining effective energy dissipation of the overflow. The details of the chutes as shown on Plates No. 2 and No. 4 were the result of these model studies.

Since the completion of the project, the spillways have experienced large rates of overflow. Operation has proven very satisfactory, confirming the behavior predicted by the model tests. Two photographs showing the spillway in operation are shown in Figure 2. Discharge is 680 cubic meters per second in each chute.

ACKNOWLEDGMENT

To the Japanese engineers of the Kyushu Electric Power Company who worked so hard and diligently in carrying out the laborious computations required by the Trial-Load Method, the authors express their appreciation. Also, acknowledged is the invaluable help of two American engineers, A. L. Parme and M. F. Fornerod, who joined the authors in Japan to help with the training and work with the Japanese engineers.

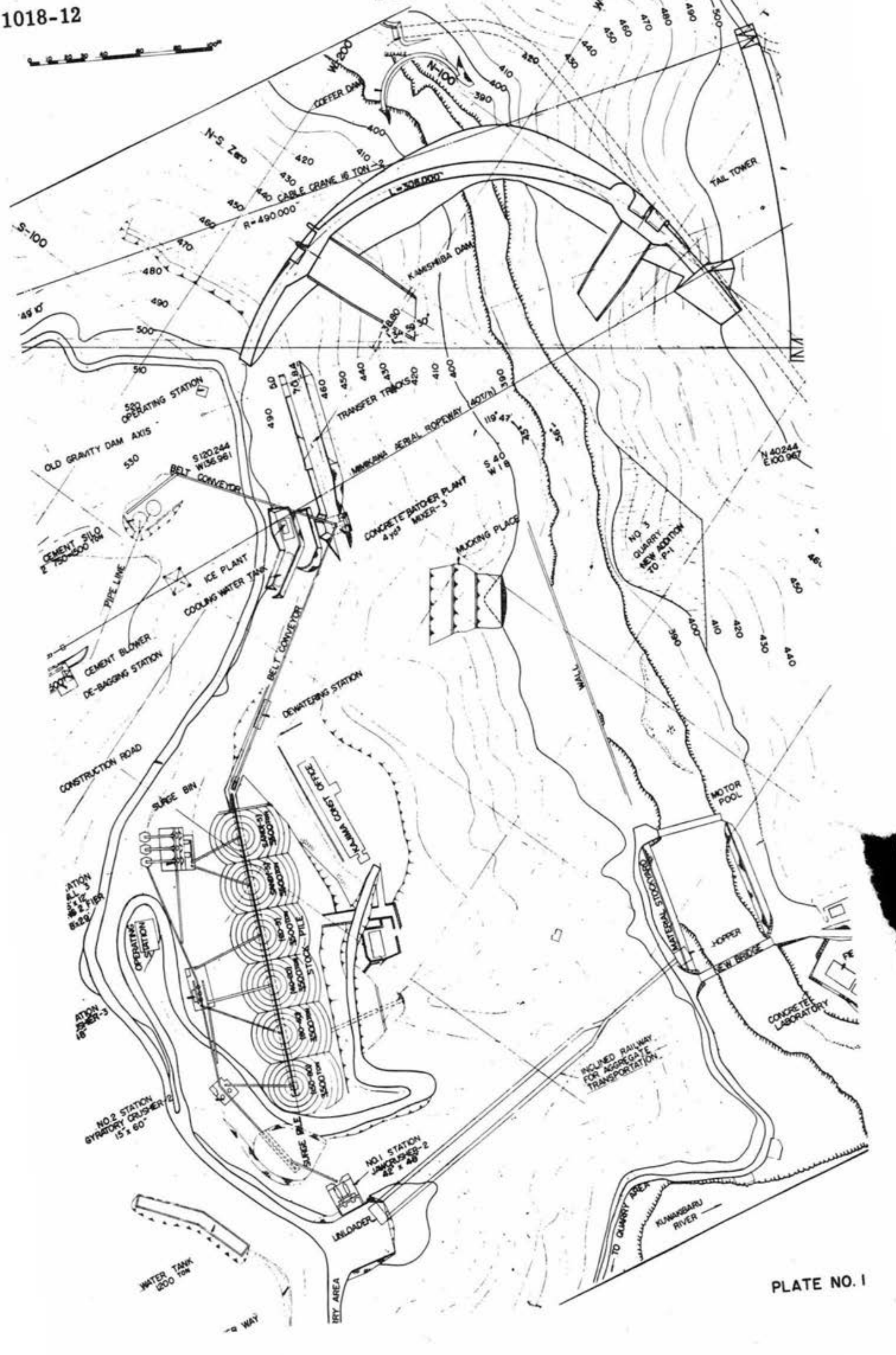
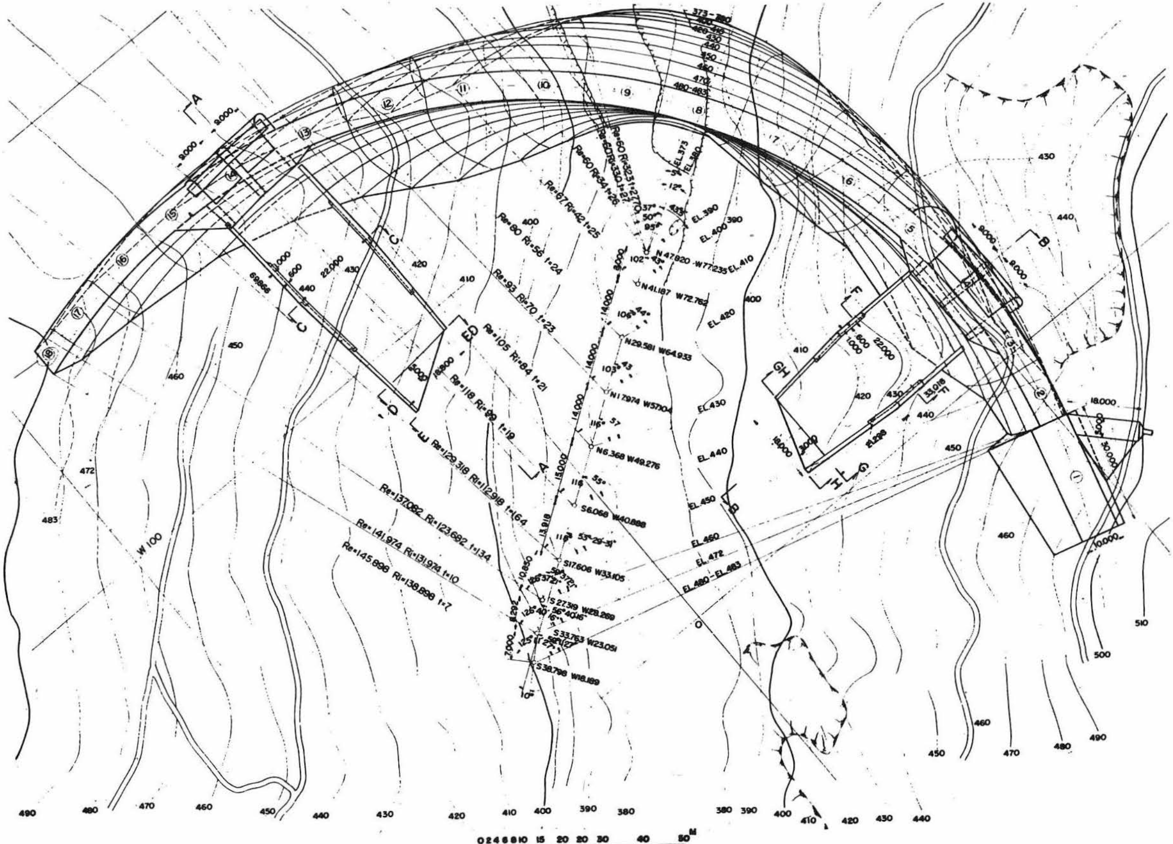

COFFER DAM
N-S Zero
CABLE CRANE 16 TON-2
R=490.000
KAMISHIBA DAM
TAIL TOWER
OPERATING STATION
OLD GRAVITY DAM AXIS
TRANSFER TRACKS
MIMIKAWA AERIAL ROPEWAY (40T/h)
BELT CONVEYOR
CEMENT SILO
CONCRETE BATCHER PLANT
MIXER-3
MUCKING PLACE
ICE PLANT
COOLING WATER TANK
PIPE LINE
CEMENT BLOWER
DE-BAGGING STATION
NO. 3 QUARRY
NEW ADDITION TO P-1
DEWATERING STATION
CONSTRUCTION ROAD
SURGE BIN
KAJIMA CONST OFFICE
STOCK PILE
WALL
MOTOR POOL
MATERIAL STOCKYARD
HOPPER
NEW BRIDGE
CONCRETE LABORATORY
INCLINED RAILWAY FOR AGGREGATE TRANSPORTATION
OPERATING STATION
NO.2 STATION GYRATORY CRUSHER-12 15" x 60"
SURGE PILE
NO.1 STATION JAW CRUSHER-2 42" x 48"
UNLOADER
TO QUARRY AREA
KUWAKIBARU RIVER
WATER TANK 1200 TON

PLATE NO. 1



FOR SECTIONS SEE
PLATE NO. 4

PLATE NO. 2

1018-14

PO 3

June, 1956

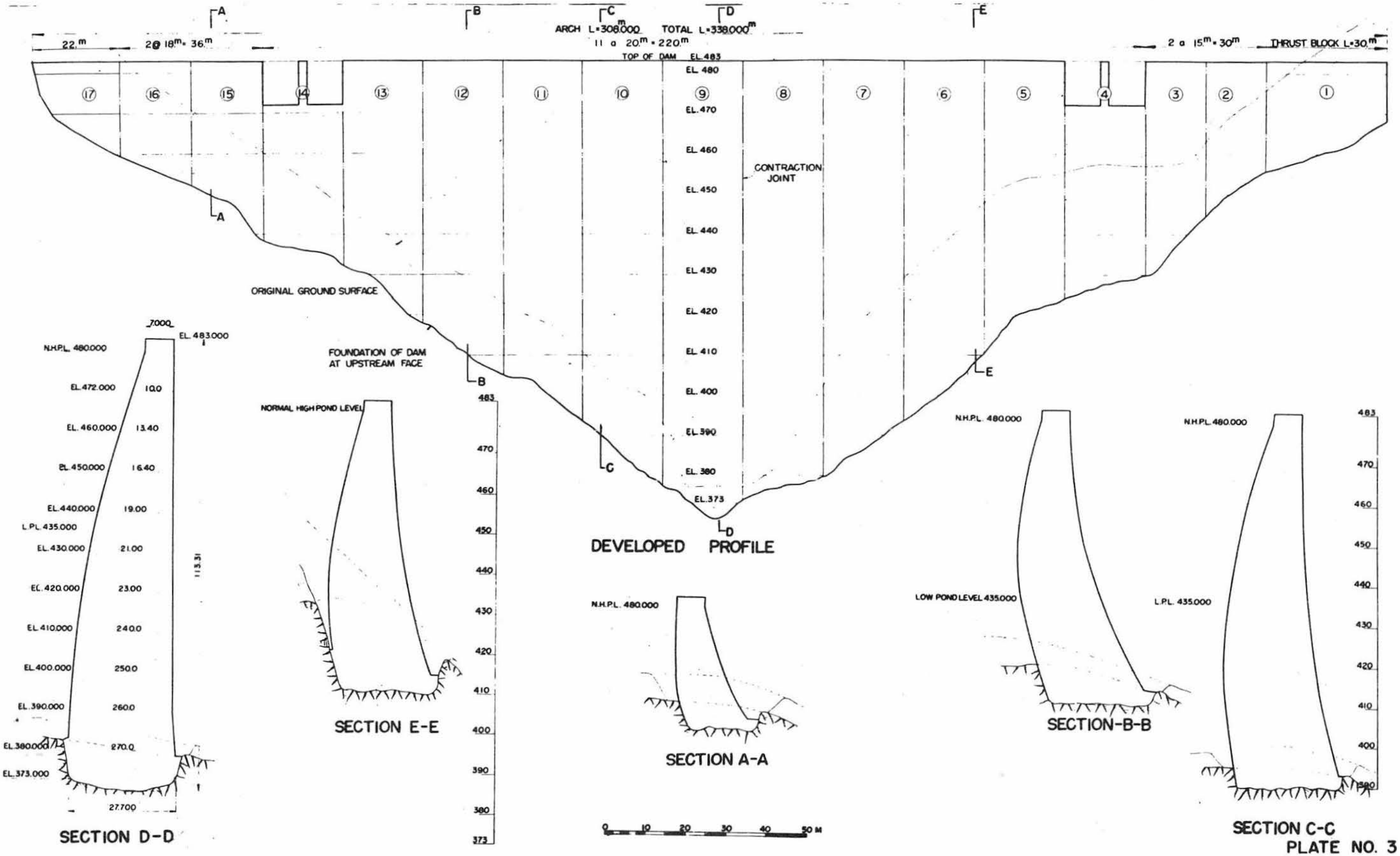
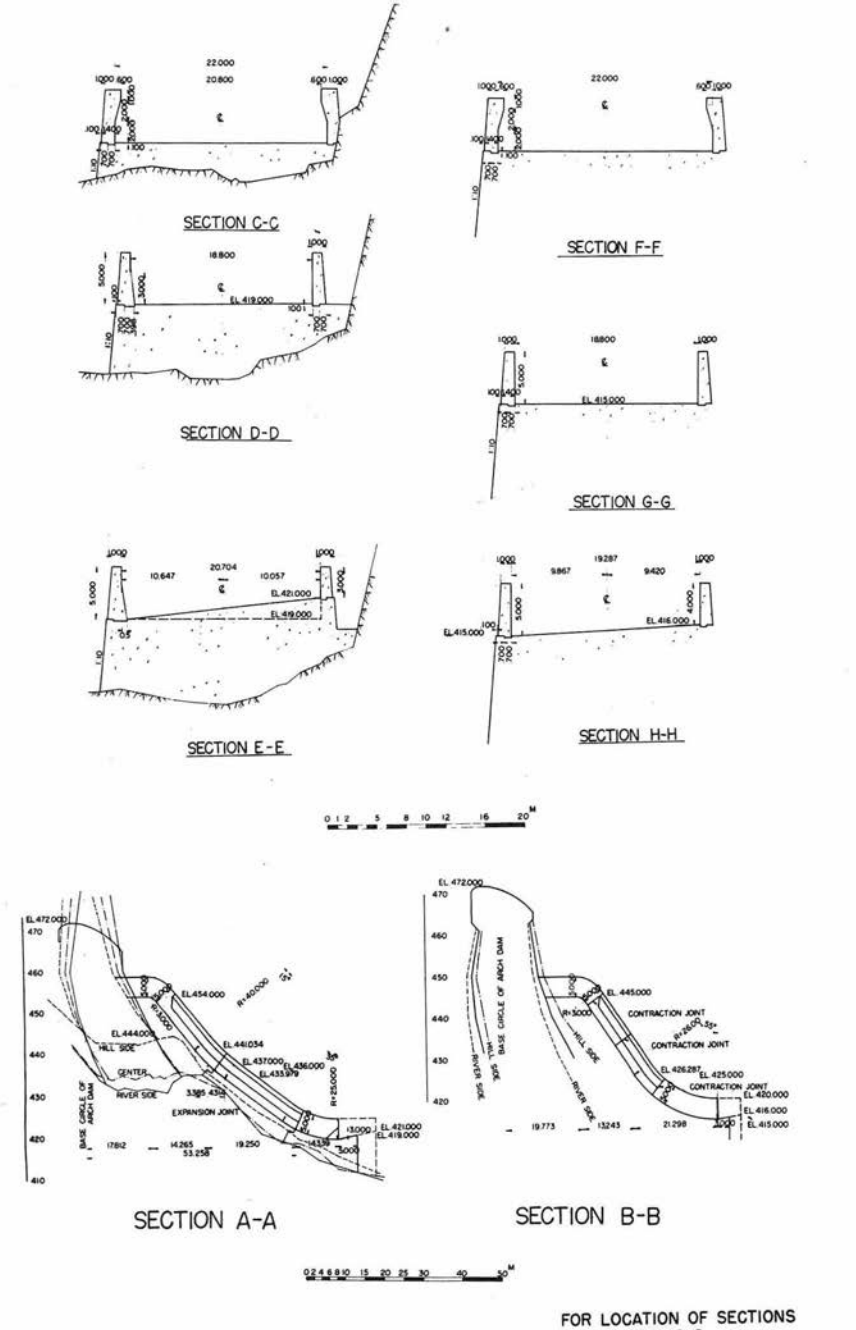
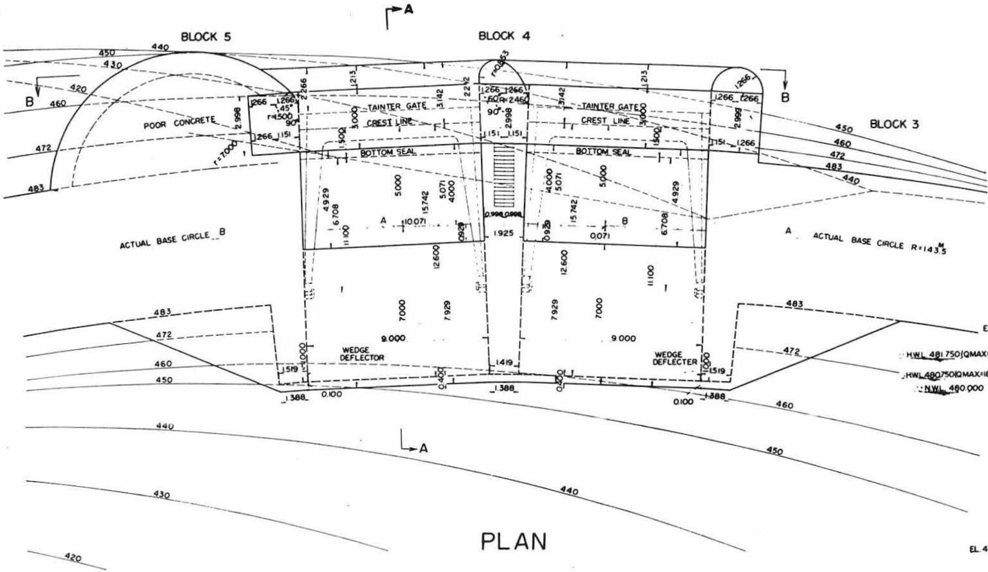


PLATE NO. 3

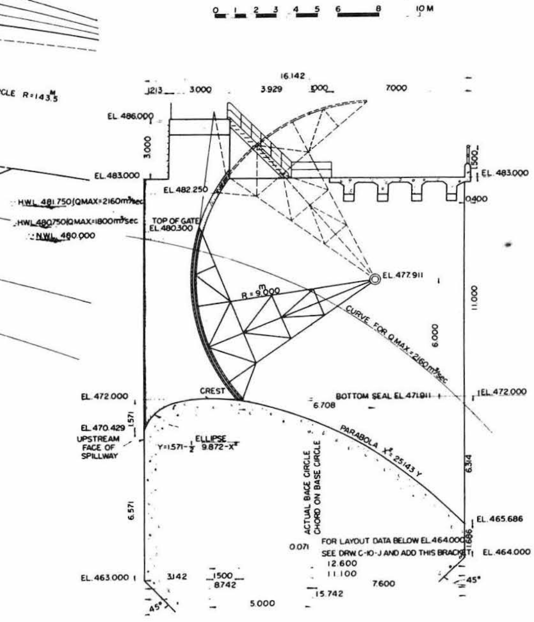


FOR LOCATION OF SECTIONS
SEE PLATE NO. 2

PLATE NO. 4

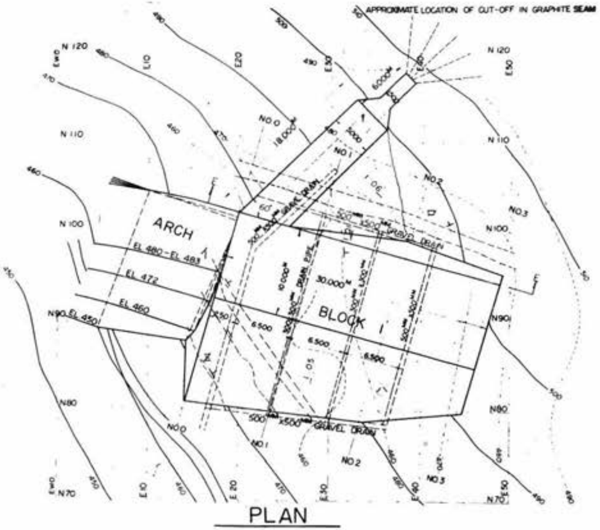


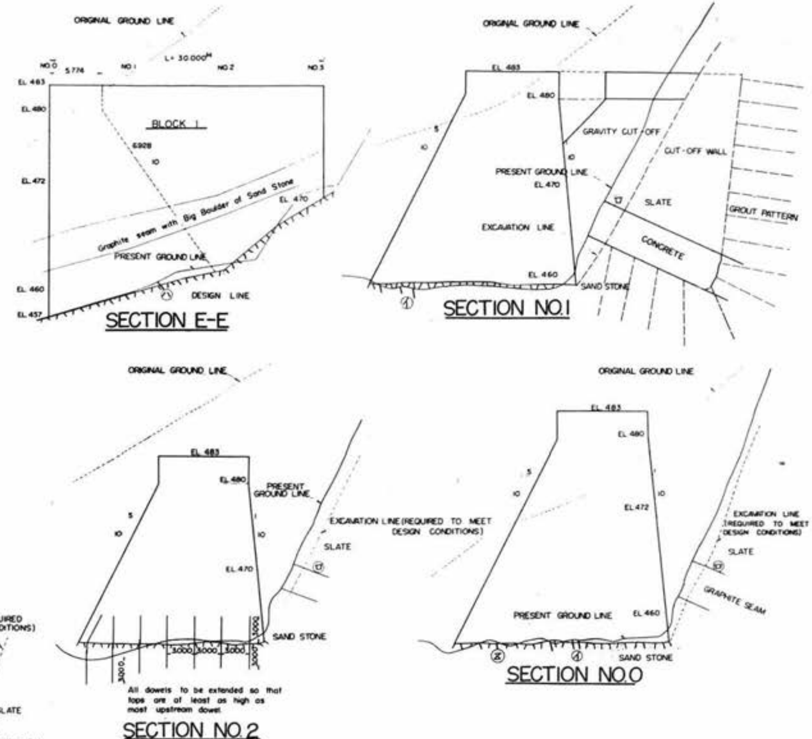
PLAN

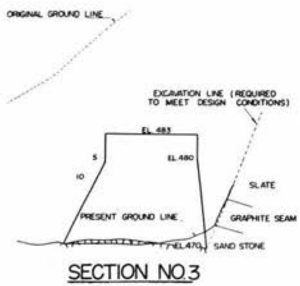


SECTION A-A

PLATE NO. 5







NOTES :

1. 30^{mm} ϕ Dowel bars will be embedded over the whole area of the foundation base of the thrust block, and the area of the foundation base of the cut-off which Lies south of the graphite seam.

2. Reinforcing bars will be placed along the construction joint between the cut-off and the thrust block with spacing approximately 500^{mm}± on center.

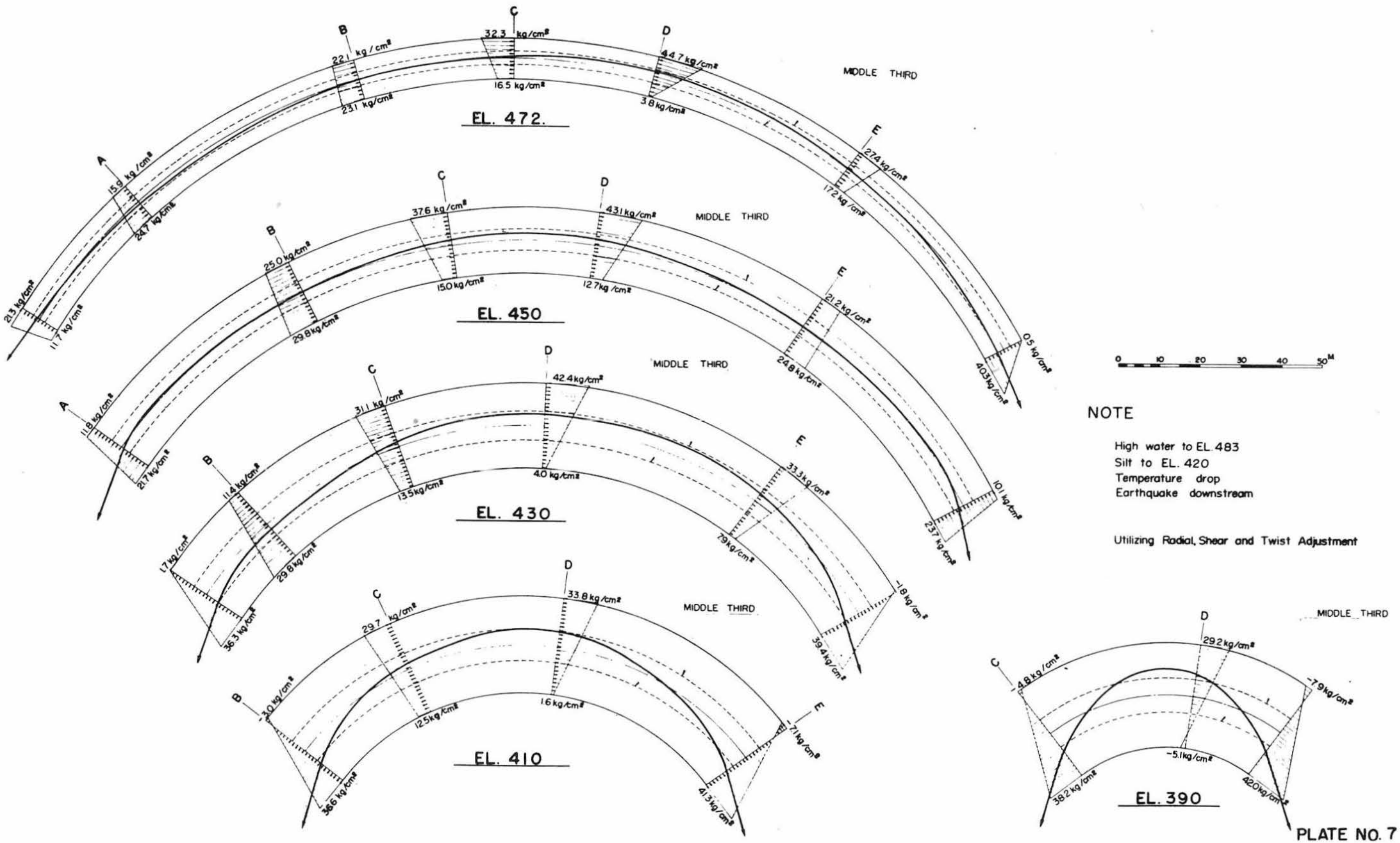
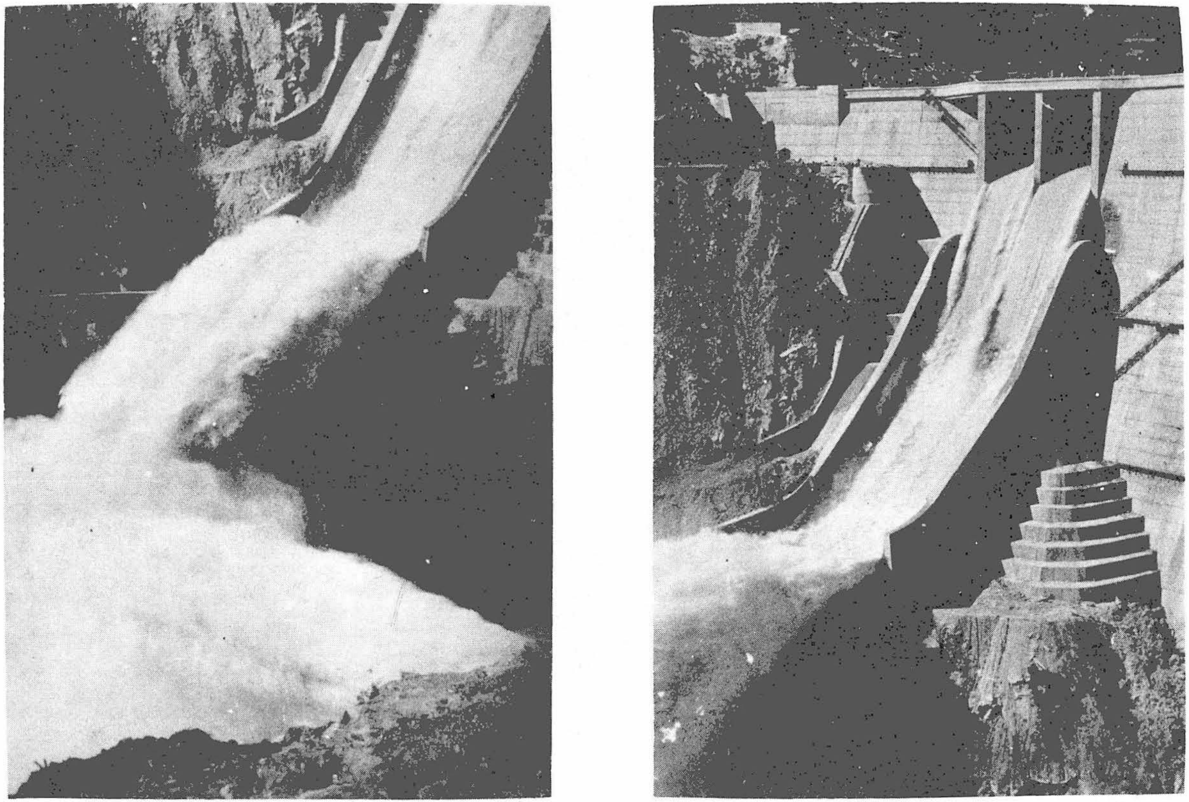

EL. 472.
EL. 450
EL. 430
EL. 410
EL. 390
MIDDLE THIRD
A
B
C
D
E
15.9 kg/cm²
24.7 kg/cm²
21.3 kg/cm²
11.7 kg/cm²
22.1 kg/cm²
23.1 kg/cm²
32.3 kg/cm²
16.5 kg/cm²
44.7 kg/cm²
3.8 kg/cm²
27.4 kg/cm²
17.2 kg/cm²
0.5 kg/cm²
40.3 kg/cm²
11.8 kg/cm²
21.7 kg/cm²
25.0 kg/cm²
29.8 kg/cm²
37.6 kg/cm²
15.0 kg/cm²
43.1 kg/cm²
12.7 kg/cm²
21.2 kg/cm²
24.8 kg/cm²
10.1 kg/cm²
23.7 kg/cm²
1.7 kg/cm²
36.3 kg/cm²
11.4 kg/cm²
29.8 kg/cm²
31.1 kg/cm²
13.5 kg/cm²
42.4 kg/cm²
4.0 kg/cm²
33.3 kg/cm²
7.9 kg/cm²
-1.8 kg/cm²
39.4 kg/cm²
-3.0 kg/cm²
36.6 kg/cm²
29.7 kg/cm²
12.5 kg/cm²
33.8 kg/cm²
1.6 kg/cm²
-7.1 kg/cm²
41.3 kg/cm²
4.8 kg/cm²
38.2 kg/cm²
29.2 kg/cm²
-5.1 kg/cm²
-7.9 kg/cm²
42.0 kg/cm²
0 10 20 30 40 50 M
NOTE
High water to EL. 483
Silt to EL. 420
Temperature drop
Earthquake downstream
Utilizing Radial, Shear and Twist Adjustment

PLATE NO. 7

KAMISHIIBA DAM FIGURE 1



680 Cubic Meters per Second per Chute

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: DESIGN AND CONSTRUCTION OF ROSS DAM

C. E. Shevling[1] and L. R. Scrivner[2]
(Proc. Paper 1045)

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams (April, 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time, it is not known exactly how many papers will be printed from the Symposium. So far, twelve papers have been approved: "Arch Dams: Their Philosophy," by Andre Coyne (Proc. Paper 959); "Arch Dams: Trial Load Studies for Hungry Horse Dam," by R. E. Glover and Merlin D. Copen (Proc. Paper 960); "Arch Dams: Portuguese Experience with Overflow Arch Dams," by A. C. Xerez (Proc. Paper 990); "Arch Dams: Theory, Methods, and Details of Joint Grouting," by A. Warren Simonds (Proc. Paper 991); "Arch Dams: Santa Giustina Single-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 992); "Arch Dams: Measurements and Studies on Santa Giustina Dam," by Claudio Marcello (Proc. Paper 993); "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 994); "Arch Dams: Isolato Double-Curvature Arch Dams," by Claudio Marcello (Proc. Paper 995); "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs," by Claudio Marcello (Proc. Paper 996); "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira (Proc. Paper 997); "Arch Dams: Development in Italy," by Carlo Semenza (Proc. Paper 1017); "Arch Dams: Design of the Kamishiiba Arch Dam," by C. C. Bonin and H. W. Stuber (Proc. Paper 1018); and "Arch Dams: Design and Construction of Ross Dam," by C. E. Shevling and L. R. Serivner (Proc. Paper 1045).

As other papers are approved, they will be published in the Proceedings.

Note: Discussion open until January 1, 1957. Paper 1045 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 4, August, 1956.

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The interested reader should watch for these papers in following issues of the Journal of the Power Division.

INTRODUCTION

Ross Dam is the most important unit in the hydroelectric development of the Skagit River by the city of Seattle, Washington. It provides the main storage for the entire system which includes the Ross, Diablo, and Gorge Powerplants. These plants have a total generating capacity of 580,000 kw.

Diablo Lake, having a storage capacity of 90,000 acre-feet, and Gorge High Dam Reservoir, which has a storage of 6,000 acre-feet, serve to supply hydrostatic head for their respective plants and to accommodate the daily fluctuations caused by peaking at the Ross Plant.

Description

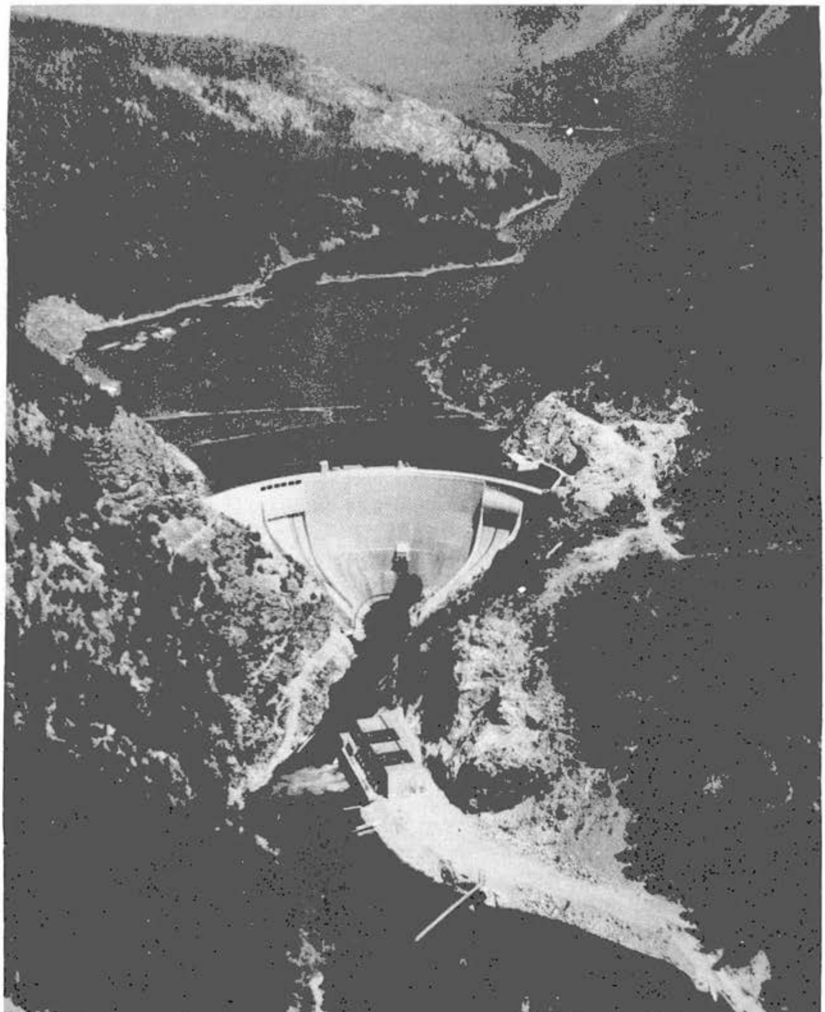
Ross Dam is located on the Skagit River approximately 100 miles northeast of Seattle, Washington. It is a variable radius arched structure which ranks as one of the world's highest arch dams. It rises 540 feet above foundation rock to an elevation 1615 feet above sea level and contains 905,000 cubic yards of concrete. At the crest, the dam has a length of 1,300 feet and an extrados radius of 600 feet. It varies in thickness, at the crown, from 208.5 feet at the base to 33 feet at the crest. It is constructed in 26 sections, separated by keyed contraction joints. Each section is 50 feet wide along the axis of the dam. During construction, the sections were raised systematically in a manner that provided for alternate low, intermediate, and high sections.

Ross Dam is unusual in some of its structural features. These include the provision for additional construction beyond the present stage to raise the dam to ultimate elevation 1733, a total height of 658 feet. This contemplated additional construction will require thickening the entire dam, from the base up, by adding to the downstream side where patterns of square and vertical keys were cast in the concrete to form a keyed bond between the old and the new concrete sections. (See photograph on next page.)

Other unusual features are the two hooded spillways, incorporated in the ends of the dam and designed to discharge 127,000 cubic feet per second. The hoods deflect the flow through the spillways, downward, to conform to the steep downstream face of the arch dam. Their design was determined from experimental scale models built in the Bureau of Reclamation Laboratories in Denver. The flows of water routed down the two spillways collide in midstream, partially dissipating their energy against each other. The spillway hoods are built of concrete, heavily reinforced with steel.

Purposes Served

At its present height, Ross Dam creates a storage reservoir of 1,405,000 acre-feet. Of this storage 1,023,000 acre-feet are usable for power generation. Thus, it serves to impound water from which the turbines of three powerplants, namely, Ross, Diablo and Gorge, extract power between



elevations 1600 (maximum) and 495. With Gorge High Dam scheduled to be completed early in 1959, every foot of this head between the forebay of the Ross Plant and the tail water of the Gorge Plant will be harnessed for power generation.

As required by the Corps of Engineers, Department of the Army, through the Federal Power Commission, the water level in Ross Lake is regulated in accordance with their prescribed rules for flood control. This provides valuable flood protection for the farm lands and communities of the lower Skagit River Valley.

History of Stepwise Construction

As a measure of economy and to match the anticipated increase in demands for power, a plan of stepwise construction was initiated for Ross Dam.

The first step was originally designed to be carried to elevation 1515 with the spillway crest at elevation 1500. This would provide a storage of 500,000 acre-feet for the Diablo and Gorge Powerplants. No power installation at Ross was planned for this step.

A contract was awarded in August 1937 for the construction of a portion of the First Step of the dam to elevation 1300. However, with money saved when excavation was not required to the depths estimated by the engineers, it was possible to continue placing concrete until elevation 1365 was reached. In addition, a temporary 15-foot timber crib was constructed on top of the concrete dam, raising its height to elevation 1380. This created a storage of 100,000 acre-feet for operation of the Diablo and Gorge Plants. This first-step construction required 330,000 cubic yards of concrete.

The Second Step, as it was then planned, required starting again by thickening the dam at the base and constructing it on a design which would permit carrying it to an ultimate height at elevation 1725. This sequence was necessary because at that time the plan was to construct the powerhouse on the face of the dam.

Studies made subsequent to the completion of the first contract indicated that it might be possible to carry the First Step to a greater height than the originally planned elevation 1515. This, together with constructing the powerhouse on the left bank downstream from the dam, instead of on the face of the dam, would make considerable power available without going to the expense of thickening the base for the high dam.

While this change in plan was being studied, the War Production Board started urging the city to proceed with increasing the height of the dam in order to aid the war effort. This the city agreed to do and a contract was awarded in February 1943 to raise the dam from elevation 1365 to elevation 1550. This was designated as the Second Step of Ross Dam, and would have required 456,900 cubic yards of concrete.

During construction of the Second Step, the consulting engineers, upon conclusion of their studies and calculations, determined that the dam could be constructed to elevation 1615 on the original First Step base. The Federal Power Commission approved construction of the dam to this elevation, its present height. An important factor which influenced this approval was the high strength of the concrete which permitted a higher allowable unit stress in the structure.

With the changes in the original concept of the construction program, the steps of construction for the dam became and remain, identified as follows:

The First Step was the first contract which brought the concrete to elevation 1365.

The Second Step raised the dam to elevation 1550.

The Third Step is the dam to its present height at elevation 1615.

The Ultimate Height is the plan for the future which will thicken the present structure and raise it to elevation 1733.

The First Step contract was awarded to the General-Shea-Columbia Construction Company in August 1937 and completed in 1940.

The Second Step contract was awarded to General-Shea-Morrison Construction Company in February 1943. Before the completion of this contract, it became evident that to meet the growing demand for electrical energy, the Third Step of the dam should be constructed without delay. In order to save the time and expense of an extra cleanup and of moving out one construction plant and moving in another, the construction of the Third Step was added to the Second Step contract by change order. Thus, the Second Step was never completed as planned. The construction carried through in one continuous operation from the beginning of the Second Step to the completion of the Third Step.

The Third Step contract was completed in June 1949.

Stress Studies

The trial-load studies which were made for the design of Ross Dam were carried out according to the Bureau of Reclamation procedure (1) for analyzing arch dams by the trial-load method.

An analysis was made of the proposed Third Step construction for a water-surface elevation of 1635. The contraction joints were assumed to be grouted with the reservoir empty and the concrete at mean (2) annual temperature, except the temperatures were assumed to be 5° and 2° F above mean annual at elevations 1200 and 1250, respectively. This analysis included effects of tangential shear and twist. The maximum computed principal stress was 1,114 psi at elevation 1200.

An analysis was also made of the proposed Second Step construction for a maximum water-surface elevation of 1548.8. Cooling studies (see section on Cooling Studies) were made in connection with this analysis and it was assumed that the contraction joints would be grouted with the water surface at elevation 1295. This study indicated a maximum principal stress of 752 psi at elevation 1200. This analysis also included effects of tangential shear and twist.

When it was decided to place the concrete for the Second and Third Steps as a continuous operation, a limit was set on the height of dam so the following allowable stresses would not be exceeded:

For normal water surface and neglecting earthquake:

Principal stress, 1,000 psi

Shear stress at concrete-to-rock contact plane, 350 psi

For normal water surface plus earthquake effects or for maximum flood conditions without earthquake:

Principal stress, 1,250 psi

Shear stress at concrete-to-rock contact plane, 425 psi

The stresses all to be determined by a complete[3] trial-load analysis.

A plan view of the dam is shown on Figure 1; representative arch elements are shown as contours on the figure. A section at the line of centers (crown cantilever) is also shown.

The trial-load stress study of the dam, as constructed, was based on the following constants, assumptions, and loading conditions:

1. Top of dam, elevation 1615
2. Base at maximum section, elevation 1075
3. Normal water surface, elevation 1600
4. Earthquake effects neglected
5. Sustained modulus of elasticity of concrete and abutment rock, 3,000,000 psi
6. Shear modulus of concrete, 1,250,000 psi, reduced to 1,000,000 psi in calculating the detrusions due to radial shears to allow for the nonlinear distribution of shearing stresses between the upstream and downstream faces of the dam
7. Poisson's ratio for concrete and rock, 0.20
8. Unit weight of concrete, 156 pounds per cubic foot
9. Coefficient of thermal expansion, 0.000,004,6 per degree Fahrenheit
10. The horizontal arch elements assumed to be symmetrical with radial abutments
11. Radial contraction joints in the original structure and in the new concrete assumed to be thoroughly grouted and satisfactory bond established between old and new concrete so the dam will act as a monolithic structure
12. Cantilever elements have radial sides 1 foot apart at the axis of the dam
13. Normal arch and cantilever stresses assumed to have a linear variation from upstream to downstream face
14. The concrete in the dam and the foundation rock assumed to have adequate strength to carry the imposed loads without exceeding their elastic limits

The effects on stress conditions in the dam due to the grouting and construction programs were included in the trial-load analysis, each stage of construction corresponding to a part of the analysis as follows:

Part 1

Contraction joints in the First Step construction grouted with the water surface at elevation 1285. All loads assumed to be carried by cantilever action. Stresses computed by gravity analysis at radial-sided cantilevers (see Figure 2 for results).

Part 2

Crest of dam raised to elevation 1500. Water surface raised to elevation 1354. Additional load carried by gravity action above elevation 1365, but divided between arches and cantilevers below elevation 1365 by means of a radial adjustment of deflections (see Figure 3).

Lift between elevations 1365 and 1400 grouted with reservoir water surface at elevation 1354.

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3. A complete trial-load analysis is used to mean that the effects of tangential shear and twist are included.

Part 3

Crest of dam raised to elevation 1615. Water surface reduced to elevation 1295. Additional load carried by gravity action above elevation 1400, but divided between arches and cantilevers below elevation 1400 by means of a radial adjustment of deflections (see Figure 4).

Portion of dam between elevations 1400 and 1615 grouted with the water surface at elevation 1295.

Part 4

Water surface raised to elevation 1600. All loads above elevation 1567 assumed to be carried by gravity action (due to through spillways). Load below elevation 1567 divided between arch and cantilever elements, by means of a complete adjustment, including the effects of temperature change from average concrete temperatures at the time the contraction joints were grouted to the minimum cyclic temperature (see Columns 2 and 3 of Table 1).

Stresses in the completed structure were obtained by super-imposing stresses calculated from the analyses made for the conditions stated in Parts 1, 2, 3, and 4 (see Figures 5 and 6). These stresses include effects of tangential shear, twist, and Poisson's ratio. In the complete analysis, deflections due to Poisson's ratio, were computed and included in the study at the end of the second radial adjustment.

Discussion of Stresses

The stresses tabulated on the figures are given in pounds per square inch (psi) and the following symbols are used:

E = stress at extrados of arch
I = stress at intrados of arch
U = stress at upstream face of cantilever
D = stress at downstream face of cantilever
\+ indicates compression
Crown of arch indicated by small square

Sections shown to the right of the stress drawings indicate the extent of construction and grouting as well as the reservoir water-surface elevations. The grouted portion of the dam is indicated by cross-hatching. Concrete which was included in a previous adjustment is indicated by hatching; new concrete has no hatch mark.

The load distribution is also shown at the crown cantilever with the cantilever load indicated by hatching. Load plotted to the right of the zero line indicates negative cantilever load, at which point the arch carries the external load plus an amount equal to the cantilever load.

Vertical cantilever stresses, due to loading Part 1, are shown on a profile of the dam (see Figure 2). The maximum computed stress of 382 psi occurs at the downstream face of the base of the crown cantilever.

Arch stresses normal to radial planes and vertical cantilever stresses for Part 2 loading are shown on Figure 3. The maximum computed stress amounts to 457 psi at the base of the dam. This would amount to 604 psi parallel to the downstream face.

Similarly, stresses through Part 3 loading are shown on Figure 4. The maximum computed stress is almost identical to the corresponding value for Part 2, as given in preceding paragraph. A tensile stress of 85 psi is indicated at the crown intrados of the arch element at elevation 1250. If a crack were developed here, instead of the tensile stress, it would have very little effect on the final distribution of load and stress in the structure. The stresses discussed above are based on adjustment of radial deflections at points of intersection of arch and cantilever elements.

Arch and cantilever stresses, for loading through Part 4, are given on the profile of Figure 5. These stresses are based on a complete analysis and included effects of Poisson's ratio and temperature changes listed in Column (4) of Table 1. Principal stresses are shown graphically on Figure 6, and numerically in Table 2. The direct and principal stresses are assumed to act parallel to the faces of the dam.

The maximum computed arch stress is 615 psi and occurs at the crown extrados at elevation 1540. Stresses were computed at elevation 1540, but were omitted from the figures to make the figures clearer. The maximum arch stress at the abutments is 433 psi at the arch intrados at elevation 1250. There were no tensile stresses in the arch elements for this loading condition.

The maximum cantilever stress, 772 psi, occurs at the base of Cantilever E at downstream face. The stress at the base of the crown cantilever is 709 psi. There were no tensile cantilever stresses for this loading condition. However, with reservoir partially full, at a time when concrete temperatures are at or near maximum values, as given in Column (3) of Table 1, tensile stresses of moderate amount would occur at the downstream face of some cantilevers. This is not considered critical, particularly since the tensile area would be under compression during maximum loading of the structure.

The maximum computed principal stress is 800 psi, 80 percent of the allowable design value of 1,000 psi. All principal stresses were compressive except for a value of 20 psi tension at the downstream face at elevation 1500.

The maximum horizontal shearing stress on the concrete-to-rock abutment plane is 340 psi (see the final column of Table 2). This is 97 percent of the design value of 350 psi.

The stresses discussed above include the effects of average concrete temperature, but do not reflect the effects of temperature gradients due to seasonal or daily variations. It is apparent that daily and seasonal changes in temperature will have considerable effect on the stress condition near the surface of the dam. The surface stress will fluctuate over quite a wide range, but will be relieved somewhat by plastic flow during periods when the surface of the structure is hot and by surface checking during extremely cold weather. However, the condition of a drop in mean concrete temperature combined with maximum loading is as severe as any to which the dam may be subjected.

Temperature Studies

Temperature studies were based on temperature records at Diablo Dam, and at Concrete (formerly Baker), Washington. From these data, it was estimated that the mean annual air temperature would be close to 48°-50° F. Reservoir water and cooling water temperatures were estimated from observed water temperatures in the existing Ross Lake, and in Diablo Lake and Happy Creek. Studies were then made to find the range of mean concrete temperatures at the several arch elevations. The first study was solely for Second-step construction, and was later extended to cover construction of the combined Second and Third steps. The results of the latter temperature study were as shown in Table 1.

The trial-load analysis of the Second Step dam was made using the above data and taking into account the desired closing temperature (40° F) and the temperature rise to the minimum temperature condition. A complete trial-load analysis was later made for the dam, as constructed, using the actual closure temperatures experienced at the several arch elevations.

Artificial cooling of the concrete was required primarily for cooling the concrete to the desired temperature so that contraction joint grouting could be accomplished either during or immediately after the construction period. Additional benefits included the reduction of the maximum temperatures and the crack prevention measures obtained by reducing the temperature gradients near the faces of the dam and near the contact of the existing concrete with the new concrete. Temperature studies indicated that a cooling transition zone would be beneficial in the first lifts of the new concrete. The horizontal spacing of the cooling tubing was, therefore, 2-1/2 feet on top of the existing concrete and on top of the first 5-foot lift of concrete above the existing concrete, 3-1/2 feet on top of the second lift, 4-1/2 feet on top of the third lift, and 5-1/2 feet on the fourth lift. This last spacing was then continued to the top of the dam.

Studies were also made as to the expected temperatures in the concrete at the time of secondary cooling and the time required to lower the temperature of the concrete to the required temperature in time for contraction joint grouting.

Method of Joining New and Old Construction

The old concrete was thoroughly cleaned and chipped to obtain good bond between the new and old concrete. Where necessary wet sandblasting was used to remove all laitance, efflorescence, grease, mud, debris, pools of water, and loose or defective concrete. The surface of the old concrete was thoroughly soaked with water for several days prior to placing new concrete. Just prior to placing new concrete, 1/2-inch of mortar was placed over the surface and thoroughly broomed into the surface.

The first new concrete placed was provided with cooling coils at reduced spacing to limit the temperature gradient at the contact plane between old and new concrete. (See paragraph on Temperature Studies.)

Water Stop Details

Copper water stops were installed across each vertical contraction joint at the upstream face of the dam. These consisted of 16-gage, 48-ounce copper strips 15 inches wide. The center 3 inches of the strip was bent double and projected 1-1/4 inches normal to the stop in the contraction joint to absorb the expansion at the contraction joint. Anchorage at the outer two edges of the strip was provided when embedded in the concrete by cutting the copper every 12 inches, 1 inch deep, and bending alternately to the right and left normal to the stop.

Horizontal water stops 10 inches wide of 16-gage, 48-ounce-per-square-foot copper strips were embedded in the old concrete when placed, and at top of each horizontal lift in new concrete. The water stops were located 9 inches from the upstream face of the dam.

The horizontal stops were brazed to vertical stops in each contraction joint to make a continuous water seal.

Abutment Preparation

The abutments were excavated to sound rock free from injurious seams or defects. Care was exercised to prevent shattering of the rock outside of the limits of the excavation. Blasting was very carefully controlled to preserve the sound rock and to prevent injury to the structure.

The foundation was prepared by removing all loose, weathered, disintegrated and shattered rock and shaping the abutment for the placing of concrete. The surfaces were left rough so that a good bond would be secured with the newly placed concrete. The rock surfaces were cleaned by brooms, hammers, picks, streams of water, air, and wet sandblasting. All water was removed from the surface of the foundation before concrete was placed thereon. The foundation was thoroughly grouted and drained.

Cooling and Grouting Operations

The concrete of the First Step was placed without provision for artificial cooling. It was grouted in the Spring of 1946, thereby taking advantage of natural cooling during the winter months. Mean concrete temperatures at time contraction joints were grouted are given in Column (1) of Table 1.

Concrete cooling operations, in Second and Third Step concrete, included an initial cooling period and a secondary cooling period. The initial cooling period was to reduce the maximum temperatures and to obtain a more beneficial temperature differential between the interior and the face of the dam. The initial cooling period varied from 6 to 12 days, depending on the time of year. Secondary cooling to lower the concrete temperatures to the desired grouting temperatures required approximately 45 to 50 days, depending on exposure conditions and the temperature of the available cooling water.

Secondary cooling was started about December 1, 1946, but was stopped about 2 weeks later due to extremely cold weather which froze the exposed header systems. Cooling was resumed about February 1, 1947, and temperatures were lowered to approximately 40° F. However, due to an early runo and the resulting high reservoir level, grouting was interrupted and only a part of the grouting planned for the Spring of 1947 was completed. Because of the difficulties experienced in the 1946-47 winter season, a refrigerating plant was obtained and installed in the Fall of 1947 to assure that the remainder of the secondary cooling would be completed in the Spring of 1948. The plant consisted of fourteen 50-ton commercial-type units which supplied 29° F brine to the cooling coils. Secondary cooling was started near the end of January 1948, and was completed to the top of the dam in March 1948.

Trial-load analyses indicated that it would be desirable to grout the contraction joints when the reservoir elevation was at or near elevation 1295. Because of the early and large river runoff in the Spring of 1947, however, the contraction joint grouting program was speeded up and grouting was started in the abutment joints when the reservoir was about elevation 1420 and dropping. The reservoir gradually dropped to elevation 1354 and then started rising. Abutment joints were grouted to elevation 1450 during this time but the central part of the dam, from Joints 10 to 18, inclusive, was only completed to elevation 1400. In the Spring of 1948, all contraction joints were grouted to elevation 1615 with the reservoir between elevations 1288 and 1301.

A summary of contraction joint grouting operations is given in Table 3. In the 1946 operations, 1,748 sacks of cement were used in grouting 204,770 square feet of joint area (keyways neglected). The average joint area covered for each sack of cement was 117 square feet. In the 1947 operations, 846 sacks of cement filled 95,393 square feet of joint area, or 113 square feet per sack of cement injected. In the 1948 operations, 2,139 sacks of cement filled 159,045 square feet of joint area, or 74 square feet per sack of cement injected.

ACKNOWLEDGMENT

The authors gratefully acknowledge the valuable assistance and contributions by members of the Bureau of Reclamation, and of the Seattle Department of Lighting.

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Table 1

MEAN CONCRETE TEMPERATURES

Elevation	Temperature in degrees Fahrenheit: At time of joint closure (1)	Minimum * (2)	Maximum * (3)	Change ** included (4)
1200	48.0	47.5	51.0	-0.5
1250	47.0	47.5	51.5	+0.5
1300	47.0	47.0	52.0	0
1350	44.0	47.0	52.5	+3.0
1400	41.0	46.5	54.0	+5.5
1450	39.1	46.5	55.5	+7.4
1500	35.9	45.5	57.0	+9.6
1567	34.9	43.0	60.0	+8.1

* Due to annual cyclic variation

** Minus sign indicates temperature drop. Plus sign a temperature rise.

Table 2

PRINCIPAL STRESSES AND MAXIMUM SHEAR* STRESS

Elevation	Upstream face			Downstream face			Max shear*
	P_1	P_2	a	P_1	P2	a	
1567	+ 51	+199	9° 50'	+293	+ 28	76° 43'	1[illegible]
1500	+ 75	+282	15° 10'	+309	- 20	50° 04'	4[illegible]
1450	+104	+139	61° 20'	+453	+ 39	64° 07'	164
1400	+ 89	+144	74° 05'	+446	+ 61	59° 21'	202
1350	+ 56	+130	70° 33'	+483	+ 75	48° 20'	228
1300	+ 66	+215	83° 39'	+455	+ 17	59° 29'	216
1250	+ 98	+243	32° 13'	+636	+216	45° 52'	267
1200	+ 27	+155	30° 44'	+800	+164	20° 52'	340
1075	+151	0	0° 00'	+709	0	0° 00'	244

* On concrete-to-rock contact planes

Table 3

GROUTING PROCEDURES--ROSS DAM

Grouting operation	Date grouted	Grouted to elevation	Joints grouted	Joint Area sq. ft	Cement placed (sacks)
1	2-4-46	1200	14 & 15	25,073	294
2	3-6-46	1250	11-15	30,539	184
3	3-11-46	1300	14-19	32,705	277
4	3-14-46	1365	15-19	27,240	208
5	3-18-46	1300	8-13	36,239	316
6	3-21-46	1365	9-14	31,806	227
7	3-27-46	1365	7, 8 & 20	21,168	242
	Subtotal 1946 operations			204,770	1,748
8	3-13-47	1400	19-21	12,197	145
9	3-19-47	1400	7-9	9,883	120
10	3-27-47	1400	21	4,181	55
11	4-16-47	1400	10-18	20,537	160
12	4-21-47	1450	19-22	21,195	146
13	4-23-47	1450	5-9	27,400	220
	Subtotal 1947 operations			95,393	846
14	2-20-48	1450	10-18	24,131	215
15	2-22-48	1500	18-23	14,984	87
16	2-29-48	1500	4-9	16,279	204
17	3- 2-48	1500	21	2,768	27
18	3- 4-48	1500	10-17	18,235	342

Table 3 (continued)

GROUTING PROCEDURES--ROSS DAM

Grouting operation	Date grouted	Grouted to elevation	Joints grouted	Joint Area sq. ft	Cement placed (sacks)
19	3- 6-48	1550	18-24	12,646	205
20	3- 8-48	1550	4-10	15,471	170
21	3- 9-48	1550	12, 14, 16-17	---	47
22	3-11-48	1550	11-17	14,464	240
23	3-12-48	1600	18-24	8,870	142
24	3-13-48	1600	2-10	11,522	158
25	3-15-48	1600	11-17	12,283	194
26	3-16-48	1614	1-25	7,392	108
		Subtotals 1948 operations		159,045	2,139
		Total all joint grouting		459,208	4,733

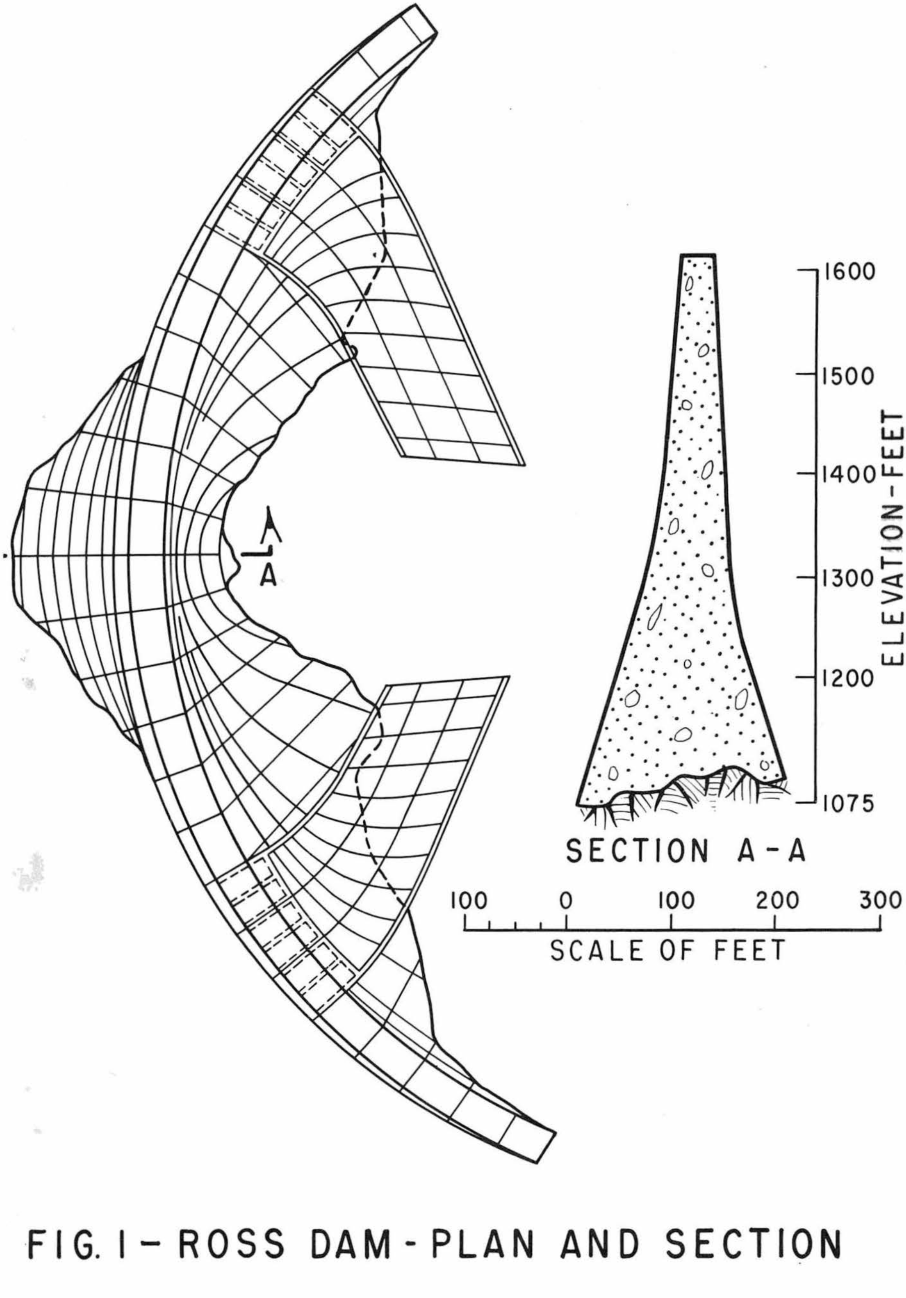


FIG. 1 – ROSS DAM - PLAN AND SECTION

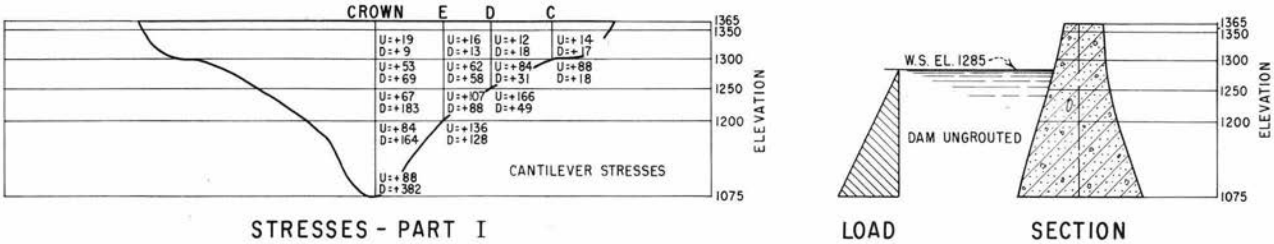


FIGURE 2

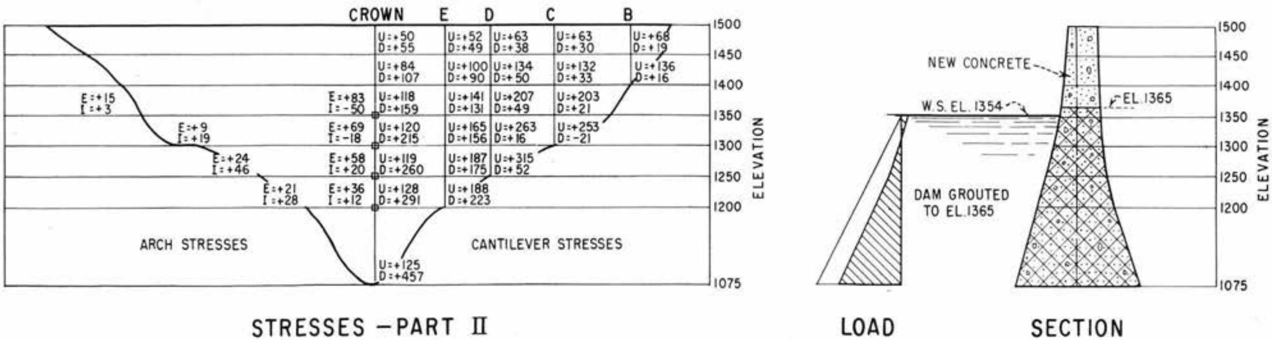
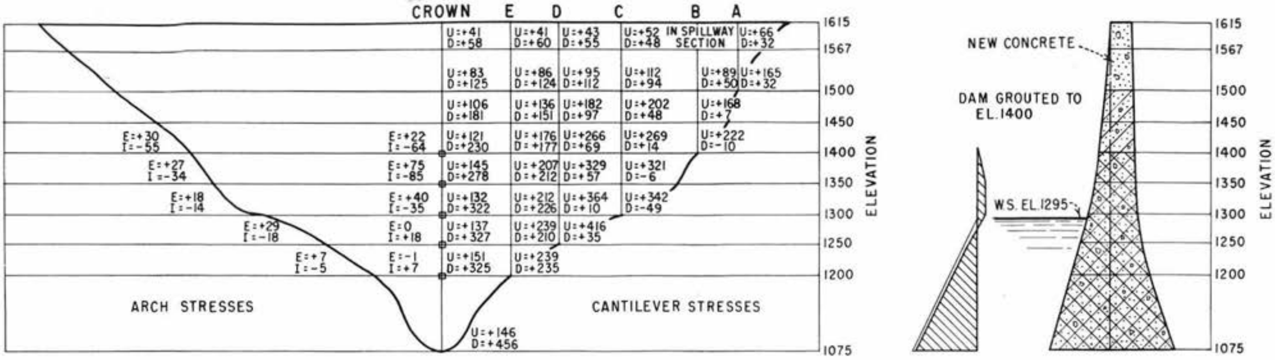


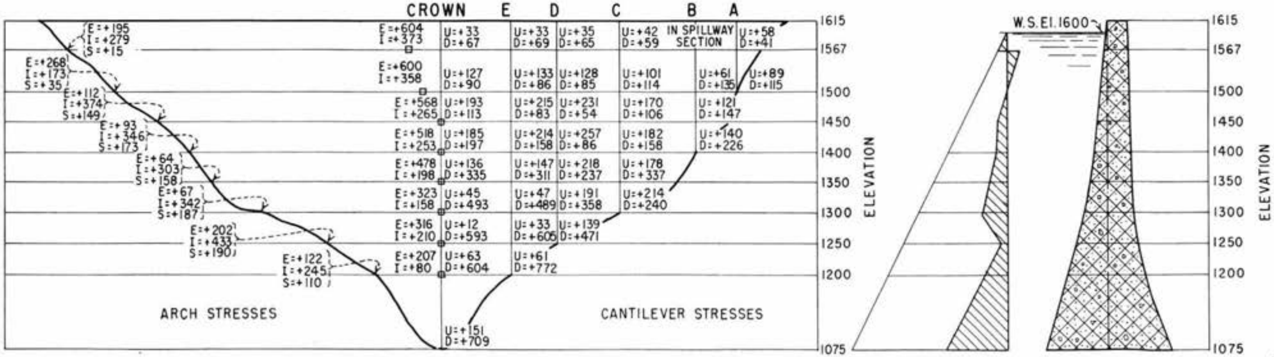
FIGURE 3



STRESSES – PART III

LOAD SECTION

FIGURE 4



STRESSES – PART IV

LOAD SECTION

FIGURE 5

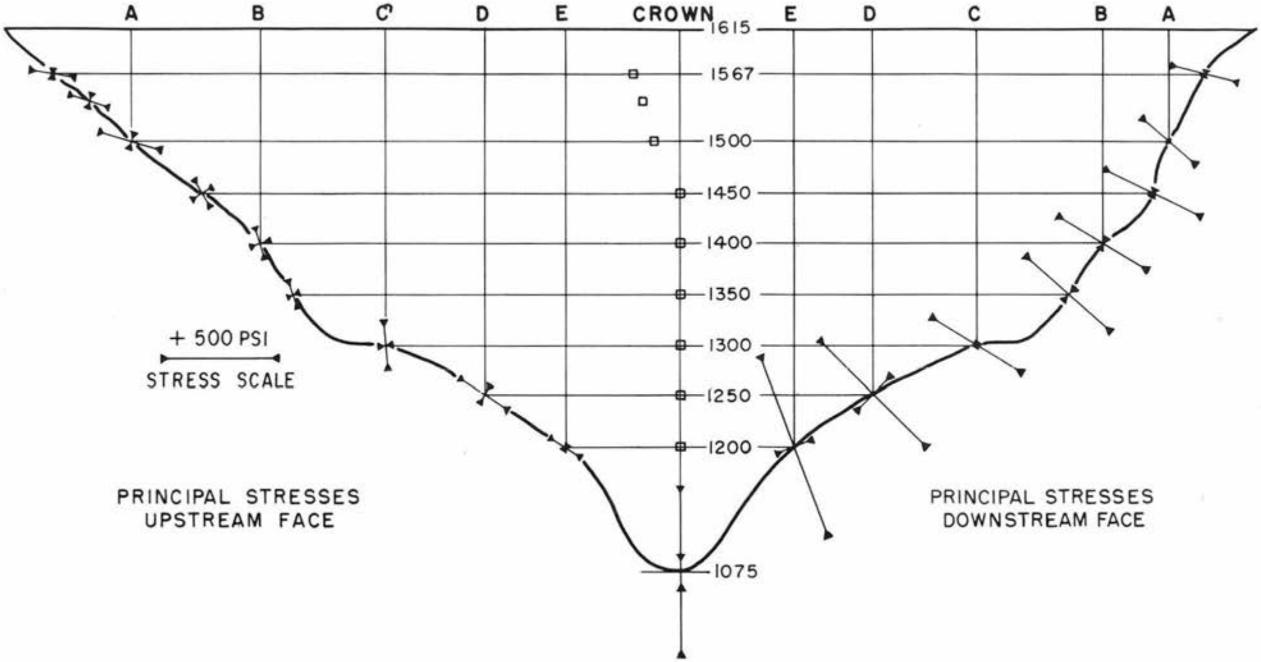


FIG. 6 PRINCIPAL STRESSES

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ARCH DAMS: OBSERVED BEHAVIOR OF SEVERAL ITALIAN ARCH DAMS

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ABSTRACT

A summary of the results obtained from the analysis of deflection, strain, temperature and other measurements on nine Italian Arch Dams is presented. The important way in which temperature changes influence arch dam behavior is shown. The results of testing for foundation modulus by seismic methods before, during and after construction of arch dams are described.

1) - Premises

1. 1) The Società Adriatica di Elettricita among a total of 25 dams has at present in service or under construction the following large arch dams (between brackets, year when the work was completed), descriptions of which may be found in various papers already published or in course of publication: 1) Comelico (1931), 2) Maina di Sauris (1947), 3) Pieve di Cadore, archgravity dam (1949), 4) Valle di Cadore (1950), 5) Val Gallina (1951), 6) Barcis (1953), 7) Ambiesta (in course of completion), 8) Pontesei (in course of completion), 9) Vajont, arch dam (under construction).

Each dam has been provided with a more or less extensive system of control which normally includes the following measurements: a) temperature and humidity in the interior of the structure; b) deflections of characteristic points of the structure and profiles of the basin upstream and downstream of the dam, by means of large range collimators, pendulums, inclinometers and high-precision triangulations and levelling surveys; c) strains and stresses in characteristic points of the structure by means of rosettes of strain and stress meters; d) various phenomena, such as the variations of uplift pressures, elastic properties of the rock, of the structure, etc.

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Close to each dam there is, as a rule, a meteorological station; near the Pieve di Cadore and Ambiesta dams a seismographic station has also been set up.

The measurements, as far as possible, are remote recorded and concentrated in a single station situated in the central control room of the intakes of the dam. This solution requires a greater length of cable than would be necessary with more measurement stations, but has been preferred on the grounds that because of the local conditions in which the dam's staff live, convenience of measurements is the basis for their success. The system of control is studied jointly by the departments charged with the planning and construction of the dam and those in charge of its operation.

In general we try to study a single phenomenon by means of different methods of measurement so as to achieve, from the possible differences that cannot be imputed to the precision of the measurement, information on the complementary phenomena which accompany what at the first approximation was considered to be the main phenomenon.

The observations are begun during the construction and continued without interruption after the dam is in service, more or less frequently, according to the sensivity of reaction of the structure to the various actions to which it is subjected.

A special office superintends the various measurements, collects them, and interpretes them in order to make the transition from the numerical figures of the observations to figures which may have a physical significance in relation to corresponding phenomena.

Actually, not all measurements can be easily and immediately interpreted, because, as it is known, they are generally the result of many factors that are not readily distinguishable. However, from the numerous series of observations available, some of the most typical have been selected concerning the functioning of dams as arch structures. The aim of this paper is, therefore, to provide some information on this subject.

1. 2) In general, to get observations of some significance, averages have been taken usually at intervals from three to thirty days. In fact, the correlation between instantaneous cause and effect cannot, in most cases be easily determined because of the multiplicity of causes which not always produce the same effects and because of the way the effects themselves occur. Besides, the search for the most probable value of the several elements in question would involve many readings for every measurement, to determine it.

In other words, given the type of phenomena under examination, the features of the instruments and the possibilities of the observers, it has been held that, in the first stage of research, an average of, for example, ten figures read successively on ten different days and so corresponding to the average conditions of the structure in that period of time, is more interesting than an average of ten values corresponding to a particular moment. The latter is an average which can give us a more accurate measurement of the state of the phenomena in that single moment, but does not allow a more general interpretation.

For geodetic survey, on the other hand, the usual procedure of compensation is adopted but duly reduced, since, excluding the first measurement, we are not concerned with the absolute figures of the various sizes, but only with the differential ones. We must note, however, that geodetic surveys also refer to an average condition of the structure corresponding to the interval of time included between the beginning and the end of the measurements. This interval, for fairly extensive triangulation nets is always of some days.

Lastly, we remind you that for some investigations mobile averages taken over a period of one year have been adopted. In fact some of the most important actions affecting the structures (temperature of the air, and the water, variations of load, etc.), show seasonal periodical variations within the year; the average values of these may be considered in many cases to be practically constant, taking into account also the attenuation in reflected phenomena. Now, if this constancy is not found in the annual mobile averages of the supposed effects (temperatures, deflections, strains, etc., of the structure) the fact can be deduced that other non-periodical phenomena have been added to the former (development of setting heat, creep of the concrete and of the rocks, inbibition of the rocks themselves, etc.). We thus have a very simple method of evaluating, in many cases, the entity of these non-periodical phenomena and in any case their progress.

2) - Measurements of Temperature

2. 1) The measurements of temperature of concrete structures are known to be among the simplest and quickest. There is also a good agreement between experimental results and those deduced from the application of Fourier's theory, when the range of external temperatures (air and water), the heat and specific weight of the concrete together with the coefficient of thermal diffusion are known.

In a given structure the number of instruments may therefore be reduced to a few for reference and control, concentrated mainly near the faces and the middle where, because of the possible interference of thermal waves coming from upstream and downstream, experimental figures less in agreement with the theoretical ones are obtained. An example of the correspondence between theoretical and experimental results is illustrated for the Lumiei dam in Fig. 1.

2. 2) The method of mobile averages has been adopted in order to determine the end of the period of dissipation of the setting heat of the concrete, being supposed, as mentioned before, that the average annual range for the temperature of air and water do not vary greatly from one year to the next.

When these mobile averages of temperatures in the interior of a dam have an asymptotic trend, it is right in assuming that the setting heat is practically exhausted: in Fig. 2 one example of this diagram is shown which illustrates how for the Pieve di Cadore dam the asymptotic trend is reached at the middle of a section of 19,3 m thickness after about 40 months.

2. 3) To give an idea of the annual ranges of temperature in the interior of the concrete dam, in the diagram of Fig. 3, for some of the structures in question, the figures checked at the middle of various sections, are recorded. These figures are not necessarily the lowest for section under examination since, at the middle there are often interferences of thermal waves from upstream and downstream.

The diagram clearly shows how for the Pieve di Cadore and Val Gallina dams, which have the same technological qualities (cement and aggregates), not very dissimilar graphs are obtained; the Lumiei dam on the other hand, differs in its behaviour having different technological and climatic features.

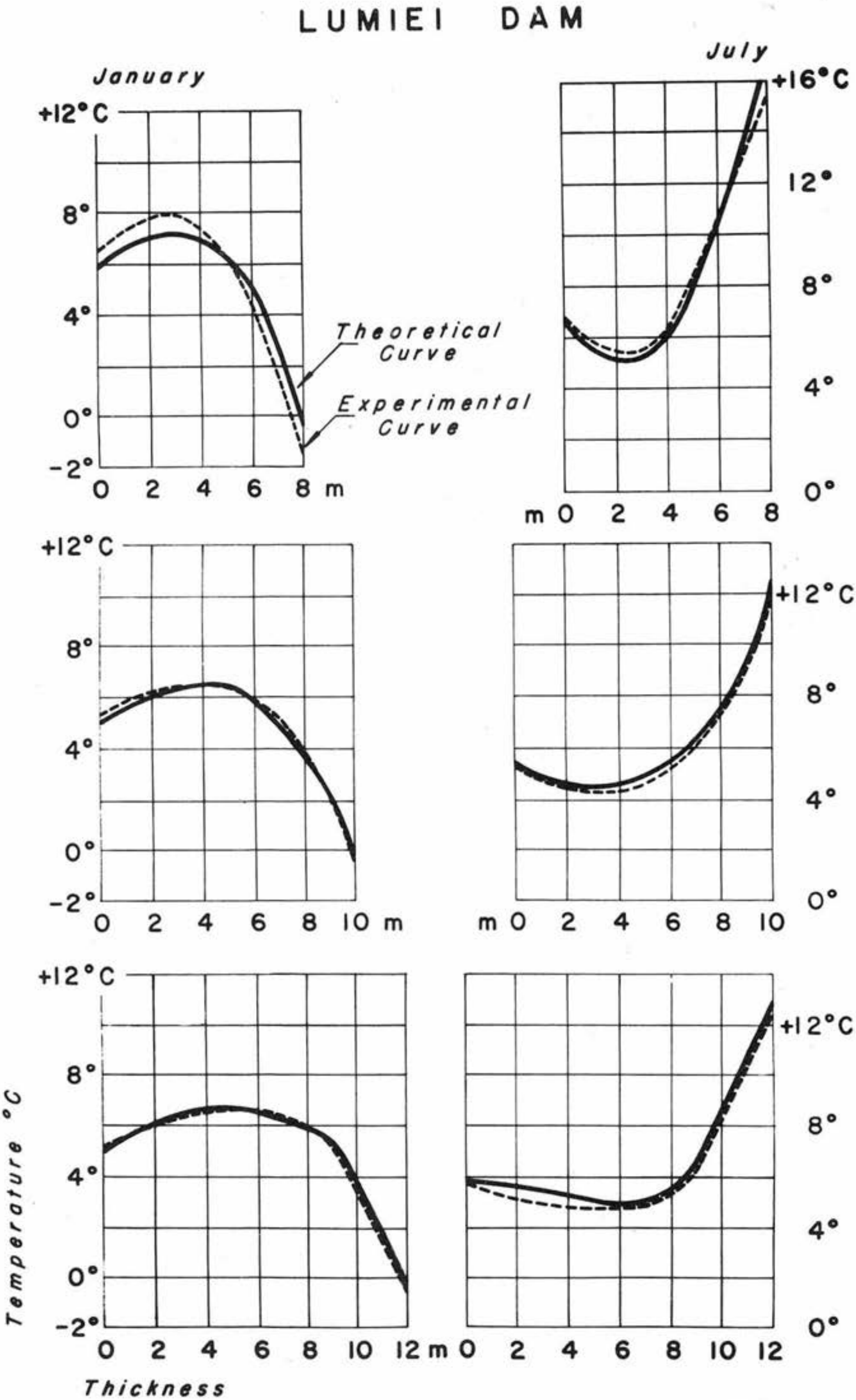

Fig. 1. Lumiei dam: distribution of temperatures, comparison between theoretical and experimental results with maximum and minimum external temperatures (July, January).

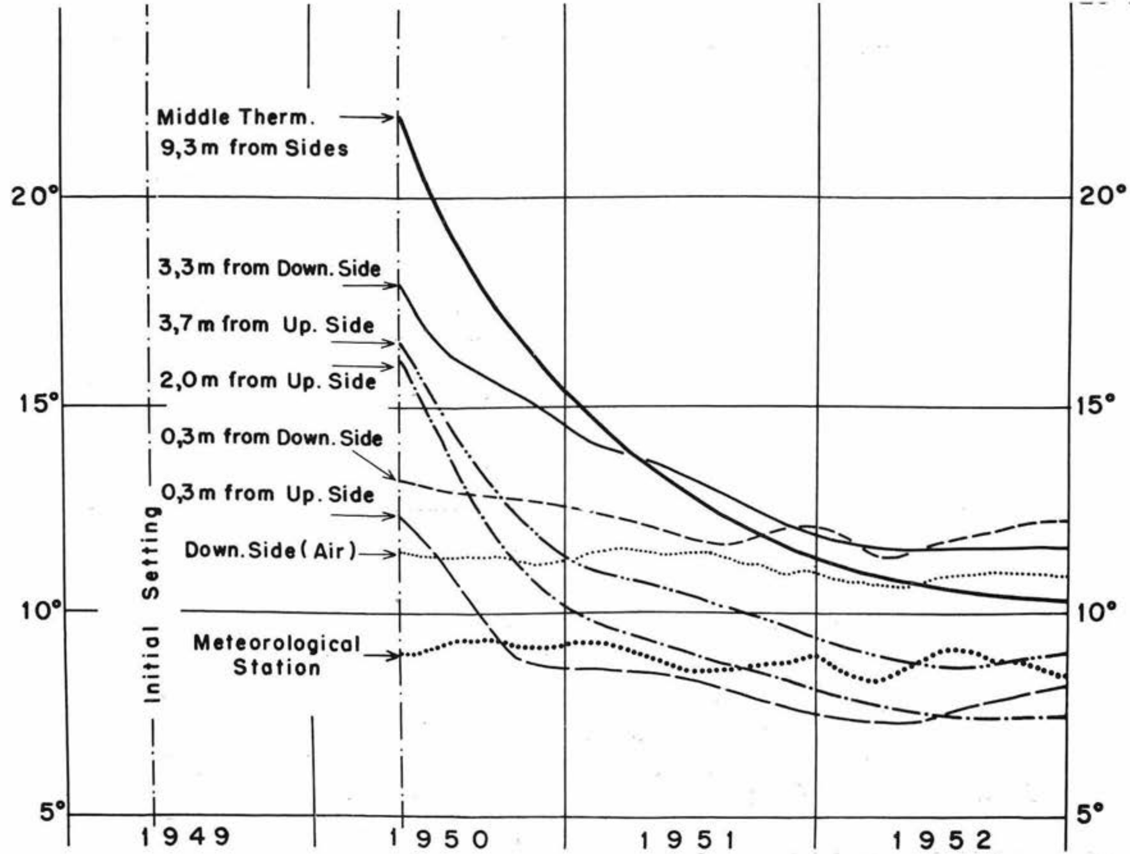

Fig. 2. Pieve di Cadore dam: mobile annual averages of temperatures in a section of a thickness of 18.6 m (exhaustion of setting heat).

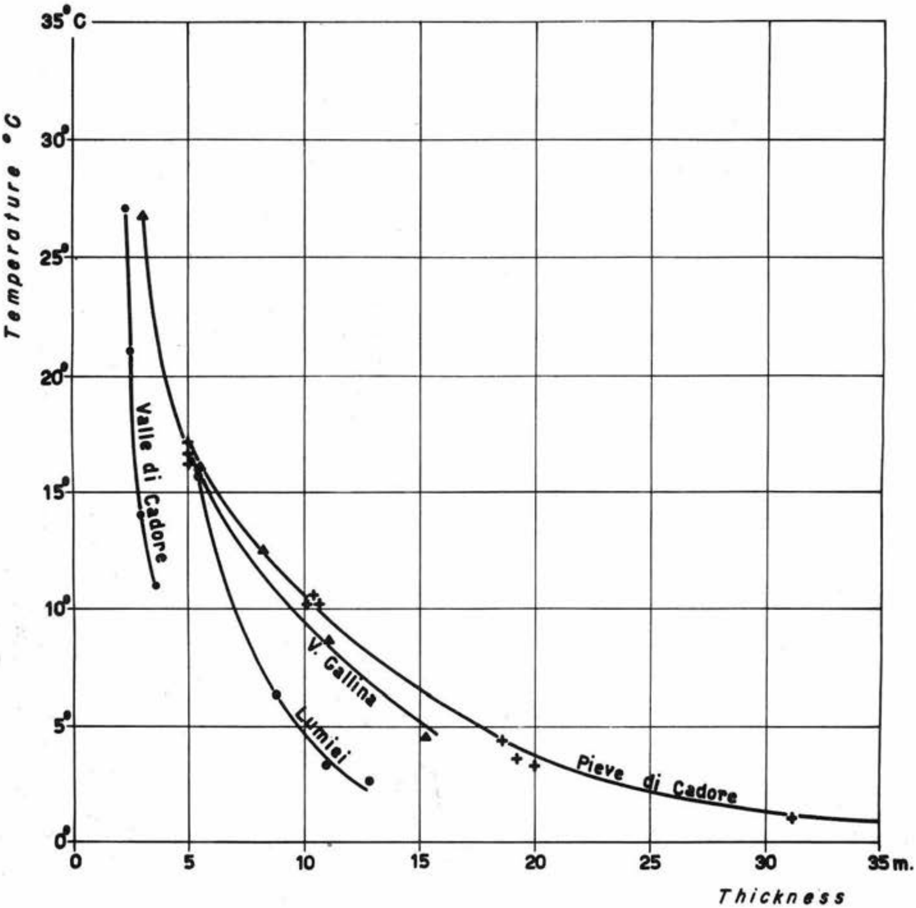

Fig. 3. Pieve di Cadore, Val Gallina, Valle di Cadore and Lumiei dams: range of temperature at the middle of sections of different thicknesses.

Finally, notably lower figures are found for the Valle di Cadore dam which has practically the same technological and climatic features as the Pieve di Cadore dam, but where the structure is protected upstream and downstream by concrete block facings of 0.30 m thickness, which clearly reduces the range of thermal variations.

On this matter it must be noted that the protection mentioned was not built to reduce the thermal variations but against the effects of frost. We believe that satisfactory protection against thermal variations could rather be achieved economically by the use of antiradiation vernish on the downstream face.

2. 4) An eloquent illustration of the temperature trend along a section of the structure as a function of the time is obtained with contour line diagrams of which Fig. 4 is a type, taken from the Pieve di Cadore dam. These diagrams can easily be used for typical stereometric drawings (Fig. 5).

From the combined investigation of these diagrams relative to the greatest possible number of sections of the dam, the average temperature of the dam (or average thermal regime) can be found. The thermal regime cannot be expressed, as sometimes done, only by means of the variations of temperature of the air and water since these variations (to which should be added those of the setting and of the rock temperature) lead, on account of successive interferences and displacements, to a thermal regime of the structure the more complex the more, as a whole, it is behind time in relation to the determining waves.

Among these waves, those coming from the downstream face have a sufficiently regular and periodic trend which is not true for those coming from the upstream face by reason of the succession of filling and emptying of the reservoir.

Still other factors interfere in the formation of the thermal regime of the structure, particularly solar radiation and action of the wind. Although the reciprocal influences of these factors cannot easily be distinguished (depending among other things on the thermal regime pre-existent in the structure), the results obtained from the contour line diagrams clearly show their importance. The fact, for instance, that a structure may, for some months even, be subjected to a partial solar radiation causes, on the sunny side, a thermal regime totally different from that of the other side which has always been in the shadow as has been found by experiment for the Valle di Cadore and Pieve di Cadore dams. For this reason a skew thermal load is established which is in contrast to the hypotheses of symmetry usually assumed by the designer.

2. 5) The finding of the thermal regime by means of the actual average temperature of the dam is certainly not immediate: requiring the drawing of fairly close isotherms on parallel and perpendicular planes and their graphic integration.

In practice, after some years of observation, it is possible, as a rule, to refer to a number of thermometers whose average observations can represent reliably enough the average temperature of the whole structure.

As an example, in fig. 6 diagrams of the average temperatures of the Pieve di Cadore, Valle, Val Gallina and Lumiei dams are shown compared with the average temperatures of the air (at intervals of ten days): these diagrams point out the differences mentioned before.

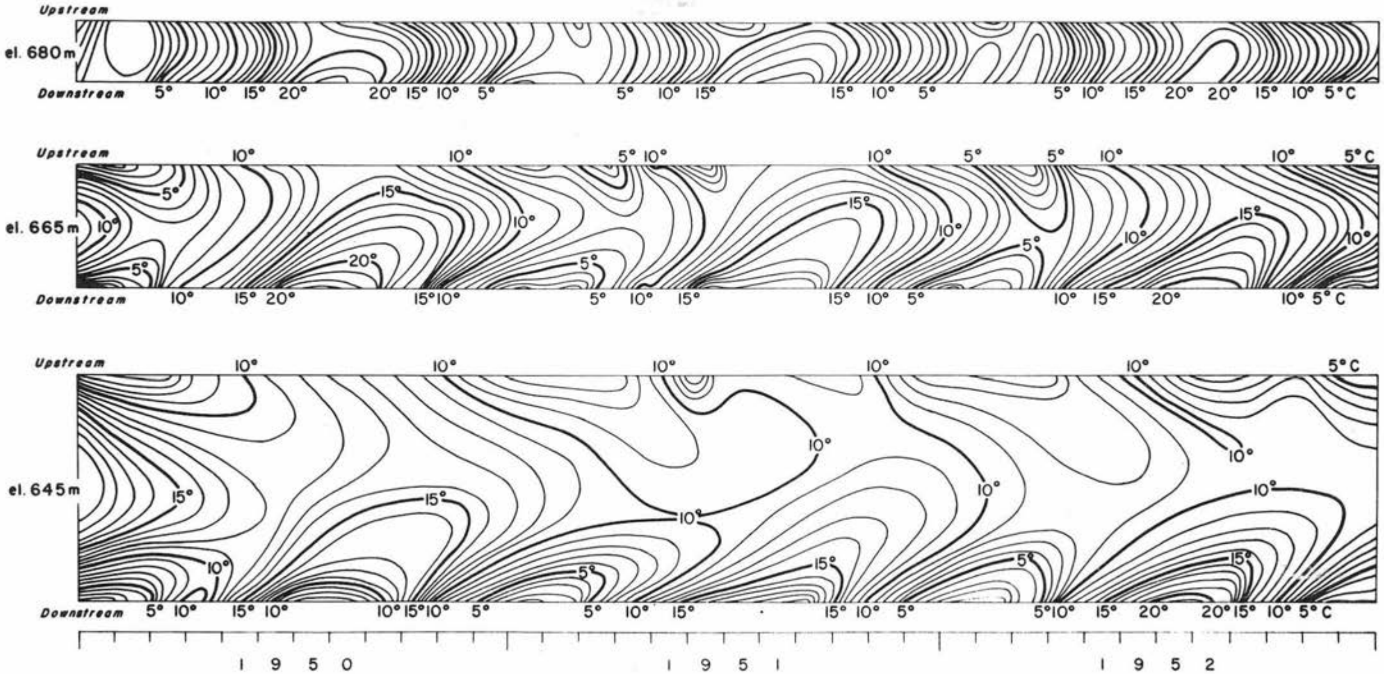

Fig. 4. Pieve di Cadore dam: distribution of temperature in relation to time and thickness, at different elevations.

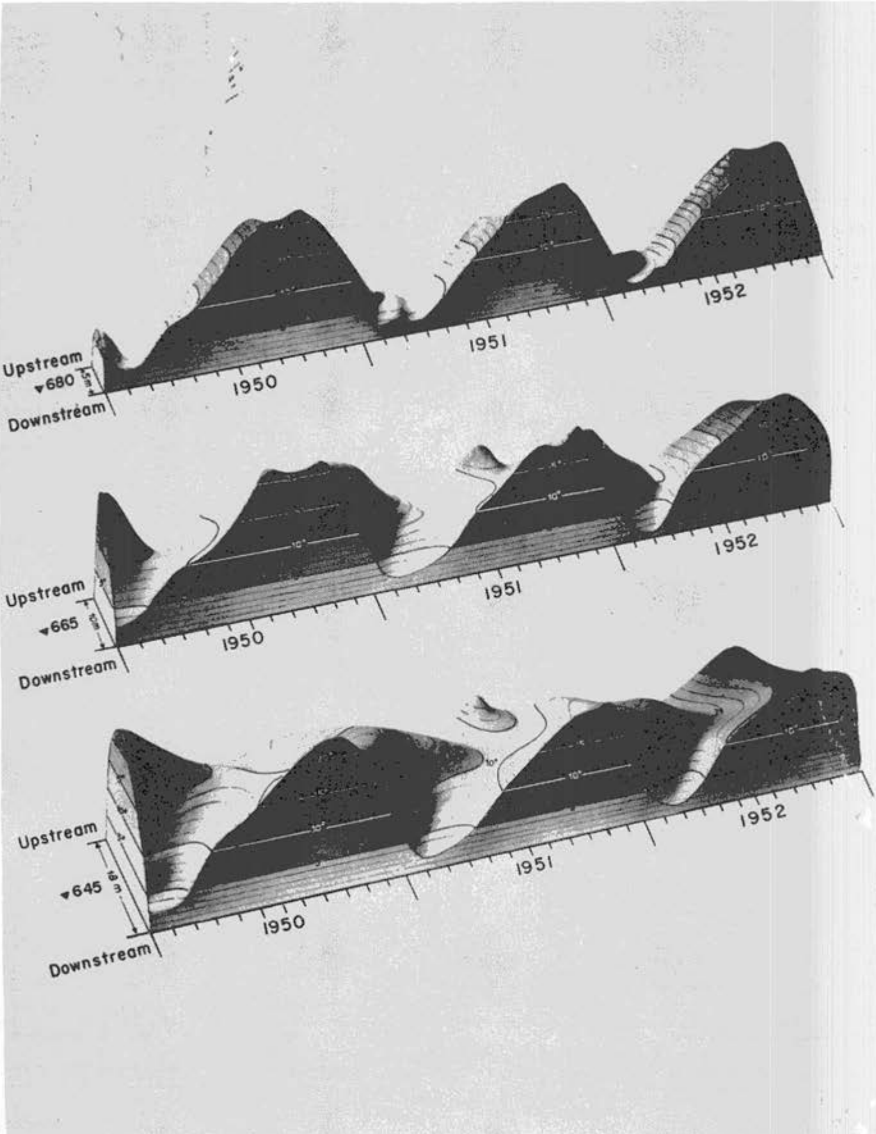

Fig. 5. Pieve di Cadore dam: stereometric representation of the distribution of temperature in relation to time and thickness at different elevations.

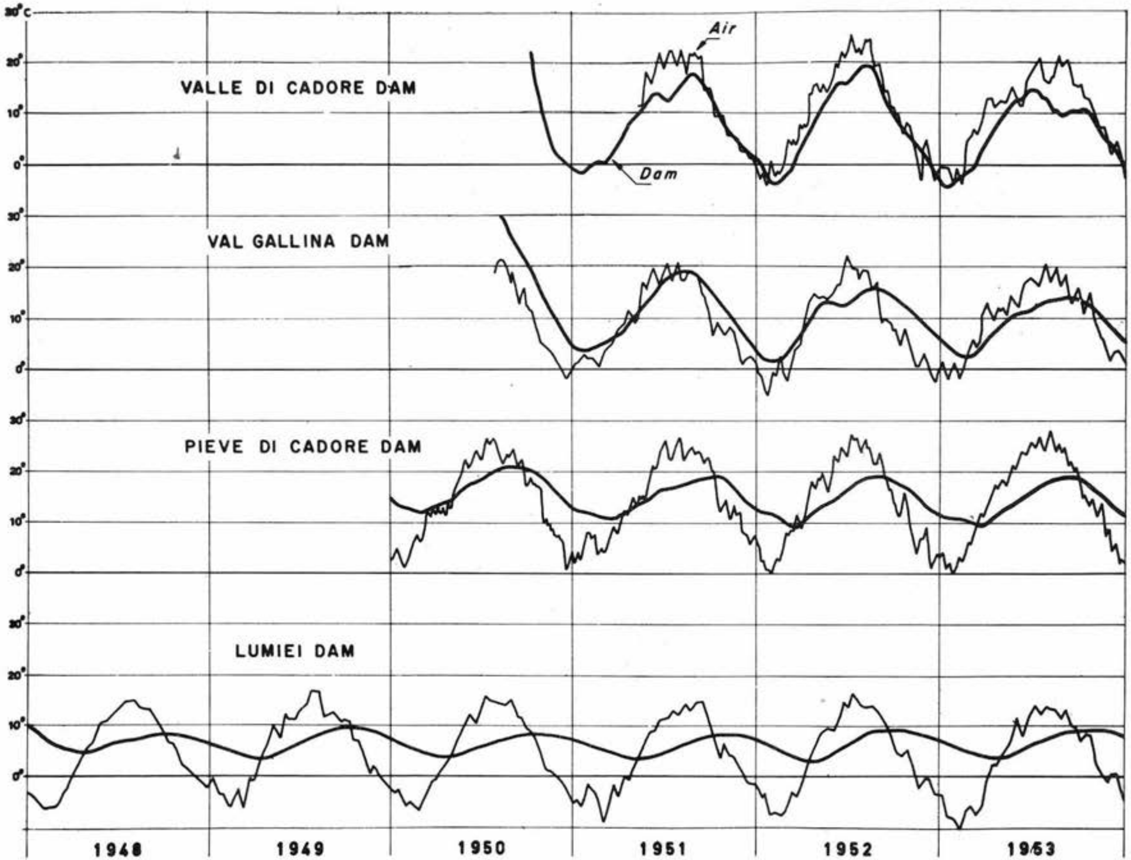

Fig. 6. Pieve di Cadore, Val Gallina, Valle di Cadore and Lumiei dams: average temperature of the dams.

But to know the thermal regime of a dam is not enough to calculate the thermal load which in turn is a function of the distribution of the regime itself in the interior of the structure.

Good results are obtained, on this point, by expressing the thermal load as a function of the difference between the average temperatures of the crest and foundation arches and the difference between the average temperatures of the downstream and the upstream faces.

Even if we confine these differences to the temperatures of a given section (the crown one), fairly reliable results can be quickly obtained, when the dam is subjected to a uniform solar radiation and ventilation.

3) - Measurements of Deflections

3. 1) The correlations between the variations in the thermal load and deflections in the structure are not immediate, not only because of the above mentioned difficulties of defining the thermal load, but more especially for the continuous interference by the actions due to the hydrostatic load, the evolution with time of other factors (creep and plasticity of the concrete and rock) and the quite frequent occurrence of non-periodic events such as microseisms, seiches, waves, etc.

Yet research with inclinographs has clearly shown a rotation of the structure due to the action of the daily thermal wave which is known to penetrate the mass of the concrete for 0.40 ÷ 0.60 m. This rotation which in the Pieve di Cadore dam reaches a maximum of $10^s \div 15^s$ occurs on days of high solar radiation, and is therefore at its minimum on cloudy or rainy days, even if the range of temperature is pronounced. The diagrams of Fig. 7 illustrate the phenomenon: the greater the difference of temperature between the upstream and downstream faces (a difference which, as observed, may be considered typical of the thermal load of the structure), the more this phenomenon is accentuated. A time-lag of about four hours between the maximum difference and the maximum deflection has been noted.

The diagram also indicates a progressive rotation towards upstream, so that at the end of each daily cycle the point does not return to the point of departure: this is due to the continuous increase of the temperature load.

The observations have brought to light another fact: that the influence of the daily thermal wave decreases with the time: probably on account of the hardening of the structure, caused by the increment of the modulus of elasticity.

3. 2) The quantity of the rotation due to the daily thermal wave is obviously linked with the type of structure and the functioning of the reservoir. The rotations recorded for the Val Gallina dam, for instance, where there are marked daily variations of load on account of the requirements of the nearby power station of Soverzene, are clearly the result of variations in the thermal and hydrostatic loads (Fig. 8).

Now by analyzing brief periods of rapid filling and emptying of the reservoir during which only slight variations in temperature are verified, it has been possible to determine an approximately linear relation between loads and rotations (from elevation 632 m to 642 m the rotation is of $6^s\,6$ for 10 m of hydrostatic load).

Using this relation, the daily global rotation has been depurated of the quota due to the hydrostatic load, thus obtaining a residual diagram, which should express the rotations exclusively due to the thermal load: this occurs fairly satisfactorily with a rotation of $1^s\,7$ for each 10^o C of thermal load.

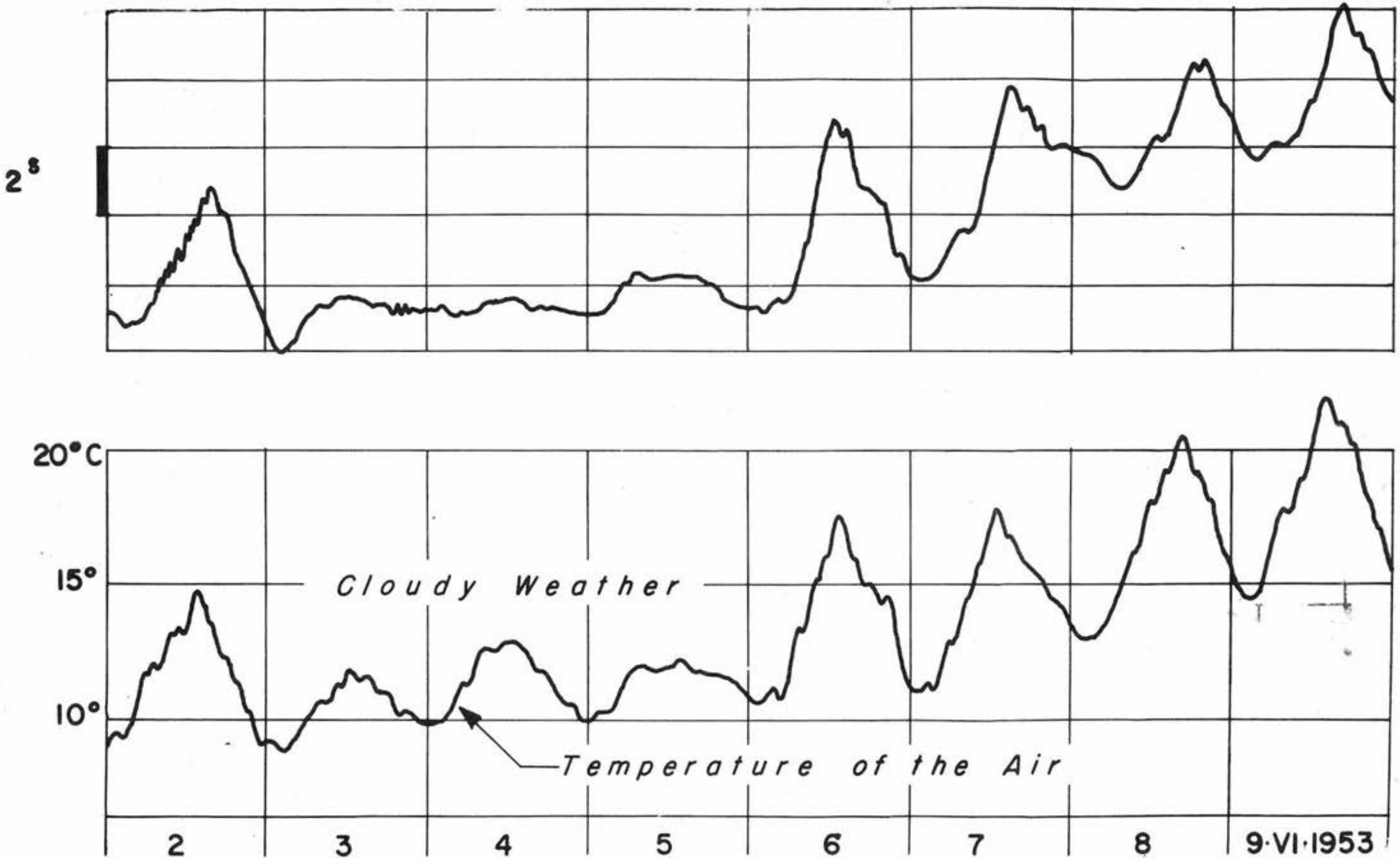

Fig. 7. Pieve di Cadore dam: daily variations of temperature and correlative rotations (Block XIV at elevation 624).

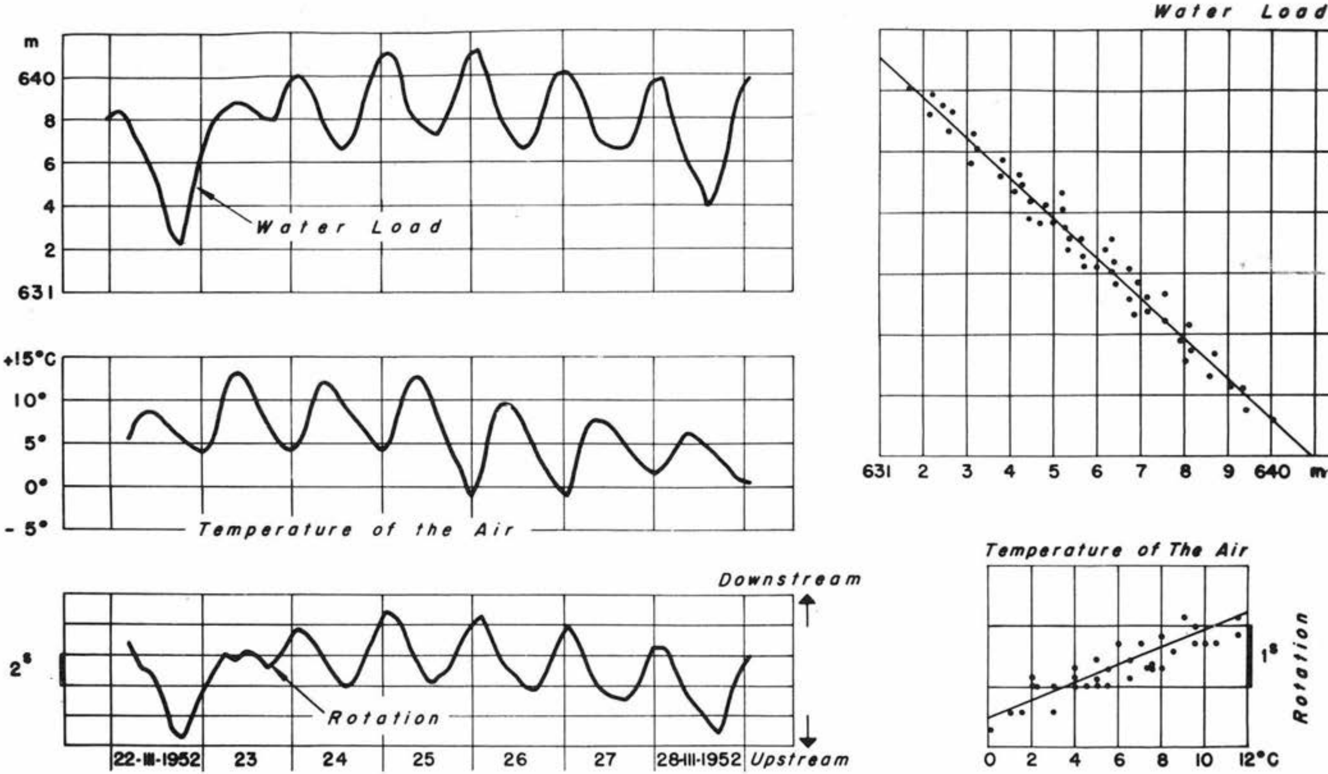

Fig. 8. Val Gallina dam: correlations between rotations and thermal and hydrostatic loads.

3. 3) In the examples studied, some correlations between the daily rotations of the structure and the daily variations of both the thermal and hydrostatic load, have been illustrated. If we are considering longer periods, however (seasons, years) because of the often mentioned difficulties of distinguishing the various acting forces, research must be extended not to the rotations alone, but to the totality of all deflections, as recorded (for the structure itself and continuously) by pendulum and collimator measurements: as shown for the Pieve di Cadore dam in diagrams of Fig. 9. This research is not always possible with geodetical surveys, considering their intermittency and the length of time needed for taking them. Geodetical surveys, however, have the undeniable advantage of giving the absolute movements of the structure. In Fig. 9, these movements are also shown (taken on the crown at the crest of the Pieve di Cadore dam) together with the relative movements which would be obtained by operating with a triangulation network from the collimation stations. The latter obviously coincide (apart from various errors due to the fact that the measurements are non-simultaneous) with the analogous deflections read from the collimator.

The difference between deflections done by pendulum and collimator measurements is due to the reciprocal movements of the dam and foundation rock. Further information on these reciprocal movements can be obtained by measuring the deflections of metal tubes inserted in the rock itself. Since practical necessities limit the length of these tubes, the data that can be noted are restricted to the corresponding deflections of the dam and the strata of rock in direct contact with the foundation of the dam itself.

3. 4) With reference, then, to pendulum measurements, we show as an example those noted for the crest arch in relation to the foundation arch of the Pieve di Cadore dam, for three typical sections. The observations refer to radial and axial deflections and, as previously indicated, in order to have an expressive illustration, mobile averages of 12 months were used (Fig. 10).

On the whole there is a tendency for the point at crown (block XIV), in radial deflections to move downstream: this tendency was accentuated after May 1952, but seems to have diminished in the latest measurements.

For the two lateral sections (blocks IV and XIX) we have the opposite tendency (upstream), still with a critical point in May 1952. This critical point appears more clearly in the diagrams of axial deflections which again show, though in the most recent measurements, a tendency towards stabilization.

The result is that because of the effect of the thermal and hydrostatic loads, the structure as a whole has settled in such a way as to diminish the curvature by a deflection downstream of the middle part and upstream of the haunches, while the axis has lengthened especially towards the right abutment.

These alterations are due to the settling of the structure, the "pulvino" (cushion), the foundation rock and the plug, but it has not yet been possible to differentiate between them.

As for the critical point which occurred in May 1952, an interpretation might be based on the fact that, in that year, unlike preceding years, there was no complete emptying of the reservoir down to elevation 630 m but only to elevation 644 m. The structure while still at the settling stage was therefore subjected to a prolonged load which cannot but have influenced the phenomena of creep and plasticity of the structure and of the rock. This hypothesis is partly confirmed by the fact that the measurements of the modulus of elasticity of the rock showed a diminution of the modulus from 1949, prior to the first

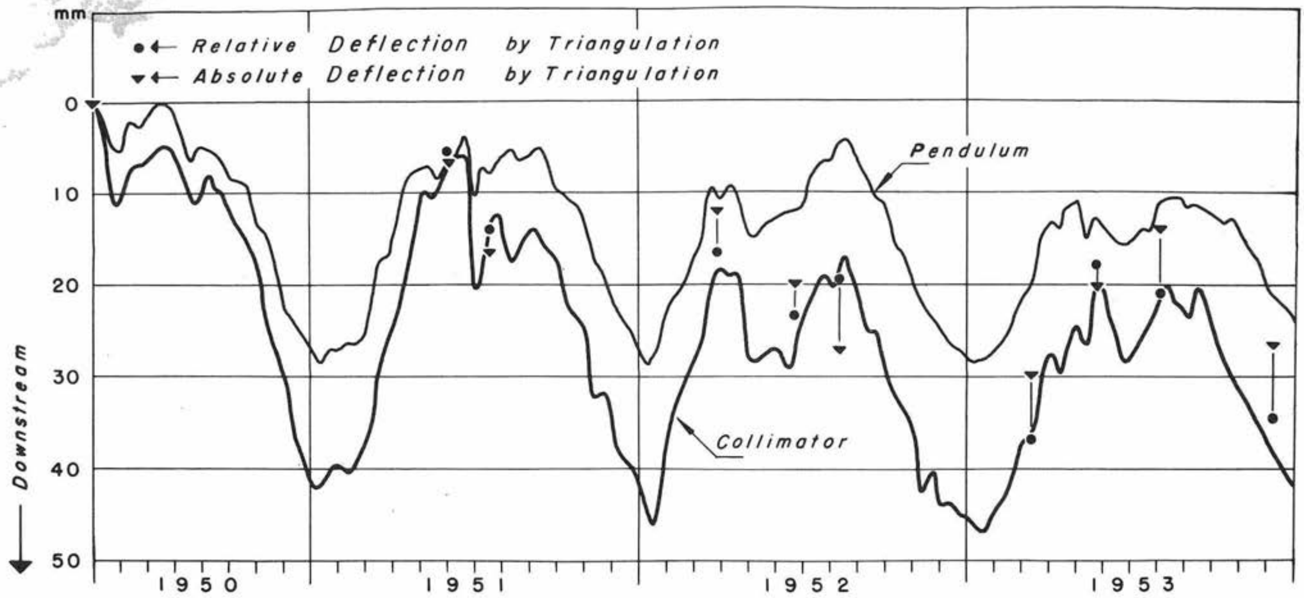

Fig. 9. Pieve di Cadore dam: deflections of the crown point of the crest arch according to the pendulum and the collimator.

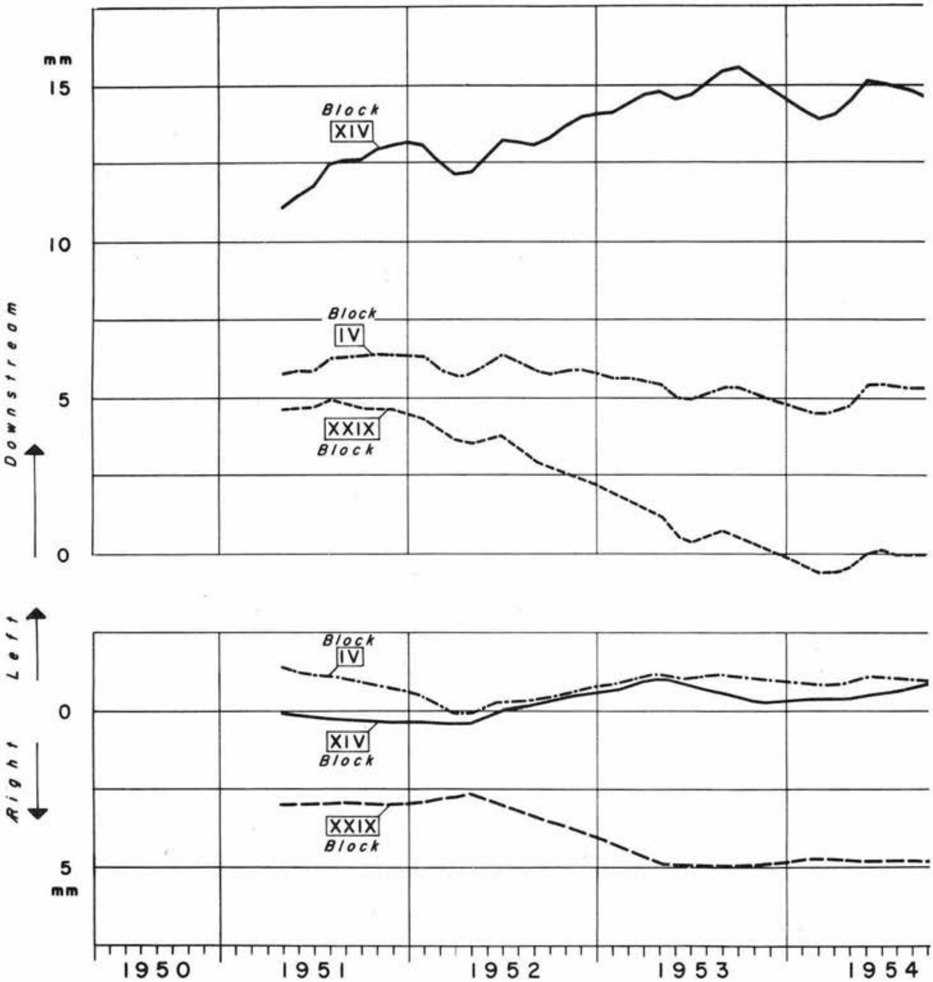

Fig. 10. Pieve di Cadore dam: mobile annual averages of radial deflections (upper) and axial deflections (lower) of the crown cantilever according to the pendulum.

complete filling, to 1952, while since this period the modulus has remained practically constant.

3. 5) Still speaking of the Pieve di Cadore dam, on completion of the research mentioned above, the results obtained with inclinometers were compared with those given by pendulums.

The comparisons were made for the three named sections, one approximately central (block XIV) and two lateral (blocks XXIX and IV) at elevations 680 m, 660 m, 630 m (625 m for section XIV) and they refer to average monthly figures in order to prevent any possible inaccuracy in the measurements.

On the whole the rotations expressed by these two methods correspond closely, but always with higher figures for the inclinometers, which indicates that the rigidity of the structure varies according to the height; the section with the greatest increase of rotation and consequently the least rigidity is from elevation 660 m to 680 m. As an example, Fig. 11 shows the average yearly range, in these figures carried out between May 1952 and December 1955.

Thus, as could be foreseen, the widest deflections marked by the pendulums are on the crown, the lateral ones being smaller and practically symmetrical. As for the rotations, on the other hand, those at the base are practically equal: at elevation 660 m there is a maximum on the crown, and lower fairly symmetrical figures at the abutments: lastly at elevation 680 m there is a maximum at the right side with decreasing figures as we move towards the crown and the left side. This is probably due to the fact that on the right side, although the rock has a greater elasticity than that of the left side, there is a lesser resistence to the action of the arch, on account of its morphology.

3. 6) The deflections of the crest arch of the Pieve di Cadore dam have been the subject of further study, carried out between May 1950 and April 1954, which completes the one already presented at the 5th Conference on large dams, held in Paris.

In this research, use has been made of the numerous experimental data available (averages of the measurements over periods of three days) introducing them as coefficients in a system of equations in which the unknown quantities should represent the laws of variation of the deflections as a function of the acting causes. This has been done by the method of the least squares in order to calculate the most likely values of the parameters.

The deflections in question (1) expressed in mm and referring to the foundation line, have been considered a function of the difference between the average temperatures (θ_a expressed in ^oC) of the crest arch at elevation 680 m and of the foundation arch at elevation 624 m; of the difference between the average temperatures of the downstream and upstream faces considered in their whole extent (θ_p expressed in ^oC) of the hydrostatic load (h) expressed in meters starting from elevation 630 m and of an asymptotic function of time (t) expressed in months beginning in May 1950 in correlation with the creep and plasticity of the concrete and the rock.

The expression resulting of the type

$$1 = \alpha_1 \theta_a + \alpha_2 \theta_a^2 + \alpha_3 \theta_a^3 + \beta_1 \theta_p + \beta_2 \theta_p^2 + \beta_3 \theta_p^3 + \gamma_1 h + \gamma_2 h^2 + \gamma_3 h^3 + \delta_1 e^{-\delta_2 t} + k$$

has been resolved: the values of the unknown parametro are practical applications proved to be the following: $\alpha_1 = +1{,}18$ mm/^oC

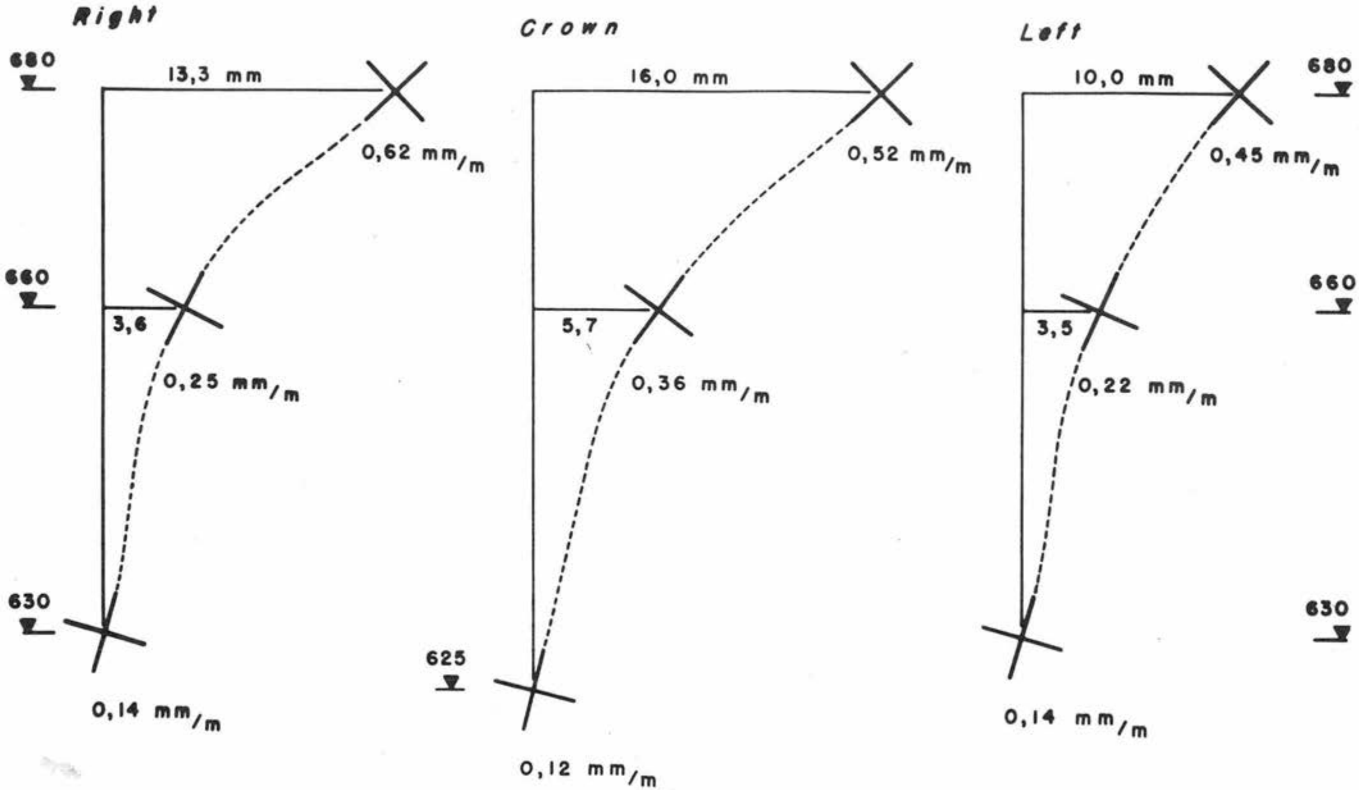

Fig. 11. Pieve di Cadore dam: average annual range of deflections and rotations for three cantilevers.

$\alpha_2 = +0{,}0073$ mm/°C $\quad \alpha_3 = -0{,}002983$ mm/°C $\quad \beta_1 = +0{,}43$ mm/°C
$\beta_2 = +0{,}0232$ mm/°C $\quad \beta_3 = -0{,}001479$ mm/°C $\quad \gamma_1 = -0{,}04$ mm/m
$\gamma_2 = -0{,}0012$ mm/m $\quad \gamma_3 = -0{,}000029$ mm/m $\quad \delta_1 = +7{,}16$ mm/month
$\delta_2 = 0{,}109$ mm/month $\quad k = +9.88$

The features of this research are summarised in Fig. 12.

Using the same data, for the deflections under examination, we have sought to classify the several effects according to their single causes, as they have been postulated. That is to say, we have verified the persistence of the loads by means of the result of single deflections for the time of their duration. In this elaboration clearly no account has been taken of the phenomena connected with time (creep, plasticity, etc.) since it is to be supposed that the external causes would every time affect a settled dam.

The following classification of the deflections results:

Quota due to the thermal load:	
I) difference of temperature between the crest and foundation arches (α)	32 %
II) difference of temperature between downstream and upstream faces (β)	13 %
	45 %
Quota due to the hydrostatic load (γ)	45 %
Quota due to various unspecified causes and to errors	10 %
	100 %

The same research has been carried out for the Val Gallina dam also, obtaining satisfactory results, of the same type as those seen for the Pieve di Cadore dam.

3. 7) Geodetical surveys have been adopted also to check the movements of the upstream shores of the reservoir caused by successive fillings and emptying. There are 14 measurements for the Pieve di Cadore dam, for the first 8 of which there is a satisfactory correlation between the filling of the reservoir and the approach of the shores: this correlation, however, is not maintained in measurements taken after May 1952.

For the Val Gallina and Lumiei dams the measurements taken are not sufficient in number to define the phenomenon with precision; this is thus confirmed only for the first two years that the Pieve di Cadore dam was in operation.

On this matter, apart from the errors, not always mutually eliminating themselves which affect these measurements, it is considered that the thermal conditions of the rock (all the dams mentioned are built on limestone) and the thermal regime of the structure itself cannot but influence the movements of the shores: besides, the fixed stations of the triangulation network, however carefully constructed, are still anchored in the surface stratum of rock, particularly subject to thermal variations.

The results observed for the Pieve di Cadore dam would lead to the conclusion, already noted for other phenomena, that the rock, as the structure, is subject to movements of a certain significance especially in the first cycle of the working of the reservoir; subsequently a settling takes place to which,

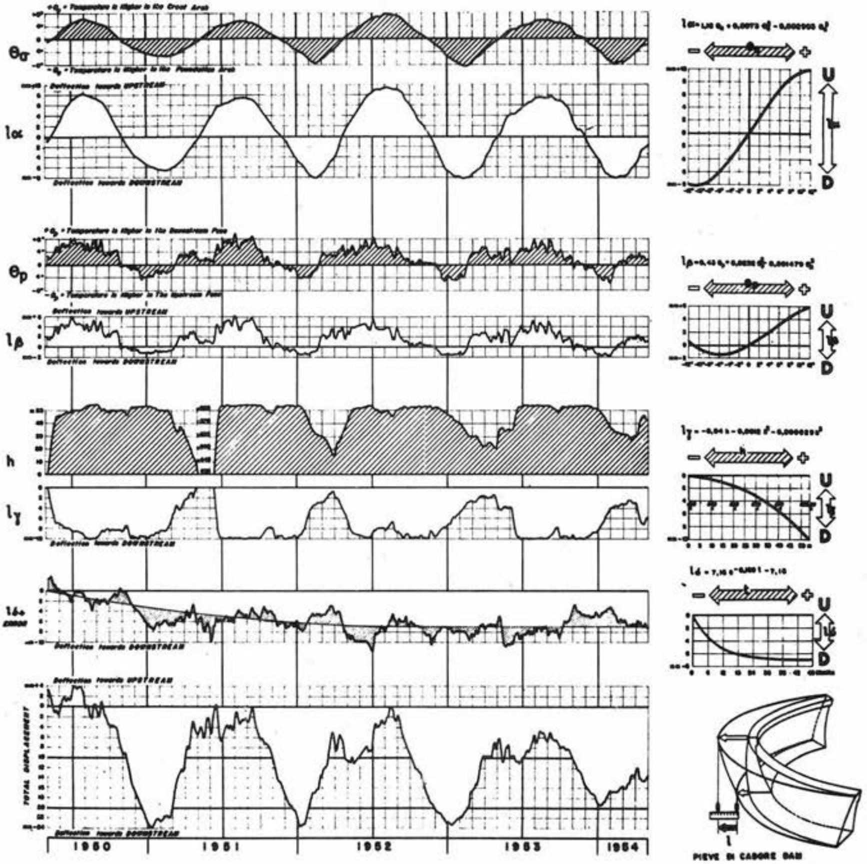

Fig. 12. Pieve di Cadore dam: analysis of deflections of crest arch.

in this specific case, in some degree corresponds the uniform imbibition of the rocks. But the problem is too closely connected with geological and morphological characteristics and those of the cycle of the reservoir to be possible to make for it any generalizations.

From what has been said, however, the great importance of research on the actions and reactions of the rock is made evident: research which up to date has perhaps been subordinated to that on the structure in itself.

4) - Measurement of Deformations

4. 1) Numerous data relating to deformations are recorded, for the structures under examination, by acoustical and resistance strain-meters either single or in rosettes. The strain-meters are generally laid in the interior of the dam at $0{,}60 \div 1{,}00$ m from sides to eliminate the troubled external zone.

But the elaboration from the elastic deformations to stresses, as we know, is not immediate because of the actual lack of information on the accuracy of the values of variations of the modulus of elasticity.

This modulus increases with time, but this increase generally is opposed and overcome by the phenomena of plasticity and creep due to the action of the loads.

For a structure subjected to continuous or periodical loads it is convenient to adopt a conventional modulus based on the behavior of the structure itself to the total of the various actions of elasticity, plasticity and creep.

Various methods have been suggested for finding this modulus for the structure as a whole (jacks, residual tensions etc.) since results from tests in the laboratory have no meaning in this field of research.

Besides the seismic method which will be mentioned later, research on tests loaded accorded to Prof. Oberti's method has been chosen, followed here with some variants: these are prismatic tests of about 2 x 0,2 x 0,2 m merely supported at the ends and loaded, including their own weight to get a stress of 15 and 25 kg/cm^2.

The load is applied 28 days from the setting and the deformations are recorded periodically on the upper and lower faces by means of removable strain-meters.

The diagrams of Fig. 13 show the variation of the deformation for elasticity, plasticity and creep due mainly to the load (half the difference of the upper and lower sides deformations) and the variation of the deformation principally due to actions of shrinkage, thermal expansion etc. (half the sum of the upper and lower sides deformations).

From the study of these deformations it appears that the initial modulus, of 260 000 kg/cm^2, is reduced, after about 40 months of load to 145 000 kg/cm^2 for the tests loaded at 25 kg/cm^2 and to 165 000 kg/cm^2 for those loaded at 15 kg/cm^2.

It is interesting to observe how the ratios of creep (ratio between the initial modulus of elasticity and the final conventional modulus) is a little less than 2: a relatively low figure compared with those met in other experiments of the kind.

The data just examined referred to tests of concrete used for the Val Gallina dam: experiments of the same type have been made also for the Pieve di Cadore dam which confirm the previous results. But even these figures of the moduluses calculated on tests in particular conditions of setting and load cannot give an idea of the true average modulus of the whole structure, i.e., the only element that can represent the resistance of the structure

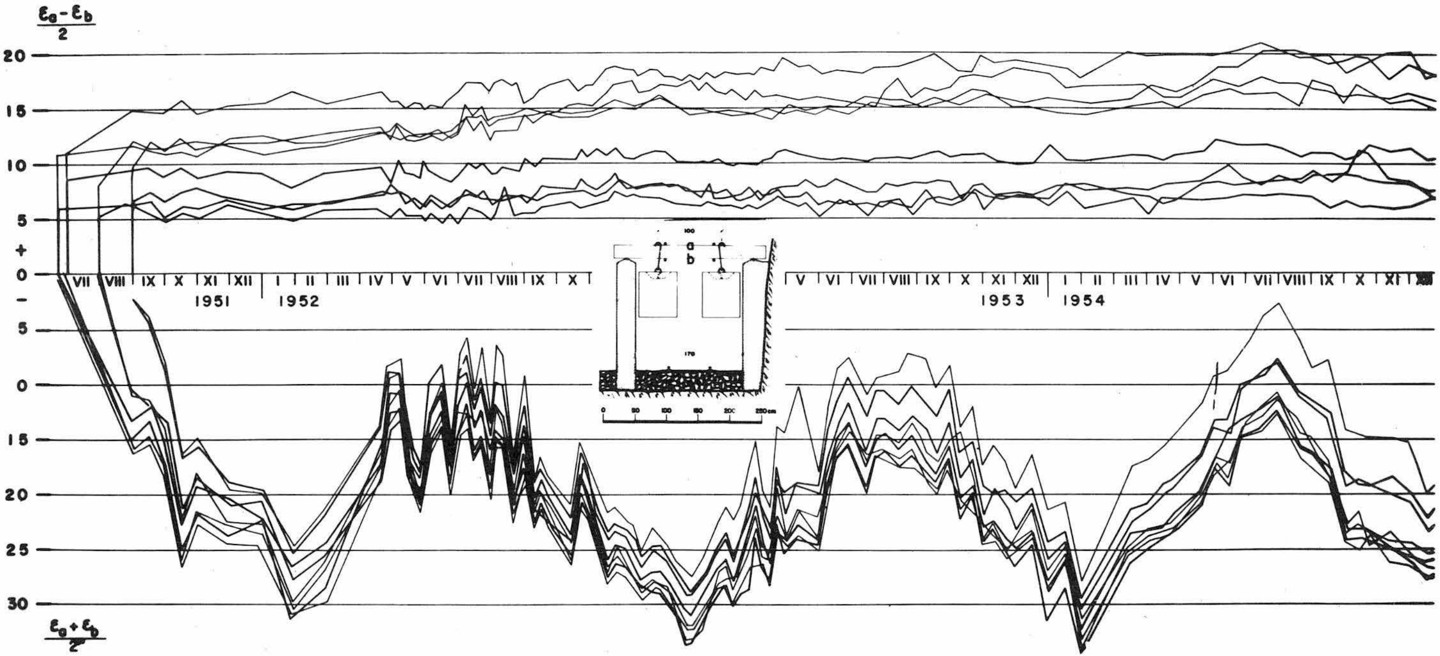

Fig. 13. Val Gallina dam: deflections in relation to time of upper and lower side of prismatic tests.

itself and that can be obtained on the contrary from tests on models. For this it would be necessary that the ratio between modulus of the tests from material of model and average total modulus of the model would have to be equal to the ratio between modulus of the test from material of structure and average total modulus of the structure.

5) - Geophysic Measurements

5. 1) Interesting information on the modulus of elasticity of the structure and of the rock can easily be obtained by seismic methods by which we can readily determine a dynamic modulus of elasticity, more or less closely correlated with the static modulus of elasticity with which we usually deal. Whatever method is used to discover the dynamic modulus it is always a question of rapid and economical processes which afford a truly statistical value of the modulus which are of interest. This also affords comparisons useful for a preliminary evaluation: for rocks of the limestone type, for example, Mr. Semenza has proposed a system of comparative evaluations of the elastic properties of the rock (middling rock up to $2,5.10^5$ kg/cm^2; good rock from 2,5 to 5.10^6 kg/cm^2; very good rock from 5 to $7.5.10^5$ kg/cm^2; excellent rock 7,5 to 10.10 kg/cm^2) which, taking into account the construction experience on various types of rock, has given more than satisfactory results.

It has been remarked that to express the dynamic modulus in kg/cm^2 may cause some confusion in regard to the static modulus which is traditionally expressed in the same units. Far from being impossible, it is advisable to refer the elastic properties of the rock calculated by the seismic method, to the velocity of propagation of the elastic waves expressed in m/s. In fact, modulus of elasticity or velocity of propagation represent conventions to express in a single figure the elastic qualities of the material which interest the constructor: between two conventions it is better to refer to the most convenient.

5. 2) By means of the dynamic modulus it is also possible to observe easily the anisotropy of the foundation rock, and so take due account of this in the design of the dam, and also to have an idea of the decline of the elastic properties of the material; a decline due to that set of phenomena of settling repeatedly mentioned.

As an example, experiments carried out with the dynamic method on the rock which forms the right abutment of the Pieve di Cadore dam, in 1948 to 1949 before and during its construction, led to the finding of an average modulus of elasticity of 480 000 kg/cm^2. When the experiments were repeated in 1952 and 1953, three and four years respectively after the first filling of the reservoir, the average modulus was found to be 330 000 kg/cm^2, a decrease of 150 000 kg/cm^2 compared with the original figure. In 1953 other experiments of the same kind were repeated at the Lumiei dam, built in 1947. Here, while downstream from the dam the modulus was about 1 000 000 kg/cm^2, upstream the figure was about 800 000 kg/cm^2. In this case, too, there was a decline on elastic properties in the area affected by the waters, although somewhat less than that observed at Pieve di Cadore.

According to Prof. Caloi who has carried out this research the decline of the modulus may be caused by an increase in the porosity of the rock, determined by the stress due to the increased pressures and their more or less sharp variations.

Seismographic observations in fact enable to conclude that in the rock which forms the bed of a reservoir especially near the foundations, from the beginning of the work to its completion and the first filling of the reservoir, and to a lesser degree in subsequent fillings, a continuous settling has been in progress, determined by the disruption of the equilibrium previously existing between the forces in the medium. The upsetting of this equilibrium may manifest itself in true small earthquakes, at least in myriad of small strokes affecting the rock surrounding the dam. This action of shocks if continued for years, causes minute fractures in the rock and innumerable small lesions which tend to increase its porosity in a wide sense of the word. This minute, wearing action is manifested particularly in the whole of the rocks supporting the dam, especially in arch dams.

We note that this interpretation might be used to explain the diversity of elastic properties found in rocks of the same geological period and the same chemical composition: a diversity which would be due to the different stresses which the various rocks have undergone during the ages.

5. 3) By geophysical means (inclinographs, seismographs, vibrometers, etc.), aspects of the behaviour of the structure and foundations can be examined which would escape other kinds of investigation.

Thus with the combined use of seismographs and inclinographs it has been possible to establish the fact that the Pieve di Cadore dam belongs to a single geodetic block. At present the geophysicists have reached the conclusion that, especially in the areas subjected to earthquakes, the surface stratum of the earth crust is made up of relatively small blocks (horizontal dimensions of the order of 7 + 14 kilometers with analogous thicknesses) bounded by recent faults or by surfaces of fractures liable as a mass to mutual sliding without perceptible deformation. Now it is very important that a dam, especially an arch dam, should be based on a single geodetic block; research of the kind is therefore necessary above all in the preliminary stage of study and planning.

5. 4) Prof. Caloi has also observed that an important correlation exists between a preliminary abnormal clinographic activity and a subsequent abnormal macro and micro-seismic activity. This would indicate an intimate dependence between the gradual movements of strata of the rocks, under the influence of tensions acting in them, and the sudden destructions of equilibrium (earthquakes) which these tensions may ultimately provoke.

As an example, in the diagram of Fig. 14 it is shown, according to their components, the variations of rotation which, since the 22nd September 1954 have gradually started in the foundation rock of the Ambiesta dam (under construction). The rotation is first in a northerly, then in a decidedly westerly direction. The rotation in this direction was shown to be particularly active from October 3rd to 8th, a period in which the first very slight instrumental shocks occurred, gradually increasing in frequency, confirming the tension present in the areas in slow movement. Subsequently the rotations again took a northerly direction; corresponding to this change of rotation, on October 11th a seismic shock of the intensity of 10^{18} erg occurred, followed by an almost uninterrupted series of small instrumental shocks with the task of exhausting the residual tensions in the stratifications affected by the earthquake.

6) - Conclusions

6. 1) The observations and results given here are intended to show the

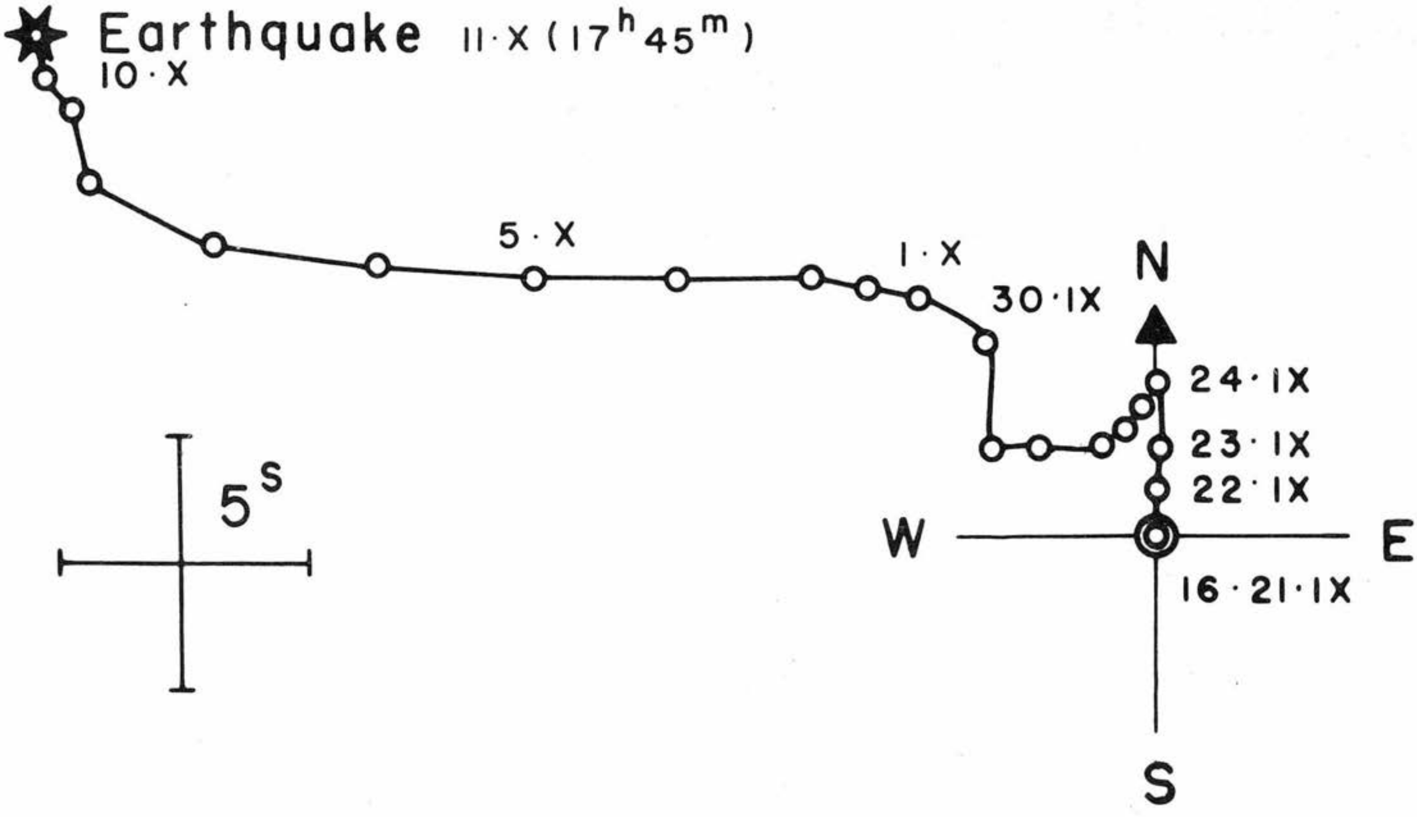

Fig. 14. Ambiesta dam: inclinographic activity observed near to the rock of foundation.

difficulty of arriving quickly at conclusions of general interest from the control measurements of dams. What can be ascertained for one structure can hardly, even qualitatively, be applied to another, even though apparently of the same type, built and situated in analogous conditions. The fact is that, although there may be dams of the same type, the technological features, the means, and the time taken for its construction, the elastic properties of the rock interfere in such a variety of ways as to give every dam its own individuality, which can only begin to be discovered after some years of accurate and continuous observation. The structure and the rock, however, form a single whole, influencing each other, and for this reason investigations on their behaviour must be extended to both these elements, which together react to the effect of the various acting forces. The need, among other things, for appropriate geophysical research which has not yet been given the scope and importance of structural research, is thus made clear.

Interpretation of measurements can be greatly assisted by experiments on models. On this point we have a significant example in the deflections of the crest arch of the Val Gallina dam, which could not entirely be interpreted, on a first examination, according to the normal behaviour of a structure of that type.

The laboratory I.S.M.E.S. of Bergamo then took charge of the construction of a model in which the elastic properties of the various types of rocks affected by the foundation of the dam were reproduced. The deformations recorded in the model proved to be in exact agreement with the experimental ones, which were thus clearly understood.

In conclusion, the control measurements of dams, not confined to simple "safety" measurements, are intended to fulfill the ambitious task of investigating the actual behaviour of a structure which is particularly complex, not only in itself, but more especially on account of the not always easily definable surrounding conditions. It is therefore only through a succession of numerous and uninterrupted observations, that we can hope to shed some light on this interesting subject, by differentiating the elementary causes of variation from the corresponding effects. For each structure and every period we must then consider the combination of the greatest possible number of elementary causes, in order to attempt to achieve an interpretation of the final resulting whole.

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POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: MEASUREMENTS AND STUDIES OF BEHAVIOR OF KAMISHIIBA DAM

H. Kimishima,[1] and C. C. Bonin,[2] M. ASCE
(Proc. Paper 1182)

FOREWORD

This paper is one of a group presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams (April, 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time, it is not known exactly how many papers will be printed from the Symposium. So far, fifteen papers have been approved: "Arch Dams: Their Philosophy," by Andre Coyne (Proc. Paper 959); "Arch Dams: Trial Load Studies for Hungry Horse Dam," by R. E. Glover and Merlin D. Copen (Proc. Paper 960); "Arch Dams: Portuguese Experience with Overflow Arch Dams," by A. C. Xerez (Proc. Paper 990); "Arch Dams: Theory, Methods, and Details of Joint Grouting," by A. Warren Simonds (Proc. Paper 991); "Arch Dams: Santa Giustina Single-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 992); "Arch Dams: Measurements and Studies on Santa Giustina Dam," by Claudio Marcello (Proc. Paper 993); "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 994); "Arch Dams: Isolato Double-Curvature Arch Dams," by Claudio Marcello (Proc. Paper 995); "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs," by Claudio Marcello (Proc. Paper 996); "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira (Proc. Paper 997); "Arch Dams: Development in Italy," by Carlo Semenza (Proc. Paper 1017); "Arch Dams: Design of the Kamishiiba Arch Dam," by C. C. Bonin and H. W. Stuber (Proc. Paper 1018); "Arch Dams: Observed Behavior of Several Italian Arch Dams," by Dino Tonini (Proc. Paper 1134); "Arch Dams: Measurements and Studies of Behavior of Kamishiiba Dam," by H. Kimishima and C. C. Bonin; and "Arch

Note: Discussion open until July 1, 1957. Paper 1182 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 1, February, 1957.

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Dams: Construction of the Kamishiiba Arch Dam," by K. M. Mathisen and C. C. Bonin (Proc. Paper 1183).

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

SYNOPSIS

The Kamishiiba Arch Dam, 113 meters high from lowest bedrock to crest of roadway, is a unique and precedent-setting structure for Japan. Since it was to be the first arch dam in Japan (in fact in the Far East) its design and construction was the subject of lengthy discussion and controversy among private and government engineers in Japan.

The Kamishiiba Arch Dam was analyzed by the trial load method; modifications to the standard way of applying this method were incorporated in the design. In accordance with standard practice, certain design assumptions were made in order to simplify the laborious computations. Actual conditions while similar, are different from those assumed. This results in actual behavior somewhat different from that computed.

For this reason, approximately 480 different types of Carlson meters were embedded in the concrete and rock foundation, to be utilized for investigation of the actual behavior of this thin arch structure. This paper describes the results of preliminary investigations, utilizing data obtained from these meters; the paper also discusses the determination of elastic properties, temperatures, deformations and stresses in the concrete and foundation rock.

Before the completion of joint grouting operations and before all concrete was placed, the Kamishiiba Dam was subjected to two unscheduled temporary water loadings. These resulted from typhoons that increased the flow of the river far beyond the diversion tunnel capacity. In each case, the reservoir rose very quickly behind the dam to nearly two-thirds of its design head. These events presented a very unusual opportunity to measure stresses and deflections in a structure of this type. This opportunity was not lost and readings were taken of all the embedded instruments at the time of these loadings. These readings made possible a number of the conclusions drawn in this paper.

Arrangement and Method of Measurement

For a structure as large as the Kamishiiba Dam the problem involved is how to arrange a necessarily limited number of meters in the proper locations. The basic principles adopted at Kamishiiba were as follows:

1. In order to obtain an overall picture of the behavior of the dam - even if there be a slight sacrifice in accuracy - meters must be distributed over the entire structure, since assumptions of homogeneity and geometrical symmetry were known not to be entirely true as far as the actual behavior of the dam was concerned.

2. The location of the meters must be closely correlated to the respective elements (arch and cantilever) utilized in the analysis by trial load method.

3. Strain meter readings must be checked to some degree, either by duplication or by stress meters.

4. In locations where high stresses are expected, or in locations of special interest, measurements must be concentrated to some extent in order to be certain of the accuracy.

Location of the meters as utilized at Kamishiiba is shown in Figures 1 and 2.

The Kamishiiba Dam, being a relatively thin structure, does not include an inspection gallery. For this reason it was necessary to bring every cable from the embedded meters to terminal boxes on the downstream face of the dam. Readings are made with portable test sets at specified intervals, from catwalks provided on the downstream face.

Preliminary Investigations

The standard tests on the various properties of cement, rock and concrete were made; these investigations are summarized in Table 1 and Figure 3. In the general area of the dam site a number of classifications of bedrock were found. These were all sampled and subjected to physical and chemical tests. They were designated from Class A to Class D, ranging from the most excellent to extremely weathered graywacke. Some of the results of these tests are indicated in Table 2. This table indicates that the rock itself is extremely hard; it should also be noted that there are many minute cracks and fissures in this hard foundation rock.

Typical concrete mixes, their strengths and instantaneous moduli of elasticity are shown in Tables 3 and 4. These test investigations were made prior to the actual construction.

Measurements of creep, instantaneous modulus of elasticity and coefficient of thermal expansion were made on copper-sealed 8-in. by 16-in. cylinders, utilizing Carlson strain meters and car springs. The results of these measurements are shown in Figures 4, 5 and 6. Poisson's ratios were measured on approximately 300 cylinders, 6 in. by 12 in., by attaching dial gage extensometers. Values obtained by these tests indicate little or no difference resulting from age or cement content; the average value of Poisson's ratio was 0.16. The thermal properties of the concrete to be utilized at the dam were measured by conventional methods and the results obtained are as follows:

1. Thermal conductivity equals 0.00480 cal/cm, sec$^{\circ}$C

2. Specific heat equals 0.23 cal/gr, $^{\circ}$C

3. Thermal diffusion equals 0.0088 cm^2/sec

Autogenous growth of the concrete was measured by embedding non-stress meters in the dam; growths were measured ranging from 30 x 10^{-6} to 70 x 10^{-6} in. per in. for the first 6 months after the concrete was placed.

Deformations of the Foundation Rock

Since the arch dam is an indeterminate structure, deformations of the foundation rock are one of the designers' greatest uncertainties. As indicated

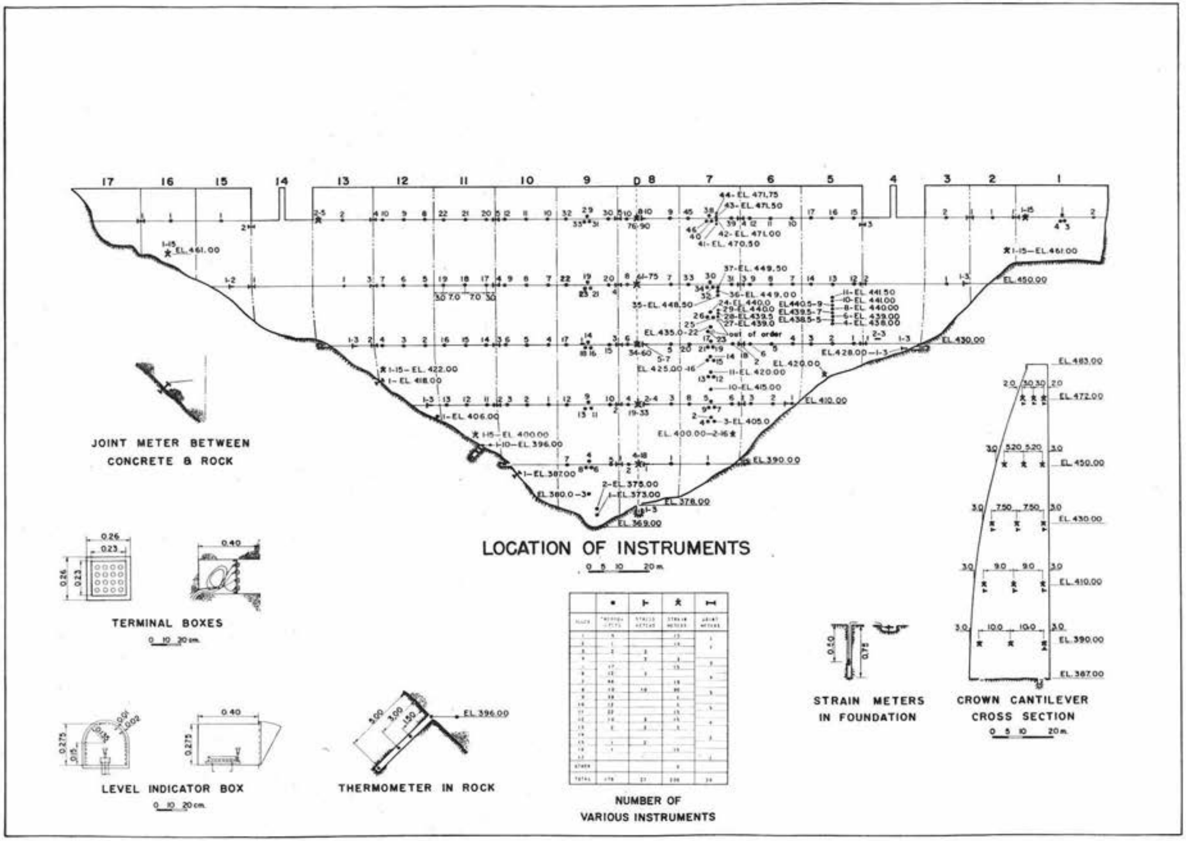

LOCATION OF INSTRUMENTS
0 5 10 20 m.
JOINT METER BETWEEN CONCRETE & ROCK
TERMINAL BOXES
0 10 20 cm.
LEVEL INDICATOR BOX
0 10 20 cm.
THERMOMETER IN ROCK
EL. 396.00
NUMBER OF VARIOUS INSTRUMENTS
STRAIN METERS IN FOUNDATION
CROWN CANTILEVER CROSS SECTION
0 5 10 20 m.
EL. 483.00
EL. 472.00
EL. 450.00
EL. 430.00
EL. 410.00
EL. 390.00
EL. 387.00
EL. 461.00
EL. 369.00
EL. 378.00

FIGURE 1

KIMISHIMA - BONIN

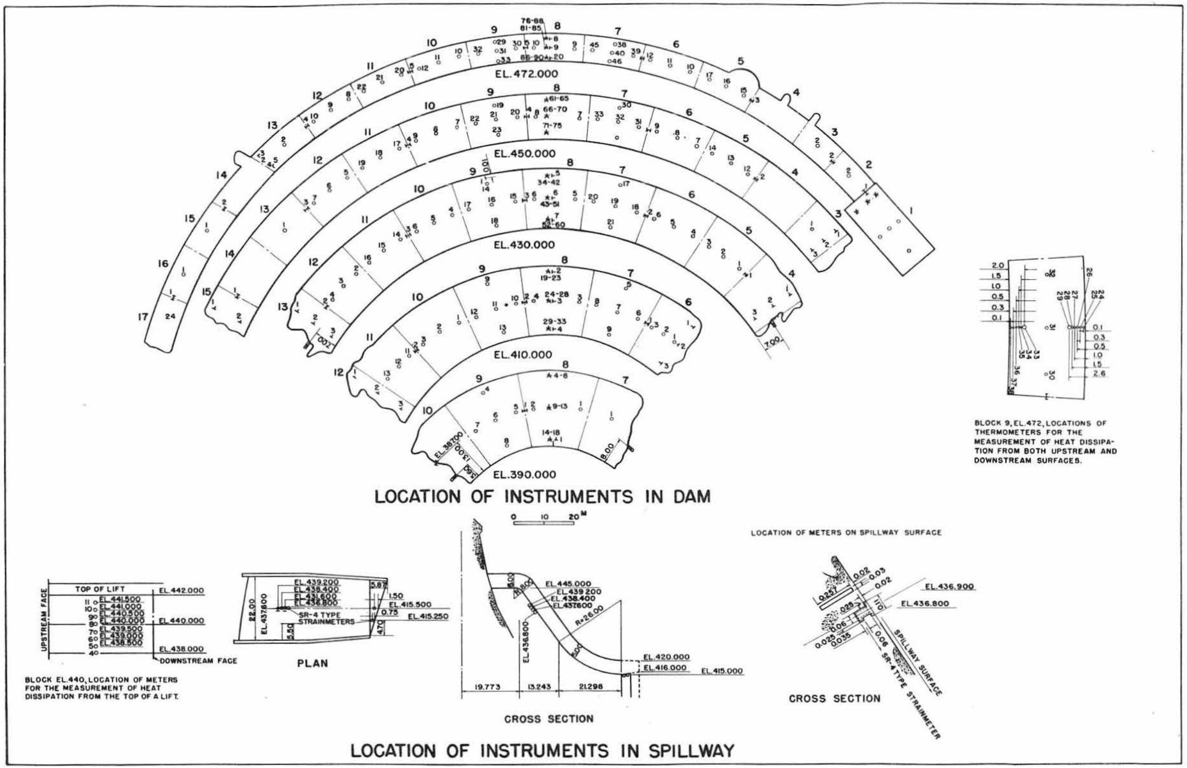


FIGURE 2

TABLE 1

CHARACTERISTICS OF CEMENT USED IN KAMISHIIBA DAM

Chemical Composition

	Specification %	Random Samples (%)
Loss on ignition	< 2.0	0.82
Insoluble residue	< 0.75	0.42
Silicon **dioxide** (SiO_2)	-	23.92
Aluminum oxide (Al_2O3)	-	4.22
Ferric oxide (Fe_2O_3)	-	4.10
Calcium oxide (CaO)	-	63.50
Magnesium oxide (MgO)	< 3.0	1.46
Sulfuric anhydride (SO_3)	< 2.0	1.26
Tri-calcium silicate ($3.CaO.SiO_2$)	<50.0	38.8
Di-calcium silicate ($2CaO.SiO_2$)	-	39.4
Tri-calcium aluminate ($3CaO.Al_2O_3$)	< 7.5	4.3
Heat of hydration- 7 days	<70 cal/g	57.8
28 days	<80 cal/g	71.4

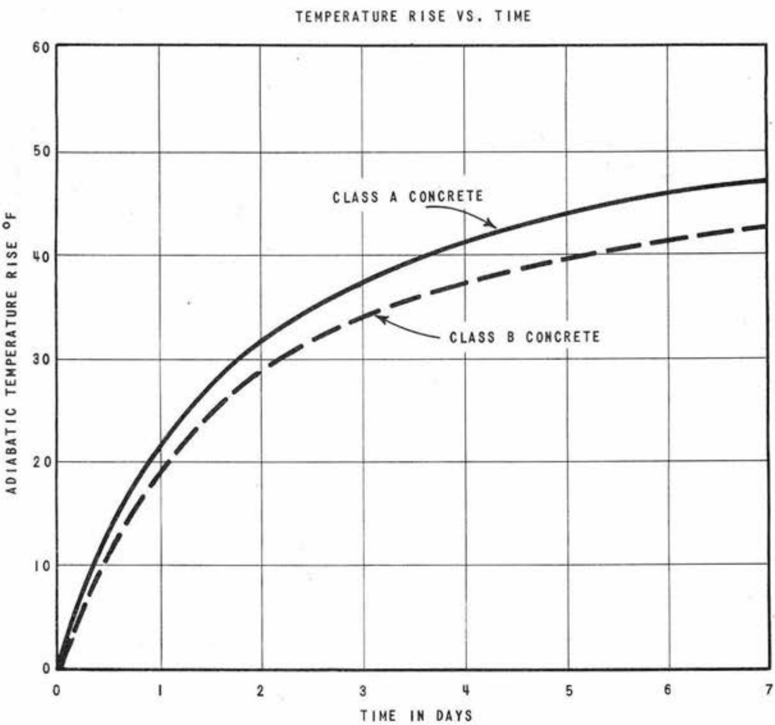
Physical Properties

		Specification	Random Samples
Specific gravity		J.I.S.	3.20
Fineness (cm^2/g)		> 1700	1830
Soundness (%)		< 0.5	0.04
Initial set (hr)		J.I.S.	2^h-14^m
Final set (hr)		J.I.S.	3^h-22^m
Compressive Strength	3 days (psi)	> 570	1230
	7 days (psi)	> 1290	2100
	28 days (psi)	> 3140	4200
	91 days (psi)	> 4600	6800
Bending Strength	3 days (psi)	J.I.S.	340
	7 days (psi)	J.I.S.	530
	28 days (psi)	J.I.S.	850
	91 days (psi)	J.I.S.	1100

J.I.S.: Japan Industrial Standards

FIGURE 3

ADIABATIC TEMPERATURE RISE OF THE SPECIFIED CONCRETE



PROPORTION OF THE CONCRETE

MIX / CLASSES	CEMENT CONTENT SACKS PER CUBIC YARD	W/C (%) BY WEIGHT	SAND POUNDS PER CUBIC YARD	COARSE AGGREGATE POUNDS PER CUBIC YARD
A	4-1/2	0.48	1070	2390
B	4	0.53	1090	2420

TABLE 2

ELASTIC PROPERTIES OF GRAYWACKE (2 TO 4 INCH CUBES)

Classes	Specific Gravity	Absorption %	Shore Hardness	Compressive Strength (psi)	Modulus of Elasticity (psi)
A (excellent)	2.72	0.04	82.5	38,000-27,000	11.4×10^6
B (partially weathered)	2.65	0.98	69.96	30,000-19,000	8.5×10^6
C (semi-weathered)	2.60	1.44	-	24,000-10,000	-
D (completely weathered)	2.53	3.69	-	17,000-2,400	-

TABLE 3

CONCRETE MIXES

	Concrete Classes		
	A	B	C
	(pounds per cubic yard)		
Cement	395	353	320
Water	204	202	210
Sand	870	875	970
1/4" - 3/4"	518	525	500
3/4" - 1-1/2"	518	525	500
1-1/2" - 3"	700	785	830
3" - 6"	700	785	680
W/C Ratio by weight (%)	51.5	57.1	65.8

Maximum size of aggregate: 6 inches
All aggregates were crushed sandstone.

CREEP UNDER A SUSTAINED LOAD
"B" MIX

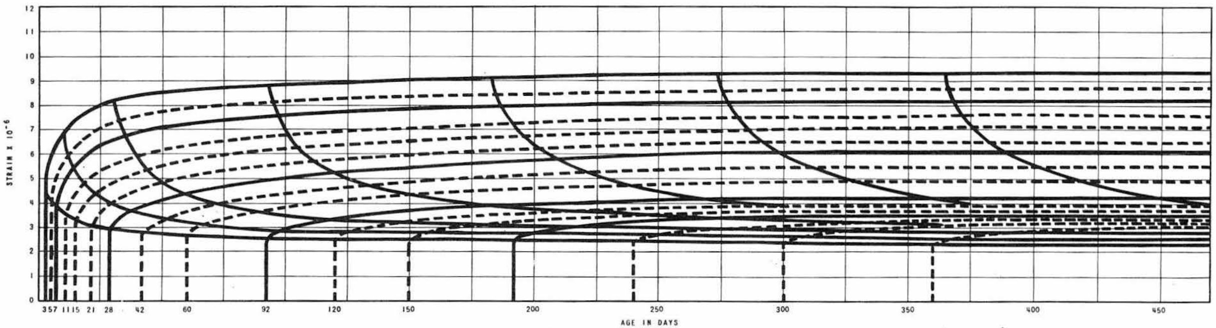


FIGURE 4

TABLE 4

STRENGTH AND INSTANTANEOUS MODULUS OF ELASTICITY OF CONCRETE USED IN KAMISHIIBA DAM

	Concrete Classes		
	A	B	C
	(pounds per square inch)		
7 days	2150	1800	1390
28 days	3950	3700	2800
91 days	5330	5250	4100
E_i @ 7 days	4.97×10^6	4.82×10^6	4.25×10^6
E_i @ 28 days	5.38×10^6	5.25×10^6	5.1×10^6

TABLE 5

EFFECTIVENESS OF ARTIFICIAL COOLING

	Maximum Temperature Rise	Age at the Maximum Temperature	Ratio of the Max. Temperature	Rate of Temperature Drop Max. = 100% Age In Days					Thickness
				20	30	50	60	70	
Pipe Cooling	$44^\circ F$	3 days	0.8	60%	40%	25%	12%	0%	54 feet
Natural Cooling	$55^\circ F$	30 days	1.0	97%	100%	93%	88%	78%	35 feet

INSTANTANEOUS MODULUS OF ELASTICITY VS. DAYS
(BY 12"x24" CYLINDERS)

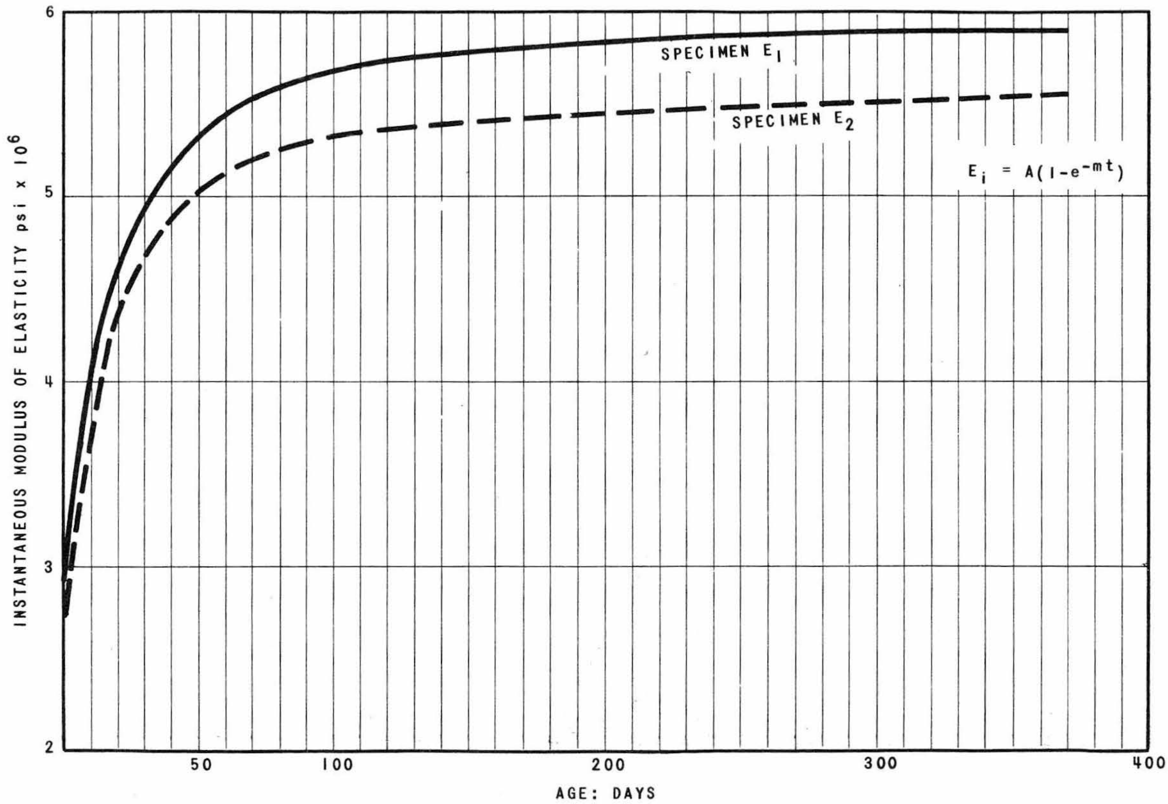
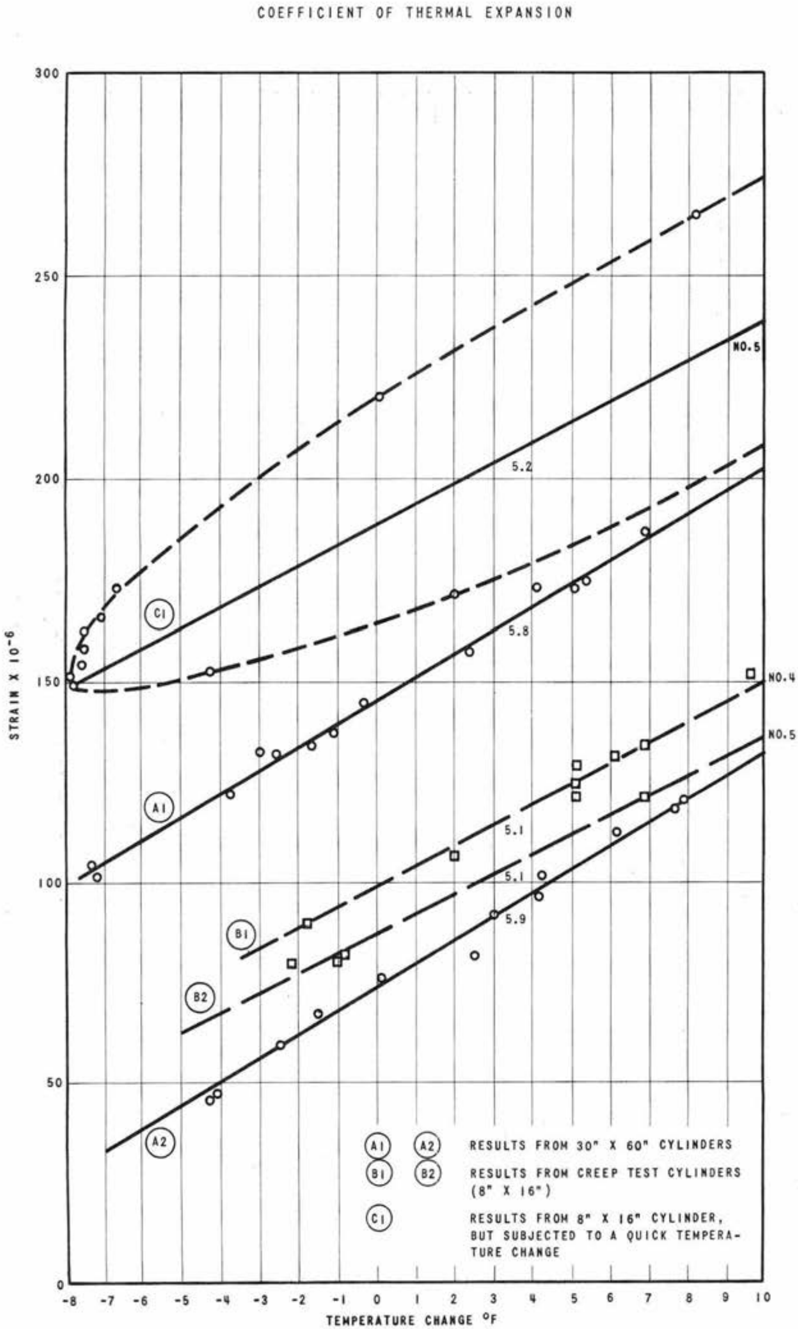


FIGURE 5.

FIGURE 6


COEFFICIENT OF THERMAL EXPANSION
STRAIN X 10⁻⁶
300
250
200
150
100
50
0
-8
-7
-6
-5
-4
-3
-2
-1
0
1
2
3
4
5
6
7
8
9
10
TEMPERATURE CHANGE °F
NO.5
5.2
C1
5.8
NO.4
NO.5
A1
5.1
5.1
5.9
B1
B2
A2
A1 A2 RESULTS FROM 30" X 60" CYLINDERS
B1 B2 RESULTS FROM CREEP TEST CYLINDERS (8" X 16")
C1 RESULTS FROM 8" X 16" CYLINDER, BUT SUBJECTED TO A QUICK TEMPERATURE CHANGE

in the paper outlining the design of Kamishiiba Dam, the effects of various assumptions concerning these deformations were studied. At the same time it was considered necessary to determine, to the greatest possible degree of accuracy, the actual deformation characteristics of the rock at the site. Measurements were made both by mechanical jack and by the use of embedded strain meters; deformations were determined and moduli of elasticity of the foundation as a whole were computed by use of Boussinesque's formula. Results varied widely, depending upon the direction and location of the test load and upon the slope of the curve drawn through the test points showing deflection against load. The moduli ranged from 0.18×10^6 psi to 8.5×10^6 psi. Figure 7 shows the deformation of the foundation at one location as determined by the jack-loading method. This wide variation in the modulus of elasticity of the foundation rock is one of the important factors causing the actual stress to vary from those calculated.

Actual Opening at Contraction Joints and at Contact Surface Between Concrete and Rock

The extensive system of joint meters made possible the continual readings of the joint openings and the openings at the contact surface between concrete and rock; the variations in these openings are illustrated in Figures 8, 9, 10 and 11. The behavior of the joints are self-evident from examination of these figures; the variations are fairly consistent with concrete temperatures except as modified by other factors, such as grouting operations and unexpected water load placed upon the dam. In general, and with respect to the particular joint openings pictured on the attached figures, the normal joint grouting pressure acting upon the joints opened them nearly 0.012 in., while the temporary water loading closed them approximately 0.004 in. In both instances there was an elastic movement, with the structure soon recovering to its initial state leaving only slight permanent effects.

However, at the joint between the concrete and rock it appears that the contact grouting had a considerably greater effect on the opening than was the case at a contraction joint; in addition, it was noted that at these joints some permanent movement was retained. On the other hand, the movement due to the temporary water load appeared to be of a permanent nature at these contact joints even in those areas where the load was applied after contact grouting had been accomplished. Generally speaking, the deflections of the arch dam were sensitive to all external pressures; this results in the obvious conclusion that extremely careful control must be maintained during all joint grouting operations.

Temperature Observations in Concrete and Foundation Rock

Resistance thermometers as well as other Carlson meters were installed to determine temperature distributions throughout the dam structure and in the foundation rock. The question of temperatures in the structure was one of great concern during the course of the construction, since the dam was built utilizing 2 meter lifts having a plan area for the largest block of nearly 680 square meters. The continuous measurement and analysis of the temperatures were necessary to determine the effectiveness of the pipe cooling, to make decisions concerning the rate of concrete placement, and to decide

FIGURE 7

DEFORMATION OF FOUNDATION ROCK

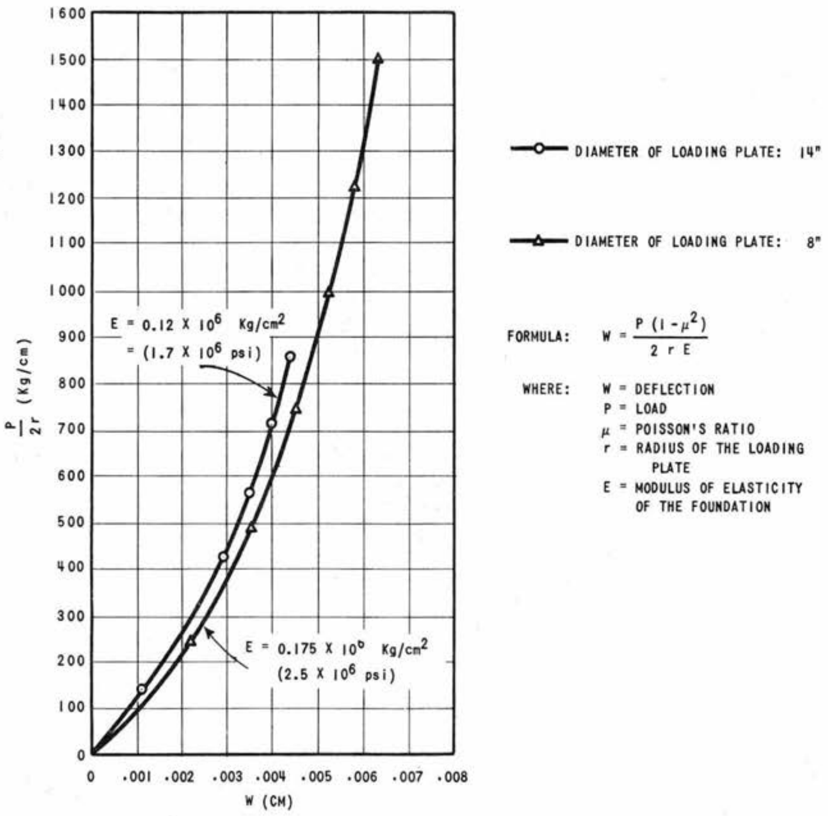
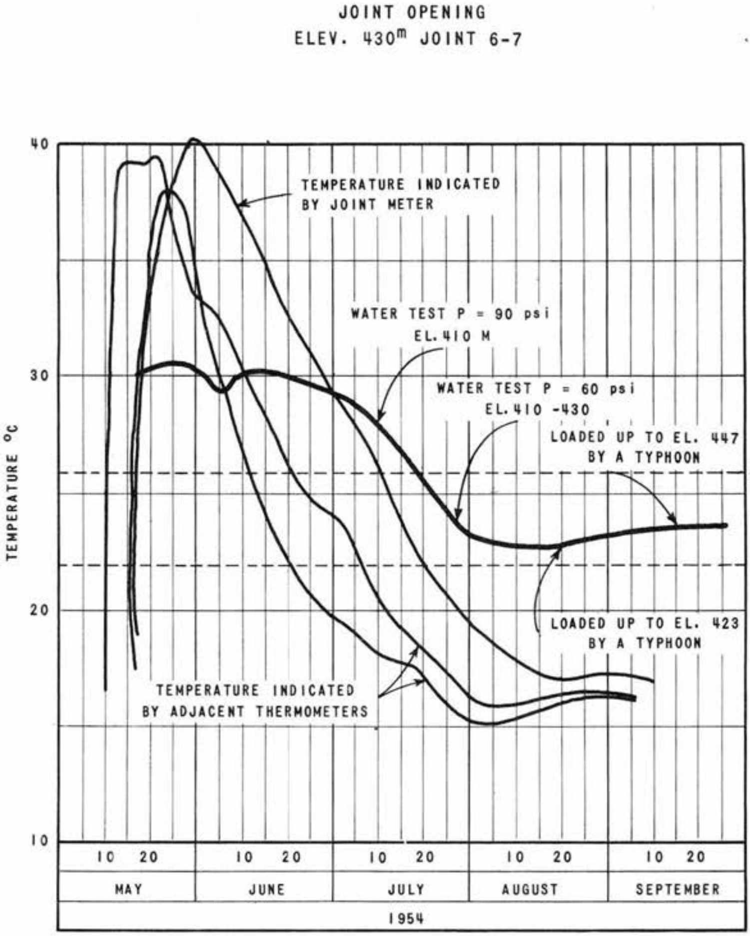


FIGURE 8


JOINT OPENING
ELEV. 430m JOINT 6-7
40
30
20
10
TEMPERATURE °C
JOINT OPENING IN MM
0.0
+1.0
+2.0
TEMPERATURE INDICATED BY JOINT METER
WATER TEST P = 90 psi EL. 410 M
WATER TEST P = 60 psi EL. 410 -430
LOADED UP TO EL. 447 BY A TYPHOON
LOADED UP TO EL. 423 BY A TYPHOON
TEMPERATURE INDICATED BY ADJACENT THERMOMETERS
10 20 10 20 10 20 10 20 10 20
MAY JUNE JULY AUGUST SEPTEMBER
1954

JOINT METER READING
ELEV. 410^m JOINT 8-9

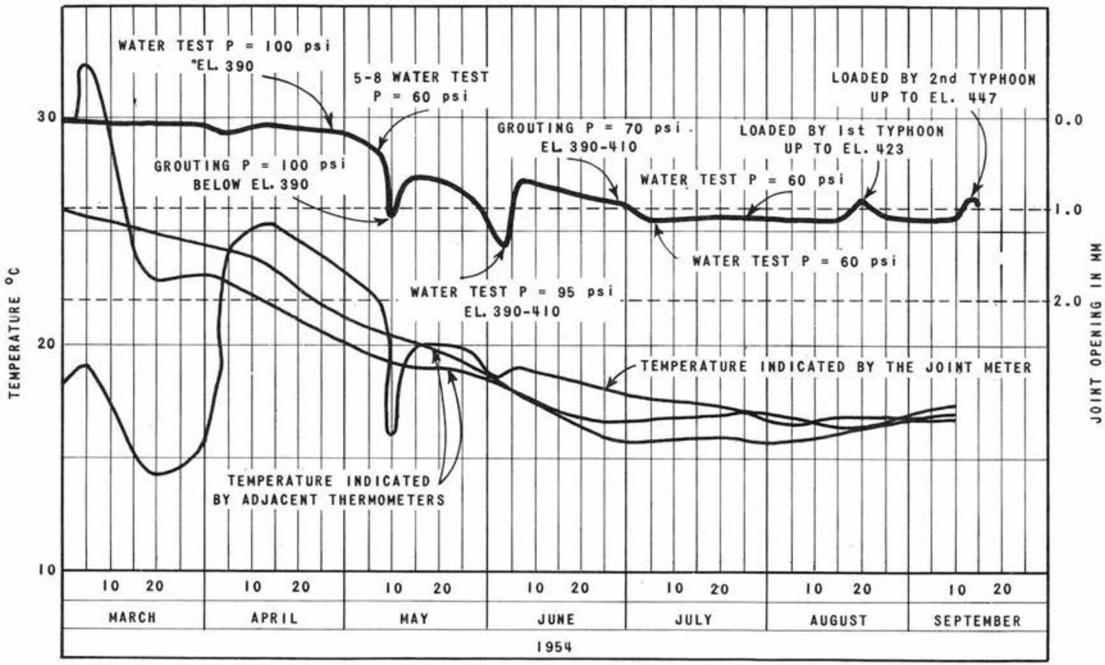


FIGURE 9

JOINT OPENING BETWEEN CONCRETE AND SLOPED FOUNDATION ROCK
BLOCK 10 ELEV. 387.00^{m}

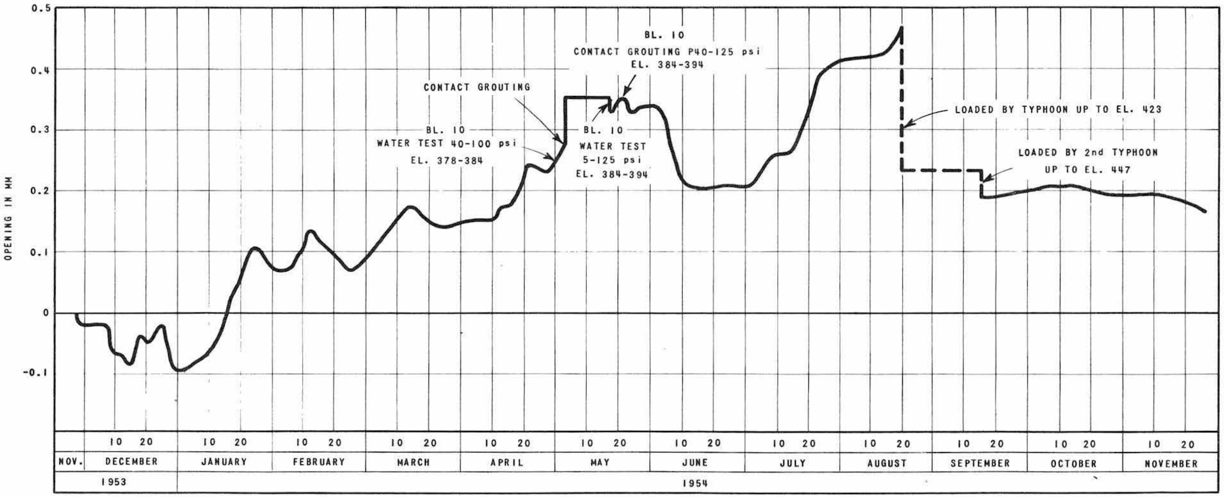


FIGURE 10

JOINT OPENING BETWEEN CONCRETE AND VERTICAL FOUNDATION ROCK
BLOCK 12 ELEV. 418.85m

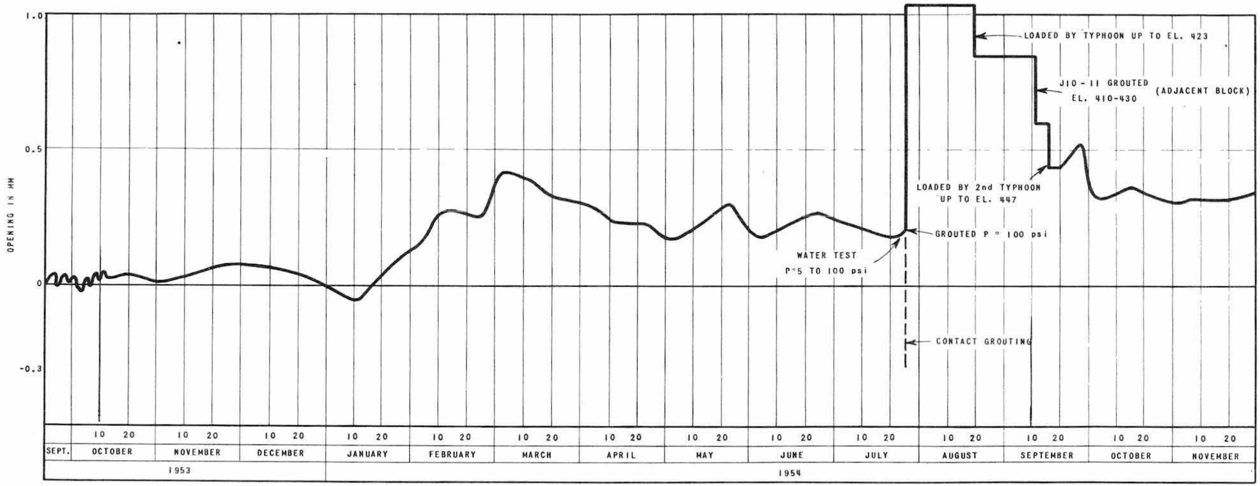


FIGURE 11

upon the joint grouting schedule. Figures 12 and 13 show temperature distributions in the concrete and the foundation rock at various ages. The effect of a facing concrete with higher cement content than the interior concrete can be seen from the steep gradient shown on Figure 12. Figure 14 shows the daily and hourly variations of temperature near both faces of the dam; the effect of solar radiation on the concrete at varying distances from the face can be seen in this figure. Figure 15 and Table 5 illustrate the marked effect of the pipe cooling adopted in the different sections of the dam; temperature gradient was greatly reduced by the use of artificial cooling.

Measurement of Deflection

In order to measure the deflection of the dam both during construction and after filling of the reservoir, accurate level indicators were installed on the downstream face of the dam at various locations. The accuracy of these indicators is such that a minimum reading of 0.00005 radians can be made. With the indicators installed at every 20 meters in height, a deflection of one millimeter can be read.

Figure 16 shows deflections at the crown section. Unexpected typhoon loadings (which occurred during the joint grouting operations of the lifts between Elevation 410 and 430 meters) caused a large deflection downstream at Elevation 430; however, this deflection was almost entirely recovered by the subsequent joint grouting operations which actually left a counter-deflection below Elevation 410 meters. During the period November 1954 to April 1955 the effects of joint grouting and temperature shrinkage tended to compensate each other - resulting in a slight downstream deflection at the end of this period, except for a slight upstream deflection at Elevation 410.

After the normal reservoir filling began, the deflection of the crown cantilever increased as the reservoir rose to its normal maximum level in October 1955; thereafter, the deflections continued to increase even though the reservoir level was lowering. It appears from these latter readings that the temperature effect on the deflection counterbalanced the water load effect. Figure 16 also shows that actual deflections are less than were computed. Although not illustrated, there was little or no variation in joint openings due to variations in water load.

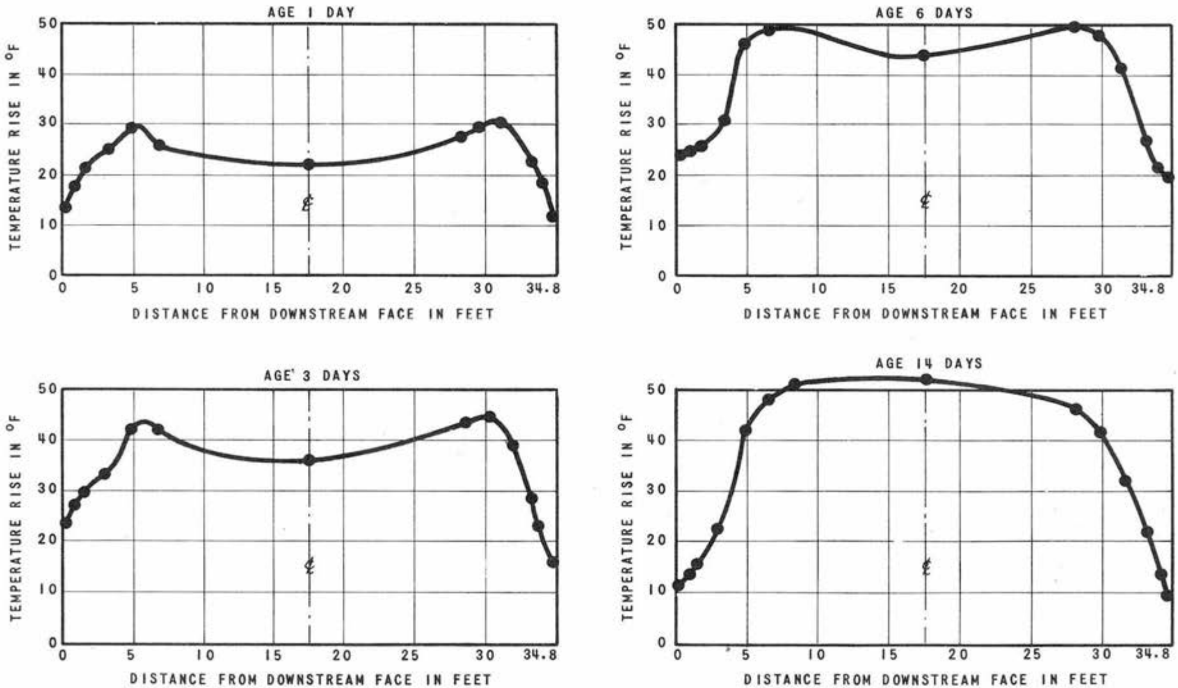
Arch Stress in the Dam

By utilizing both stress and strain meter readings it was possible to determine the actual arch stresses at various locations. Figures 17, 18 and 19 illustrate respectively:

1. elevations of the reservoir water surface during the course of the loading of the dam,
2. arch stresses at various locations and various times, and
3. computed arch stresses by the trial load analysis.

Analyses of these three figures, and the calculations involved in making them, show that in a dam of the proportions of Kamishiiba, the initial stress existing before the water loading occurs is a relatively large portion of the total stress in the dam after the water loading. These figures also show that the

TEMPERATURE DISTRIBUTION ACROSS A SECTION AT VARIOUS AGES
BLOCK 9, THICKNESS 34.8 FEET, NATURAL COOLING



TEMPERATURE DISTRIBUTION ACROSS A SECTION AT VARIOUS AGES
BLOCK 9, THICKNESS 34.8 FEET, NATURAL COOLING
(CONTINUED)

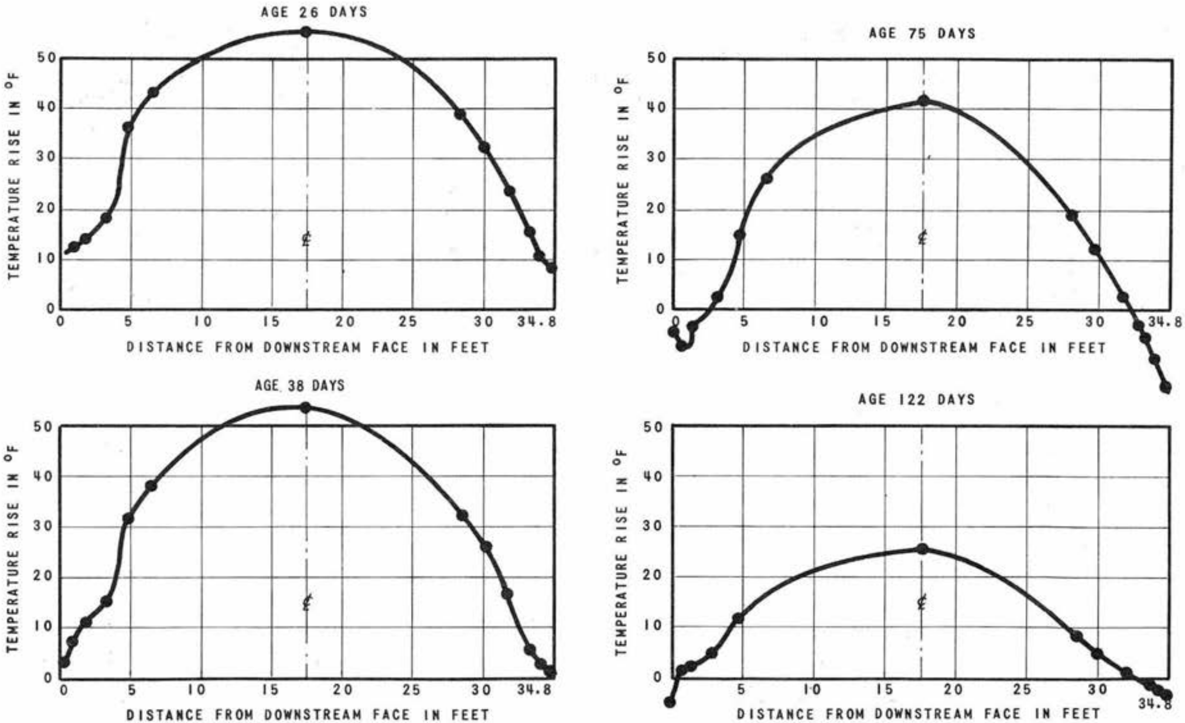
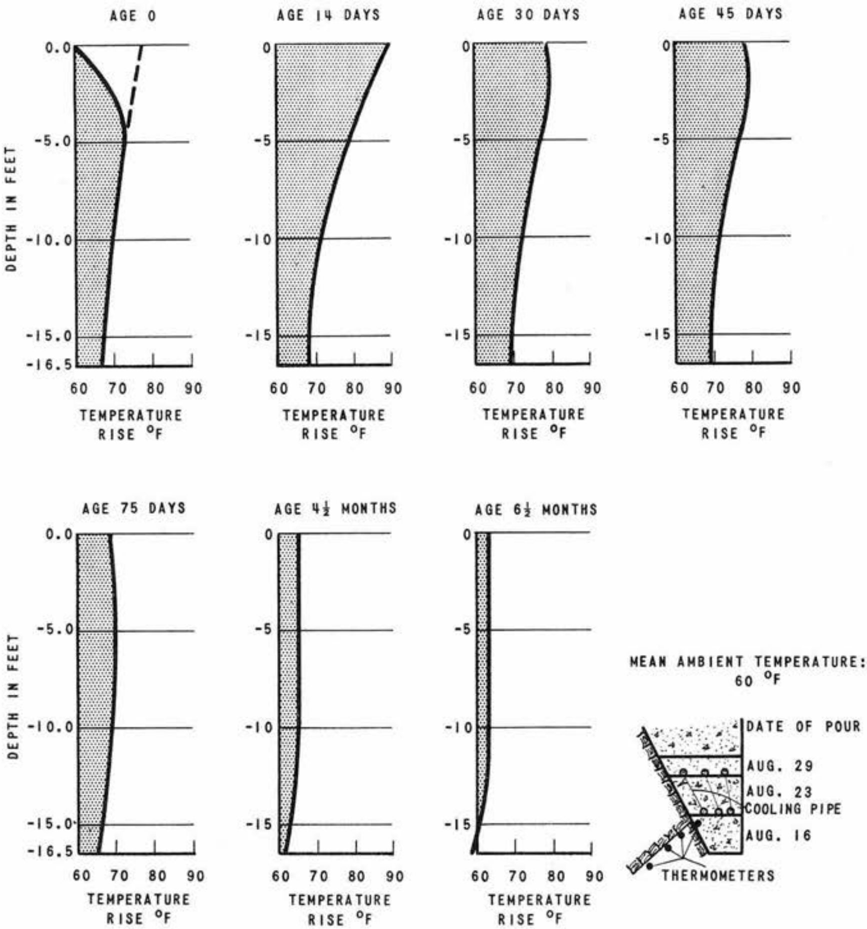
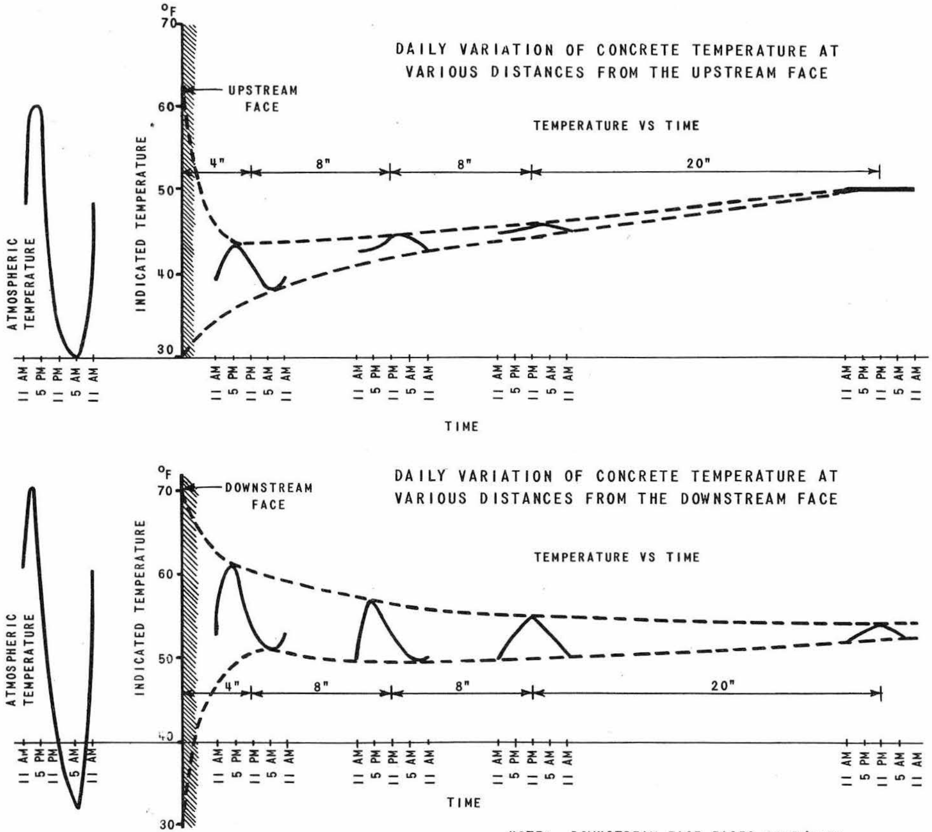


FIGURE 12
2 OF 2

FIGURE 13

TEMPERATURE DISTRIBUTION UNDER GROUND SURFACE
AT VARIOUS PERIODS AFTER CONCRETE PLACEMENT

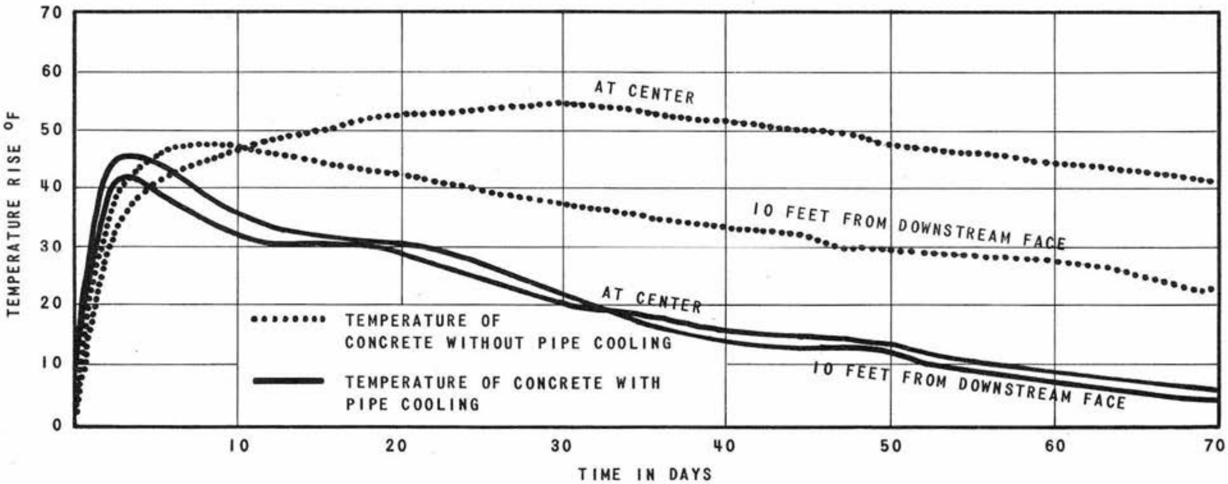




NOTE: DOWNSTREAM FACE FACES SOUTHEAST.

FIGURE 14

TEMPERATURE VARIATION BY NATURAL COOLING COMPARED
WITH PIPE COOLING



February, 1957

FIGURE 15

DEFLECTION OF CROWN CANTILEVER DURING CONSTRUCTION

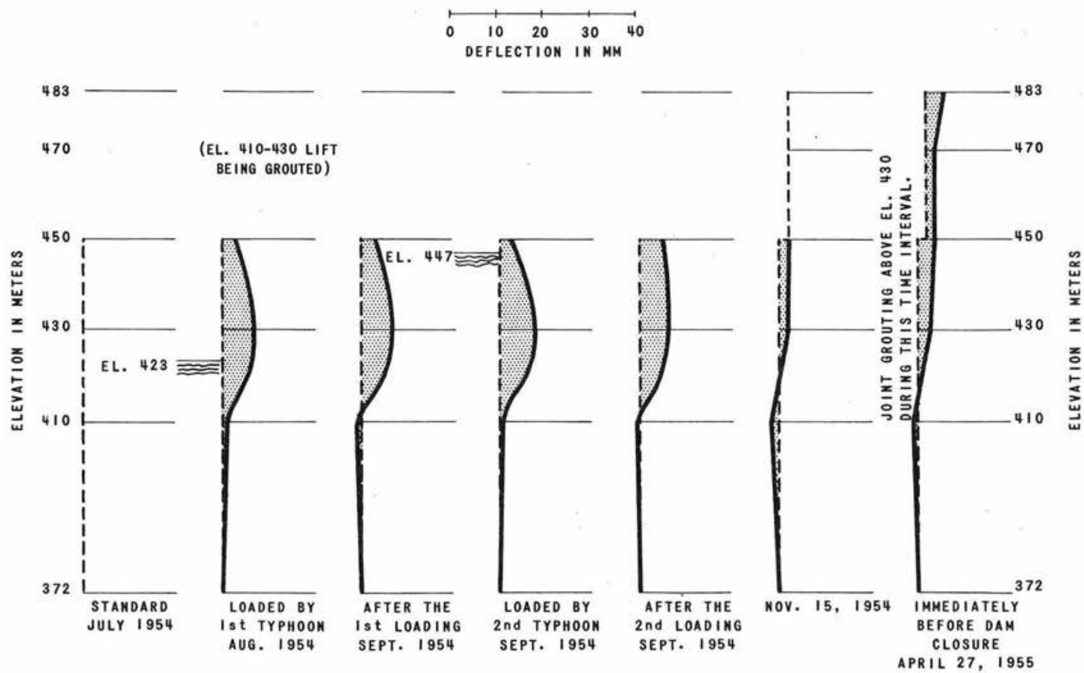


FIGURE 16
1 OF 2

DEFLECTION OF CROWN CANTILEVER AFTER DAM CLOSURE

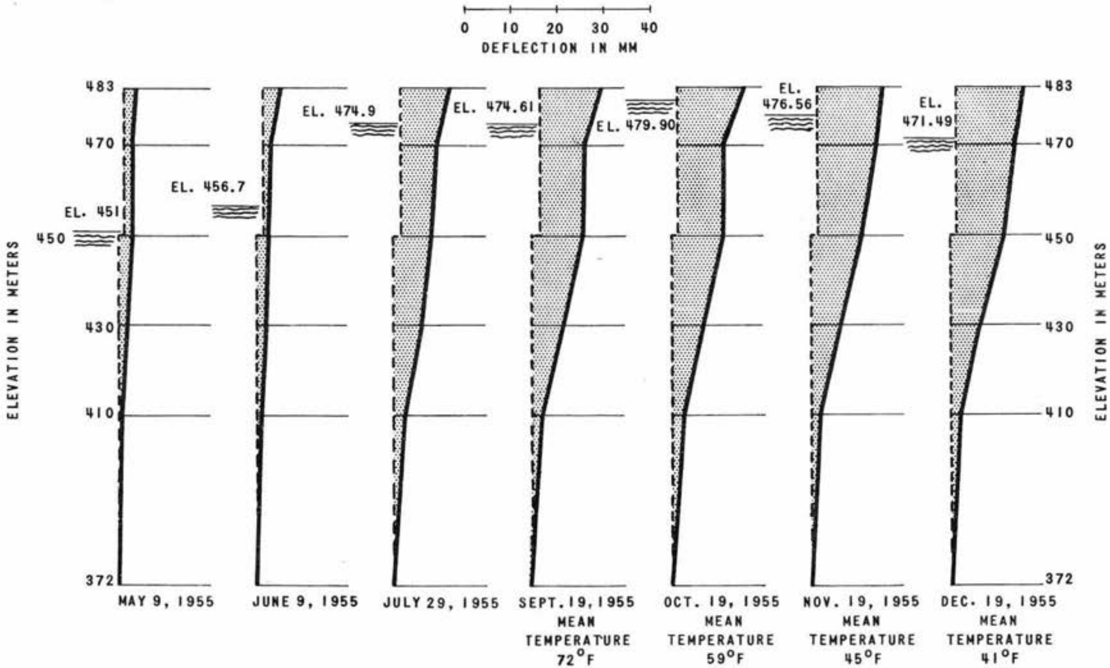
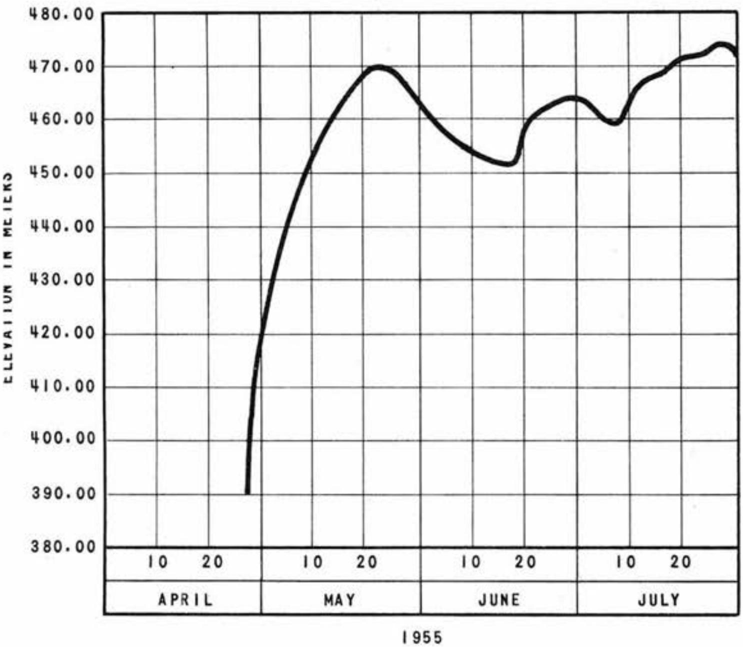


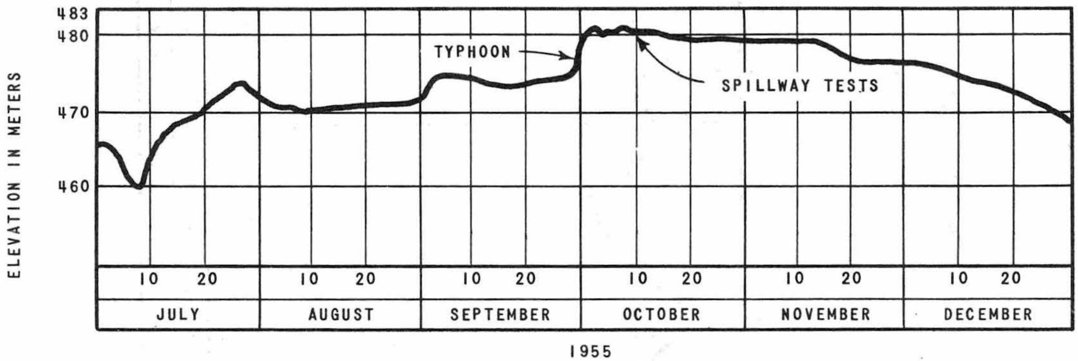
FIGURE 16
2 OF 2

FIGURE 17
1 OF 2

ELEVATIONS OF WATER SURFACE AFTER
THE LOADING STARTED



RESERVOIR ELEVATION



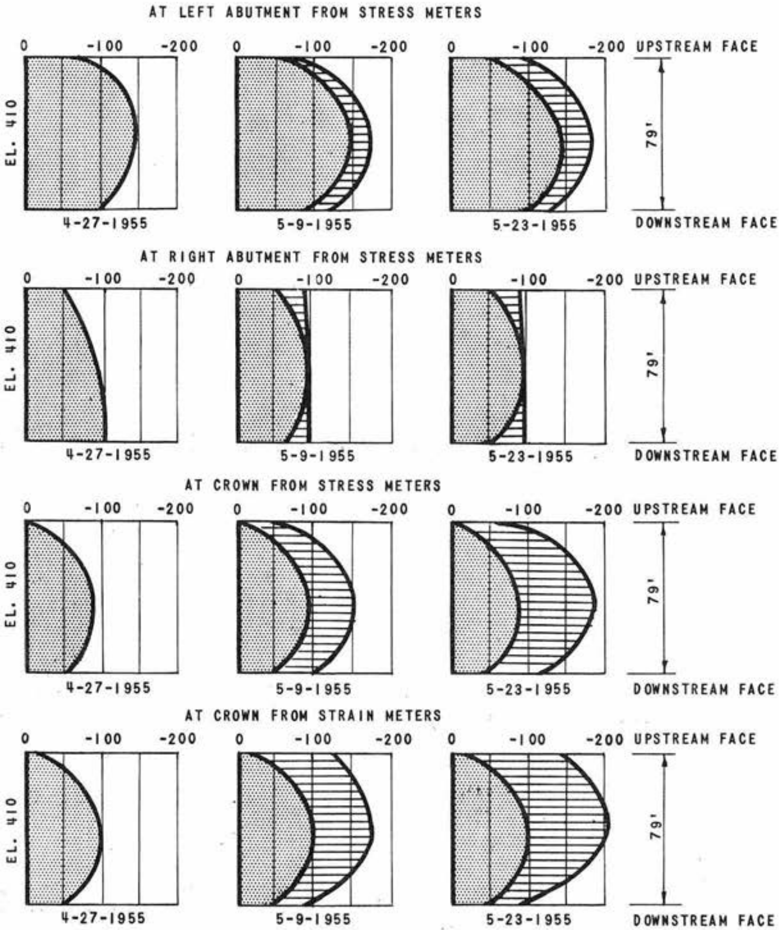
PO 1

FIGURE 17
2 OF 2

February, 1957

FIGURE 18
1 OF 6

ARCH STRESSES AT VARIOUS POINTS



NOTES:

FIGURES SHOW STRESSES IN psi

\+ INDICATES TENSILE STRESSES, - INDICATES COMPRESSIVE STRESSES

BLACK SHADOWS INDICATE INITIAL STRESSES EXISTED BEFORE CLOSURE OF DAM

HATCHED LINES INDICATE STRESSES DUE TO EXTERNAL LOADS AFTER CLOSURE

ARCH STRESSES AT VARIOUS POINTS

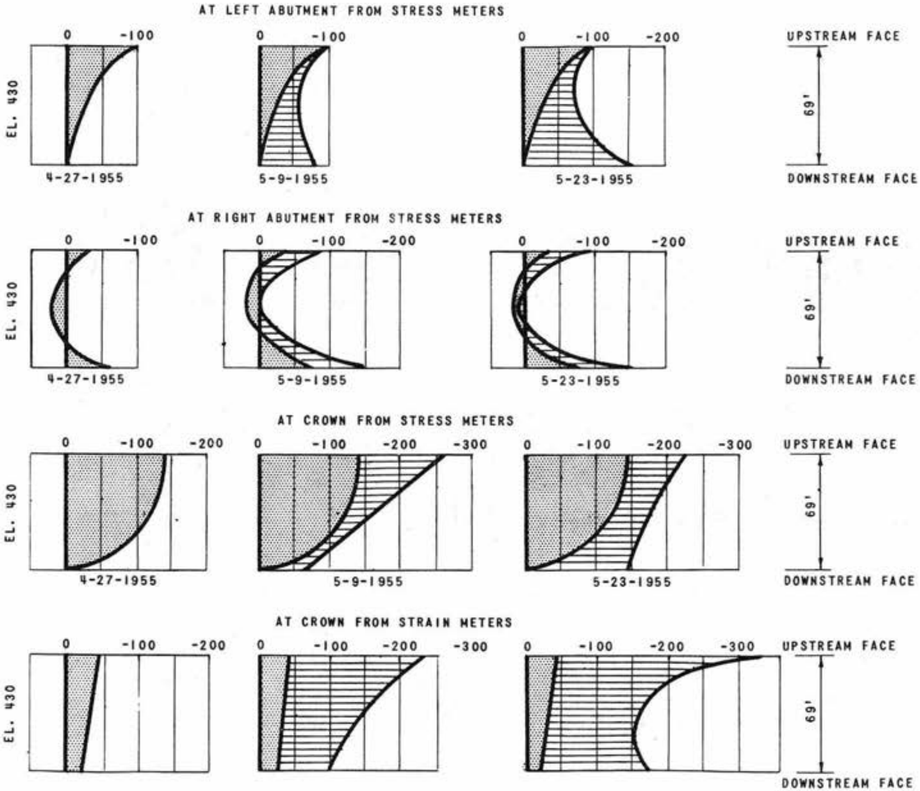


FIGURE 18
2 OF 6

ARCH STRESSES AT VARIOUS POINTS

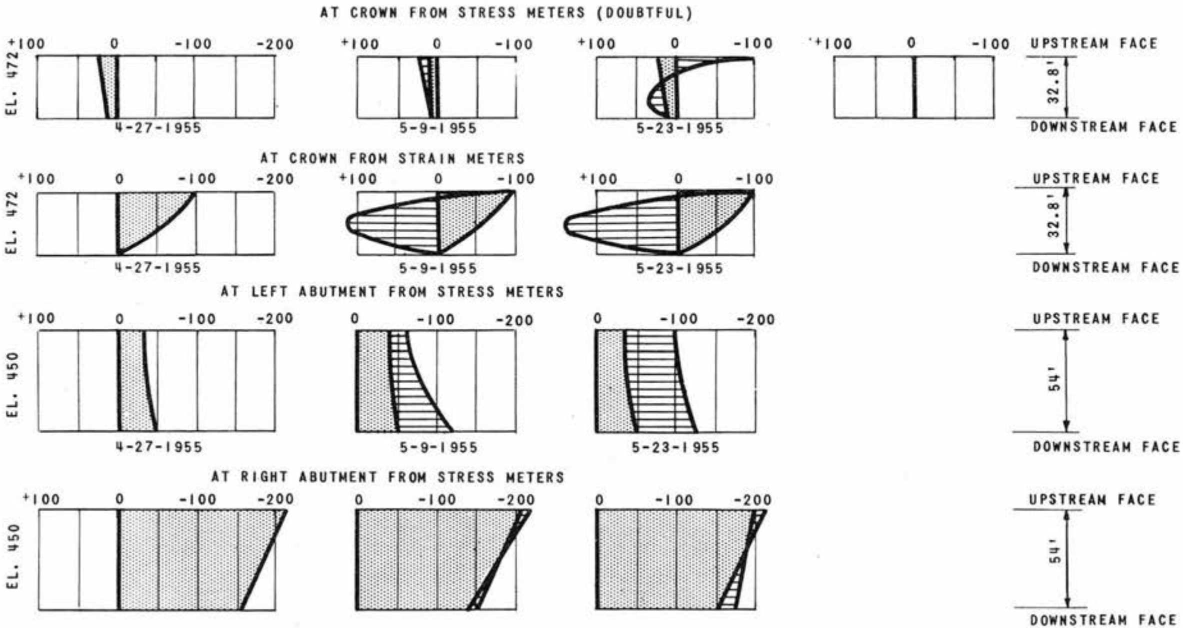
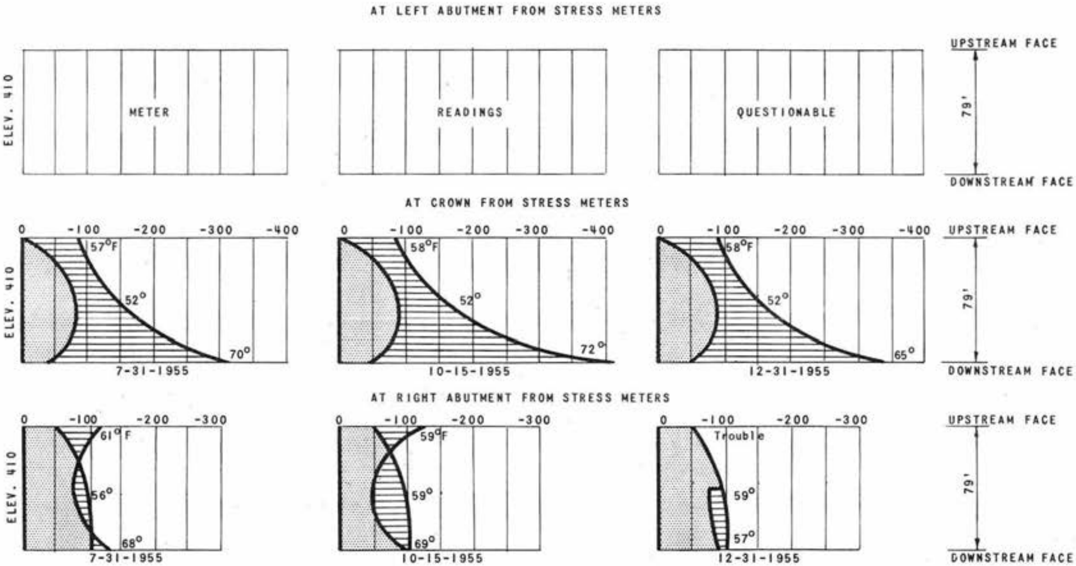


FIGURE 18
3 OF 6

ARCH STRESSES AND TEMPERATURES AT VARIOUS POINTS



NOTES:

FIGURES SHOW STRESSES IN psi

+ INDICATES TENSILE STRESSES, - INDICATES COMPRESSIVE STRESSES

BLACK SHADOWS INDICATE INITIAL STRESSES EXISTED BEFORE CLOSURE OF DAM

HATCHED LINES INDICATE STRESSES DUE TO EXTERNAL LOADS AFTER CLOSURE

TEMPERATURES WERE MEASURED ON INDICATED DATE

TENSILE STRESSES ARE QUESTIONABLE BECAUSE OF STRESS METERS CHARACTERISTICS

FIGURE 18
4 OF 6

ARCH STRESSES AND TEMPERATURES AT VARIOUS POINTS

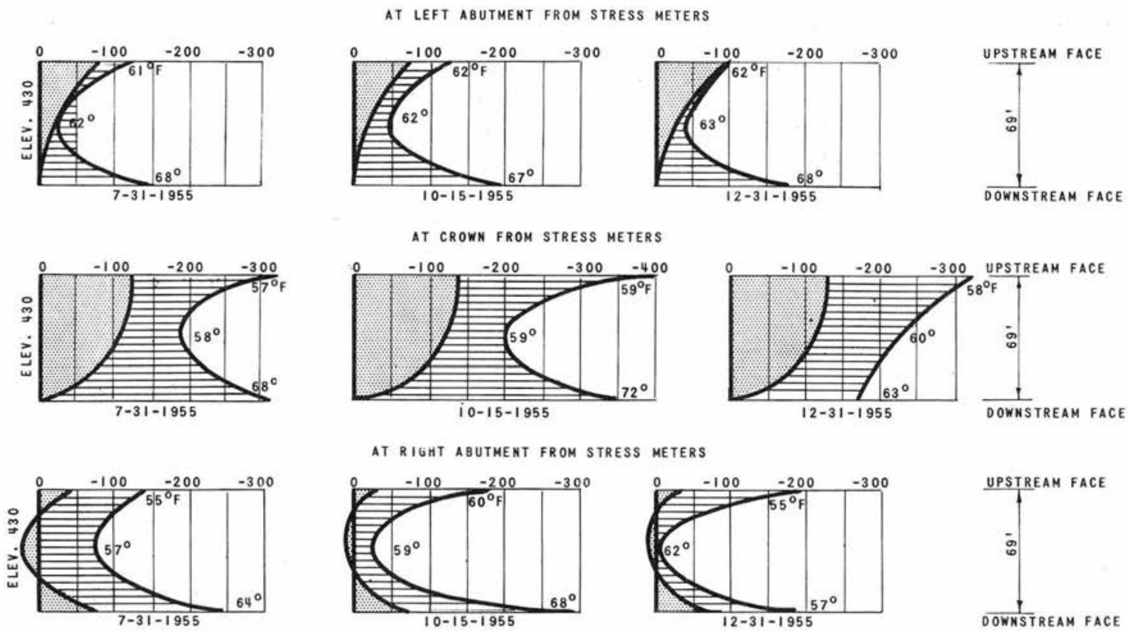


FIGURE 18
5 OF 6

ARCH STRESSES AND TEMPERATURES AT VARIOUS POINTS

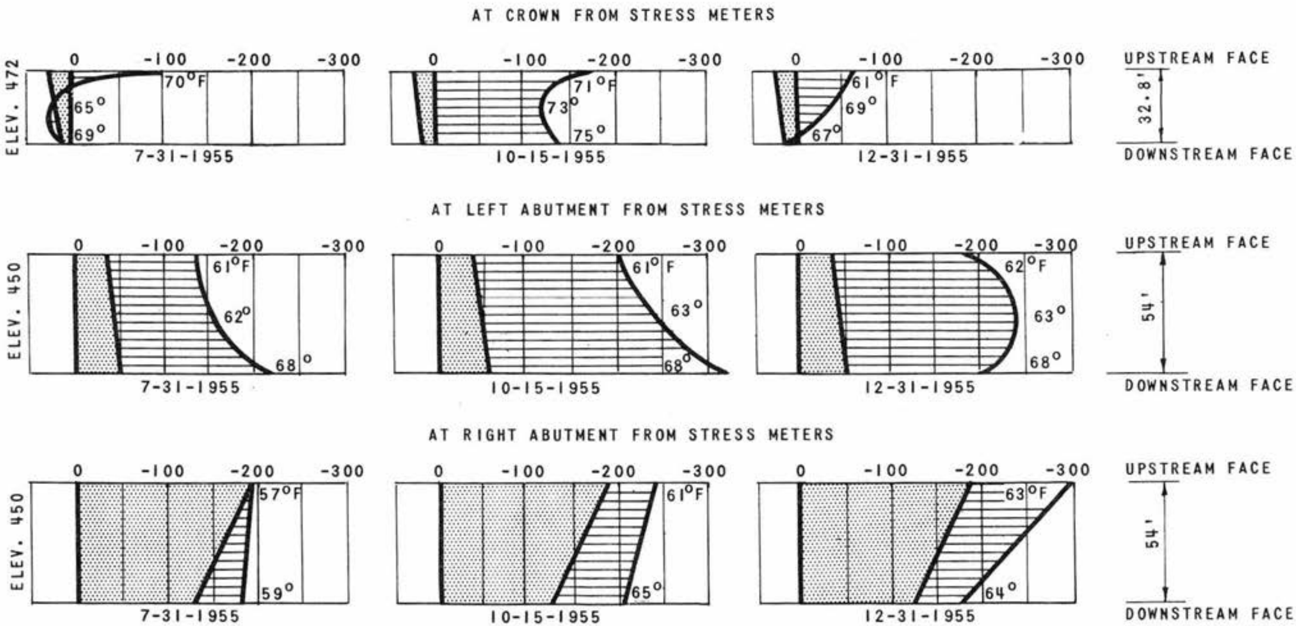
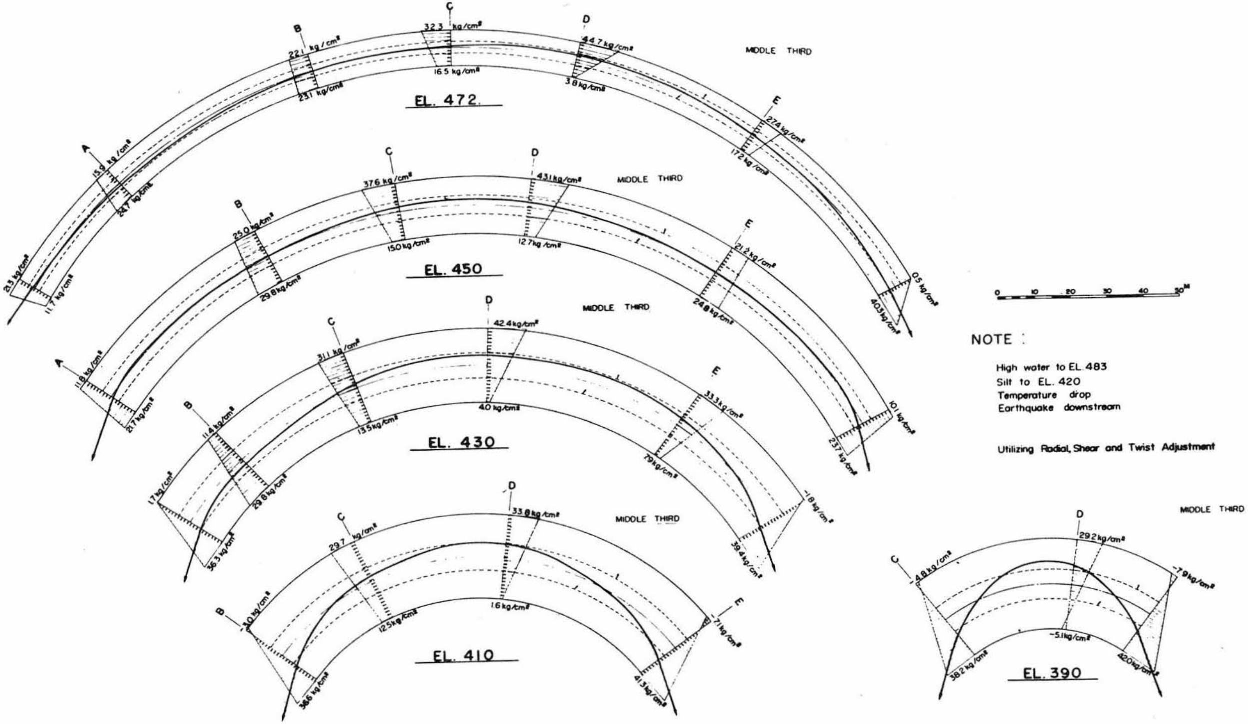


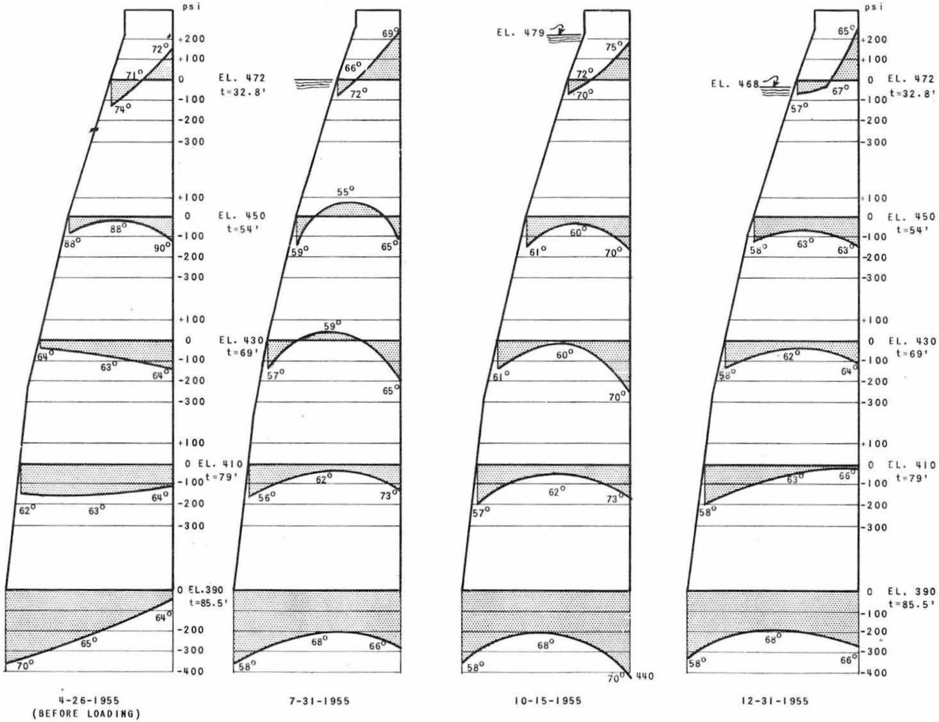
FIGURE 18
6 OF 6



KIMISHIMA - BONIN

FIGURE 19

STRESS DISTRIBUTION IN THE CROWN CANTILEVER



NOTE: - DESIGNATES COMPRESSION
TEMPERATURE SHOWN IN °F

FIGURE 20

stress distribution is non-linear and is not always consistent with the calculated stresses; tensile stresses calculated by any one of the various methods of computation do not appear in the actual structure in either the crown or the abutment sections. Most important, the figures indicate that the total stresses are less than the calculations indicate. It can be seen from the more recent data that temperature effects exert a greater influence on the arch dam than the effects of water load; and, in addition, that the crown section stresses are greater than the abutment stresses - although the opposite was expected from the calculations. It should be noted that the loading conditions have not completely reached the maximum assumed in the design calculations, and the period of loading has been relatively short.

Figure 20 is included to show the cantilever stress distributions in the crown cantilever. Again it can be seen that the general tendency of the stress distribution is non-linear. It should be mentioned that these forms of stress distributions are frequently seen in mass concrete where temperature distribution is non-linear. The reversed stress distribution at the base of the crown cantilever before loading the dam is believed to be caused by the contraction joint grouting.

A number of strain meters were installed in the right abutment at Elevation 461 for the purpose of determining radial, cantilever, and arch stresses in this area. Figure 21 indicates the variation of stress in this part of the dam. In general, the stresses at Elevation 461 in the cantilever at the right abutment appear similar to the case where a concentrated force has been applied on a semi-infinite elastic body of certain modulus of elasticity.

Stresses Due to Restraint

Each lift of concrete in a structure of this nature receives restraint from the lift immediately below, if their moduli of elasticity are different. A number of meters were embedded in the dam to measure the effects of the restraint of one lift upon another. Figure 22 indicates stress variations immediately following the placing of a lift; this variation is an example where the normal 3-to 4-day time lag existed between the placing of successive lifts. Figure 22 also shows stress variations where the lift below had been placed 50 days before the lift above was placed. (The former were determined by use of stress meters and the latter by strain meters.) These stresses were computed by the use of the several elastic constants previously mentioned. At the location of each of these examples pipe cooling was used. As can be seen on Figure 22 tensile stress is developed approximately 45 to 60 days after placement of the lift.

CONCLUSIONS

Any attempt to draw definite conclusions from either the involved trial load calculations of an arch dam or from the readings of stress and strain meters, at best, depends upon individual judgment. Nevertheless, the authors feel that the following conclusions are warranted from the data presently at hand:

1. An arch dam, properly designed, has an actual factor of safety, as far as stresses are concerned, considerably in excess of the factor of safety indicated by the stress calculations.

STRESS DISTRIBUTION AT THE RIGHT ABUTMENT
BLOCK 16 ELEVATION 461, (MEASURED BY STRAIN METERS)

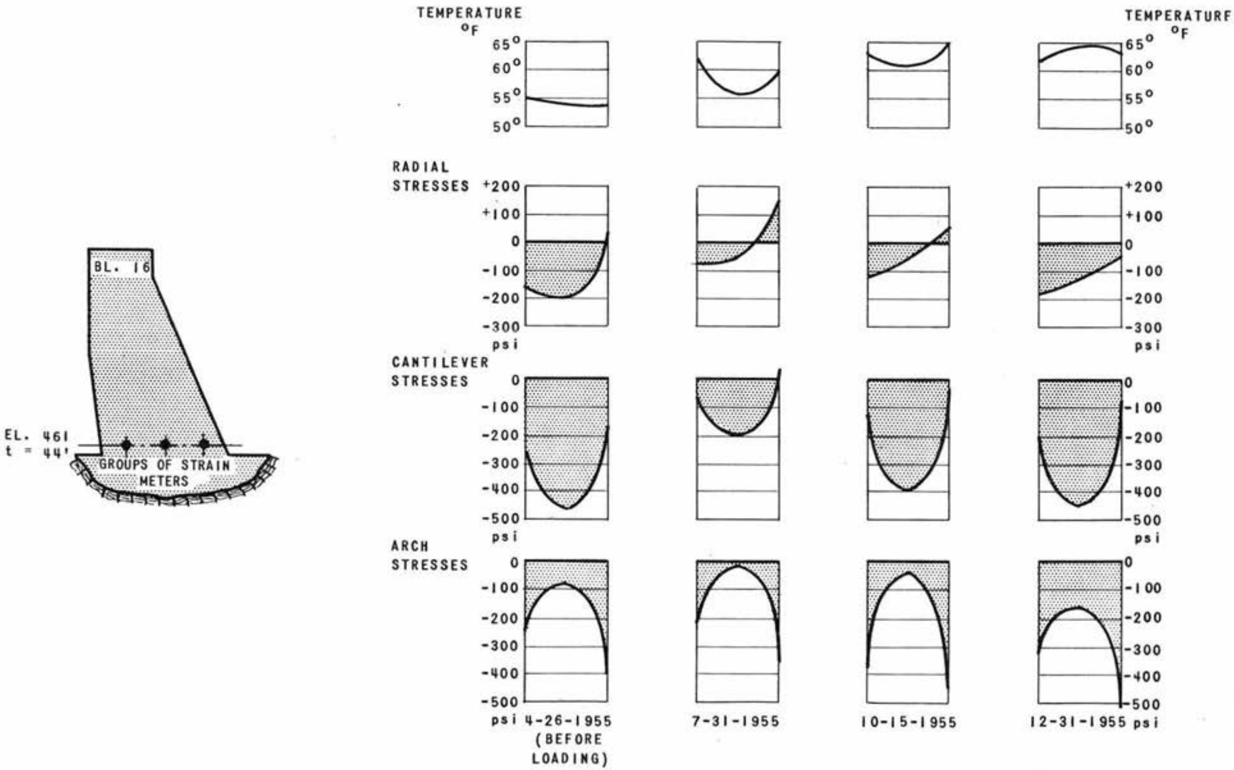
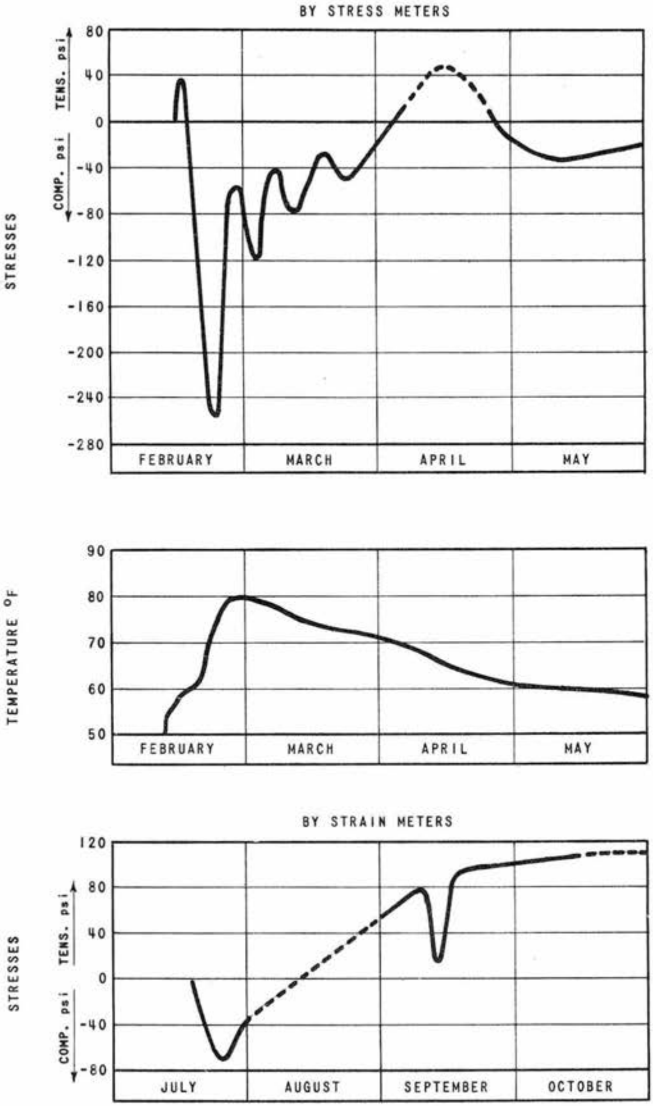


FIGURE 21

FIGURE 22

STRESSES LARGELY DUE TO RESTRAINT



2. The actual stress distribution in the concrete is at variance with that obtained from the detailed trial-load calculations (this may be said to be especially true of a young structure).

3. Stress distribution is non-linear. It is the temperature distribution rather than the mass temperature of a concrete section that plays an important role in the stress distribution.

4. The calculated tensile stresses do not appear in either the crown or abutment sections; it is possible that these are absorbed by joint opening (which could be called, for calculation purposes, "cracked arches" similar to the assumption of "cracked cantilevers").

5. The stresses tend to concentrate from the abutment toward the crown section.

6. The use of pipe cooling is very effective in reducing temperature stresses; the effect of daily temperature variation on the mass concrete was consistent with the calculated effects.

7. The elastic properties of the foundation rock varied greatly from point to point; however, it is probable that after consolidation grouting this variation was greatly reduced.

8. The construction of a structure like an arch dam on a relatively steep slope definitely requires contact grouting of the joints between the concrete and rock.

9. The effect of a temporary external load (as occurred during the unexpected typhoon water loading of the Kamishiiba Dam) on the grouted portion of an arch dam is relatively small and is easily recovered by subsequent grouting.

10. The effect of creep in the concrete on stress relaxation is significant, since the sustained modulus of elasticity was found to be almost half of the instantaneous modulus of elasticity - even after a duration of loading of one half year.

11. Necessity of duplication or double installation of stress and strain meters to check results is readily recognized.

12. It is questionable whether the extensive time and effort required for a trial-load analysis is justifiable in view of the relative "accuracy" of this method as contrasted to some of the less laborious methods of calculation.

Comments on the Measurements

H. Kimishima is continuing research in this field and includes the following clarifying statements with respect to the use of the various types of meters:

1. In order to make stress computations from strain meter readings, it is necessary to make creep tests on small laboratory cylinders (6 in. or 8 in.) with these results applied directly to mass concrete. Correlation between such a small specimen and mass concrete must be clarified so as to determine:

a. Effect of size of specimen,
b. Effect of maximum size of aggregate, and
c. Effect of adiabatic temperature changes on creep.

2. Poisson's ratio during creep was assumed to be the same as during elastic deformation. This assumption must be checked on mass concrete under the mass curing condition.

3. Until these effects are clarified, the interpretation of stresses from strains is not entirely correct; this leads to the conclusion that stress meters are more reliable than strain meters.

ACKNOWLEDGMENTS

The majority of the work reported upon in this paper was under the direct supervision of Mr. Kimishima. Mr. Kimishima will continue the analyses of stresses, strains, and deflections in the Kamishiiba Dam. It is planned that continuing reports on the behavior of the dam will be made from time to time. The authors are deeply indebted to the many other engineers of the Kyushu Electric Power Company and Ebasco Services who assisted in the installation, readings, and interpretation of the results.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAM: CONSTRUCTION OF THE KAMISHIIBA ARCH DAM

K. M. Mathisen,[1] and Charles C. Bonin,[2] M. ASCE
(Proc. Paper 1183)

FOREWORD

This paper is one of a group presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams (April, 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time, it is not known exactly how many papers will be printed from the Symposium. So far, fifteen papers have been approved: "Arch Dams: Their Philosophy," by Andre Coyne (Proc. Paper 959); "Arch Dams: Trial Load Studies for Hungry Horse Dam," by R. E. Glover and Merlin D. Copen (Proc. Paper 960); "Arch Dams: Portuguese Experience with Overflow Arch Dams," by A. C. Xerez (Proc. Paper 990); "Arch Dams: Theory, Methods, and Details of Joint Grouting," by A. Warren Simonds (Proc. Paper 991); "Arch Dams: Santa Giustina Single-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 992); "Arch Dams: Measurements and Studies on Santa Giustina Dam," by Claudio Marcello (Proc. Paper 993); "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 994); "Arch Dams: Isolato Double-Curvature Arch Dams," by Claudio Marcello (Proc. Paper 995); "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs," by Claudio Marcello (Proc. Paper 996); "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira (Proc. Paper 997); "Arch Dams: Development in Italy," by Carlo Semenza (Proc. Paper 1017); "Arch Dams: Design of the Kamishiiba Arch Dam," by C. C. Bonin and H. W. Stuber (Proc. Paper 1018); "Arch Dams: Observed Behavior of Several Italian Arch Dams," by Dino Tonini (Proc. Paper 1134); "Arch Dams: Measurements and Studies of Behavior of Kamishiiba Dam," by H. Kimishima and C. C. Bonin (Proc. Paper 1182); and "Arch Dams: Construction of the Kamishiiba Arch Dam," by K. M. Mathisen and C. C. Bonin (Proc. Paper 1183).

Note: Discussion open until July 1, 1957. Paper 1183 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 1, February, 1957.

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As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

SYNOPSIS

The Kamishiiba Arch Dam was constructed between September, 1952 and June, 1955. In anticipation of the adoption of either an arch or gravity dam, the diversion tunnel and upstream cofferdam had been completed prior to September, 1952.

Construction engineers from Ebasco Services Incorporated were assigned to assist engineers of the Kyushu Electric Power Company in the construction of the arch dam. It was the main task of the American engineers to assist in the preparation of engineering and design specifications and to confirm that these were being met by the construction forces.

As a result of many unforeseen factors, the actual construction period was 30 months although this was 6 months more than actually required for the job. Delays were caused by shortages of units of the construction equipment, tools, and spare parts, and by damages caused by two severe typhoons in the autumn of 1954 to portions of the construction plant and partially completed project.

A token placement of concrete was made in the dam on June 2, 1953; major concreting started in November, 1953. The diversion tunnel was closed during January, 1955; the construction sluiceway through the arch dam was closed on April 27, 1955, at which time storage of water in the reservoir began. On May 26, 1955, the powerhouse was put into operation.

Plant and Equipment

Assembling the equipment and erecting the construction plant was begun in September, 1952. This was to be the largest and most modern plant that had ever been used for heavy construction in Japan. The plant and most of the large equipment were owned by the power company and used by the contractor and subcontractors employed to build the dam. The contractor supplied some of the equipment and was responsible for all equipment and plant maintenance.

The plant was arranged on the right side of the river valley as indicated on Plate 1. It was dominated by the crushing and screening plant which was capable of producing 300 tons of crushed rock and sand per hour. The rock was graded into 16, 8, 4 and 2 centimeter sizes. Primary crushing was done by two 150-hp jaw crushers; two 100-hp gyratory crushers and three 100-hp cone crushers were employed for secondary crushing. Sand was manufactured by three 35-ton-per-hour rod mills. The sand and aggregate were delivered to the concrete batching and mixing plant by belt conveyors which passed the aggregate through a final washing and dewatering station.

An important feature of the plant was the 57-kilometer aerial cableway used for delivering cement, sea sand for concrete for other portions of the development and miscellaneous materials. The cableway had been built originally to supply sand and cement for the construction of two existing downstream developments; the cableway was rehabilitated and extended to serve this project. It was capable of delivering materials, packed in its 9-cu-ft buckets, at the rate of 40 tons per hour. Approximately 130,000 tons of the cement used for the development were delivered by this cableway.

The cement for the dam was delivered to a warehouse at the cableway terminal where the cement was unsacked and pneumatically transported to two 750-ton storage silos. From these, it was delivered by a 40-ton-per-hour capacity system of conveyors to the concrete plant. The concrete plant was capable of producing 120 cubic meters per hour in its three 4-cu-yd mixers. The concrete plant was fully automatic and designed to be operated by one man. Laboratory facilities were installed for the concrete control inspectors.

Water for construction uses was obtained from a small tributary near the dam site. The pumping station contained seven multiple-stage centrifugal pumps capable of supplying 13 cubic meters of water per minute to the main storage tank. Located adjacent to the batch plant was the refrigeration and ice plant capable of cooling 1800 liters of water per hour to 1.6° C. This cooled water was used in the concrete mix and for cooling by circulation through embedded cooling coils in the dam. The ice plant was also used for producing up to 30 tons of ice per day that could be substituted for a part of the concrete mixing water.

All concrete, except small quantities used for miscellaneous construction, was delivered from the concrete plant to the placing site in single chamber 4.5-cubic-meter buckets. A double-track transfer railway was employed to move the empty and full buckets between the plant and the pickup zone of the construction cableways. Two 13.5-ton capacity flatcars and three 81-hp diesel locomotives were used in this service. The buckets were operated by applying compressed air to the dumping mechanism at the placing site.

Twin 13.5-metric-ton capacity cableways, with a span of 430 meters between towers, served the dam site. Each was capable of delivering concrete to any location in the dam at approximately 60 cubic meters per hour. They had a common fixed head tower on the right side of the valley and individual movable tail towers mounted on a curved tail-tower track located above the left abutment and thrust block. Their carriages traveled at speeds up to 360 meters per minute, and their load lifting speed was as high as 90 meters per minute. Vertical and horizontal motions were independent; the mechanism was not designed for both to be operated simultaneously. The tail towers traversed upstream or downstream at 12.5 meters per minute in a separate motion or simultaneously with either one of the other two motions.

Other features of the plant included the necessary warehouses and shops, and included a double-track funicular railway which was used to haul aggregate rock salvaged from excavation operations from the river bottom to the primary crusher. It had two 7.7-ton capacity skips that were alternately loaded and dumped giving the machine a maximum capacity of 75 tons per hour.

Two small bulldozers were delivered to Kamishiiba in the summer of 1951. A few other pieces of equipment followed at odd intervals until early in 1953 when the bulk of the equipment arrived. After this, additional equipment was delivered at scattered intervals until the end of 1954. The principal units of equipment used in the construction of the dam are listed below:

2 American-built 1-3/4-cu-yd diesel-power shovels
1 American-built 3/4-cu-yd diesel-power shovel
3 Japanese-built 1.2-cubic-meter diesel-power shovels
1 Japanese-built 1.5-cubic-meter electric-power shovel
1 European-built 2.0-cu-yd diesel-power shovel
6 Japanese-built D-50, 44-hp bulldozers
4 Japanese-built D-80, 75-hp bulldozers
6 American-built D-8, 1113-hp bulldozers
2 American-built D-7, 80-hp bulldozers
1 Japanese-built 75-hp road grader (removed in 1953)
9 American-built 10-cu-yd end dump trucks
13 Japanese-built 4-cu-yd end dump trucks
16 American-built wagon drills
10 Japanese-built wagon drills
73 Japanese-built jack hammers
17 Rotary drilling machines
5 Japanese-built grout pumps with mixers
2 American-built grout pumps with mixers
3 American-built 600-cu-ft-per-minute portable air compressors
55 Japanese-built concrete vibrators
10 American-built concrete vibrators
7 Japanese-built portable electric-arc welders
5 Japanese-built portable acetylene welders
Several Japanese-built 2-ton trucks used for hauling materials and supplies

Most of the pieces of lesser equipment, not listed, were of Japanese manufacture. The concrete plant and most of the refrigeration and ice plant machinery were the largest units of the construction plant that were imported into Japan. The remainder of the plant was of Japanese manufacture or built by Japanese firms under license from American and European manufacturers.

Foundation Excavation

The location of the arch dam was governed by the foundation rock at the site; alignment was not symmetrical with the river channel and valley. This condition, plus the fact that a thrust block and cutoff wall were necessary on the left abutment, required that a greater volume of rock be excavated on the left abutment than on the right abutment. It was also necessary to do some excavation in the left forebay to provide an adequate channel to the spillway in block #4 (see Plate 2). The depth of excavation at any elevation was governed by the requirements that the rock be sound and adequately strong, the excavated foundation face be radial, and the slope of the abutment be continuous with no sharp breaks or wide benches.

Removing and wasting the excavated material presented a difficult problem in the confined river valley. It had been planned that all acceptable rock be delivered to the crushing plant to be made into aggregate. This was never fully accomplished because of the late installation of the funicular railway for hauling the rock, and the failure to segregate and stockpile the acceptable rock. Therefore, most of the material was eventually wasted in a spoil area on the right river bank about one kilometer below the dam. Little material was wasted in the reservoir area because there were no roads on which to operate equipment on either side of the valley above the dam.

The foundations for blocks 11, 12 and 13 on the right abutment were completed first. The second area completed was the river channel in which blocks 7, 8, 9 and 10 were located. Thus, difficult excavation remained to be done on both abutments above concrete-placing operations in the completed areas. At times these operations conflicted and required delicate scheduling in order not to result in excessive delays. Excavation of the upper right abutment was complicated by the crossing of the transfer railway and sometimes delayed by slides into the abutment area above the railway.

In March 1955 foundation excavation was completed. In spite of the tedious manner in which the foundation was excavated (because of the necessity of using local methods and equipment), the foundation rock was prepared in a thorough and acceptable manner. The rock exposed on the right abutment was jointed and weathered deeper than anticipated. In the river bottom, on the left abutment and especially in the thrust block area the foundation rock was better than anticipated. The final profile varied only slightly from the assumed profile for the final design. Approximately 290,000 cubic meters of rock and overburden were excavated to provide a sound foundation for the arch dam, thrust block and cutoff, and spillway chutes.

Foundation Grouting

In general, foundation "consolidation grouting" was begun in an area only after it had been satisfactorily excavated and accepted for construction. "Curtain grouting" was considered the second phase of the foundation grouting program.

Consolidation grouting was started in blocks 12 and 13 in May 1953. Eight-, 12- and 16-meter deep holes were drilled on 5-meter centers in a rectangular pattern. Those holes 8-meters deep were drilled with 3-in. wagon drills. The others were 60-millimeter shot or diamond drill holes. The holes were washed with an air-water jet pipe, then threaded 2-in. pipe nipples were set in the holes; pressure testing and grouting were generally done at 50 psi.

The results of this early work showed that the planned grouting program required revision. A new hole pattern was adopted that was a compromise between grouting before placing any concrete and allowing concrete placing to begin without delay. The revised program planned alternate rows of 3- and 7-meter deep holes across the foundation. The rows were 2-1/2 meters apart, and the holes were drilled normal to the general slope of the foundation, 5 meters apart in the rows. The holes in one row were offset 2-1/2 meters from the holes in the adjacent rows. Drilling, washing, testing and grouting the rows of holes proceeded in an uphill direction ahead of placing the lifts of concrete, or were done between placing lifts of concrete from the top of the concrete in place. The latter procedure reduced troublesome surface leakage that often interfered with the work. In some areas all the holes were drilled 7 meters deep. When this pattern was completed in a block, two, four or six holes were drilled 16 meters deep. These holes were piped to the downstream face of the dam, to be grouted after concrete was placed in the block to a height of 16 meters or more above the foundation.

A system was developed to control the grout mix and the grouting pressure. This was based on the rate of grout take during a specified period of time. The mixes varied from a W/C ratio of 10 to a W/C ratio of 1 by weight. The pressure ranges were:

3 meter holes - 15-psi start to 25-psi finish
7 meter holes - 25-psi start to 50-psi finish
16 meter holes - 50-psi start to 75-psi finish

Additional holes were drilled and grouted in the area of any hole that took over five sacks of cement. This pattern and procedure were used over the entire foundation. The total drilling for consolidation grouting was 7300 meters.

A brief summary of the consolidation grouting is as follows:

Block	Average Take: 50-Kg Sacks Per Meter		
	3-Meter	7-Meter	16-Meter
Thrust B1, 2 and 3	0.19	0.14	0.44
4, 5, 6 and 7	0.23	0.25	0.95
8, 9 and 10	0.25	0.51	0.85
11, 12, 13 and 14	0.52	0.61	2.59
15, 16 and 17	0.13	0.33	0.92
TOTAL	0.34	0.42	1.32

These "takes" reflect the quality of the foundation in the various areas; the rock under blocks 11, 12, 13 and 14 possessed the most seams and fissures, while that under the thrust block and blocks 2 and 3 was the soundest.

A continuous grout curtain was produced by testing and grouting a row of holes drilled into the rock beneath the dam. The curtain was extended a short distance along the reservoir rim beyond each end of the dam. Curtain grouting was started early in 1953 on the right reservoir rim and was completed in July 1955 on the left reservoir rim. Approximately 7000 meters of hole were drilled and grouted to complete the rim curtain work. It was done entirely by the stage drilling and grouting method. Stage grouting using deep packers was considered, but rejected because it was believed this would not be as satisfactory with the type of worker available at the site.

Curtain grouting under the dam was begun in January 1954 in block 9 in the river channel. From this point the grouting was carried upward on both abutments as the blocks were completed to heights approximately 30 meters above their foundations. All work was done in this order:

1) "Exploratory" holes, 50 meters deep, 20 meters center to center were drilled in three equal length stages that were grouted at 100-, 200- and 300-psi pressures.
2) "First intermediate" holes, located halfway between "exploratory" holes, were started when the first two stages of the adjacent holes were completed. They were drilled one-third of the waterhead but not less than 25 meters deep. They were drilled in two equal length stages and grouted at 100- and 200-psi pressures.
3) "Second intermediate" and "third intermediate" holes were similar to and followed the "first intermediate" holes in order. They were located halfway between adjacent existing holes making the final hole spacing 2-1/2 meters center to center.

The curtain hole spacing was spread to 3-1/3 meters center to center in blocks 5 and 6, and to 5 meters in block 4. Several diagonal check holes were required on the right abutment in zones of relatively high take and to check-grout several small vertical seams.

Most of the earlier drilling was done using 65-millimeter shot drills. This drilling was very slow; the advance of the bit being controlled entirely by the force exerted on the machine by the driller. It was used for reasons of economy, diamond bits being very expensive and labor costs being relatively low. Diamond drilling, using "EX" size bits eventually replaced the shot drilling; at the conclusion of the curtain grouting program, it was the only method being used. The curtain grouting has proven very satisfactory; seepage under the dam has been negligible.

The grout takes for the curtain were:

Exploratory Holes	1.5	50-kg sacks per meter
First Intermediate Holes	0.6	50-kg sacks per meter
Second Intermediate Holes	0.4	50-kg sacks per meter
Third Intermediate Holes	0.4	50-kg sacks per meter
Right Abutment Check Holes	0.3	50-kg sacks per meter

Quarry Operations and Aggregate Production

Shortages of aggregate and sand caused several minor delays in the construction of the dam. The problem was primarily one of quarry operation, and secondarily one of scheduling the heavy earth-moving equipment between dam-site excavation and the quarry. The area selected as a quarry contained quantities of sound graywacke adequate for the concrete required in the dam, but it also contained thick beds of slate and zones of deep weathering. Coyote-hole blasting was used in the quarry; this mixed quantities of the unacceptable rock and some overburden with the good rock and also left a number of extra large pieces which slightly delayed operations by requiring drilling and secondary blasting.

Between July 1953 and July 1954, 81,200 tons of sound rock were salvaged from the power tunnel-excavation muck. This was 9% of the aggregate rock hauled to the crusher. Between October 1953 and March 1955, 181,700 tons of rock were salvaged from the foundation excavation and from river-channel improvement excavation at the end of the spillway chutes. This accounted for 19% of the aggregate rock. A total of 980,000 tons of broken rock were delivered to the crushing plant for crushed-rock aggregate and manufactured sand for concrete production.

Various means were employed to assure that the highest quality of aggregate was used in the concrete. Washing stations were set up before and after the primary crusher. Pickers were stationed along the various conveyor belts to remove by hand pieces of unacceptable aggregate. Rock from the three sources was blended to produce the best possible aggregate from the material delivered to the crusher.

Concrete Control

The design of concrete mixes, and the investigation and testing of available brands and types of cement, was begun in October 1952. Concrete specifications required a minimum of 235 kg of cement per cubic meter of concrete placed in contact with the foundation, placed within one meter of the faces of the dam, or placed in reinforced sections. This was designated "A" mix. A minimum of 210 kg of cement per cubic meter was specified for interior mass concrete for the dam, designated "B" mix. In 1954, a "C" mix containing 190 kg of cement per cubic meter was adopted for miscellaneous concrete work. Type II cement as described in U.S. Federal Specifications SS-C-192 was adopted for all cement in the dam.

A thorough concrete and aggregate testing program was established. Inspectors were stationed at the quarry, crushing plant, concrete plant and the placing site. The gradation and moisture of the aggregate and sand were checked at least once each shift. Slump tests were made and the entrained air content was determined at frequent intervals. Standard 6" x 12" test cylinders were cast during each shift for 7, 28 and 91 days, 6 months' and 1-year break tests. Testing of the cement was carried out in the manufacturers' laboratories and periodically witnessed by engineers from the field laboratory. Overall quality control was entirely adequate and remarkably effective.

The monthly average of concrete strengths obtained at 91 days are given in Table 1.

Construction Operations

The dam, as constructed, is shown in plan and profile on Plates 2 and 3. The arch blocks, thrust block and cutoff were constructed with two-meter concrete lifts, except on the foundation where one-meter lifts were specified.

Contact grouting systems were installed on all foundation surfaces having a general slope steeper than 30 degrees from the horizontal. Each system contained two horizontal 1-1/2-in. header pipes joined by 3/4-in. riser tubes to which the necessary grout outlets were attached. All headers opened at the downstream side of the dam.

Timber forms were utilized for all concrete placed. Most of the lumber and timber required was cut by sawmills at the job site. Once off the foundation, raisable form panels were used at the faces of the blocks. These panels had vertical sheathing planks and were sufficiently flexible to be easily warped to the curvature of the arch. Two-inch chamfer strips were used at the horizontal construction joints and the vertical block joints. The joint forms of the high blocks consisted of a series of keyway forms connected by narrow panels backed by timber whalers. All forms were anchored by embedded metal tie rods. The vertical water and grout stops, the horizontal grout stops, the grout boxes and tubing systems and the grout vent grooves were all fastened to the joint forms.

The concrete placing procedure was to begin at the downstream face of a block and finish at the upstream face. All concrete was placed in terraces about one-half meter thick and thoroughly vibrated into place. One cableway was used per block, except in the large base lift of the blocks in the river bottom, where both cableways were used and placing was started at both faces of the dam to be completed at the center of the block. The surface of each lift was green-cut using jets of air and water after the concrete had taken its final set. It was often necessary to start this operation on the downstream portion of a lift while concrete was still being placed in the upstream portion. After green-cutting, and during forming for the next lift, the cooling coils were installed on the concrete surface. The 1-in. thin wall steel tubes were spaced at 1, 1-1/2 and 2 meters depending upon the location in the block and in the dam. They were fastened by means of wires set in the surfaces of the concrete lifts. Slip-type tubular couplings were used to fasten the lengths of tubes and the elbows together.

A cooling plan was set up in which refrigerated water was circulated through the embedded coils for a primary cooling period of approximately 14 days. After several weeks a secondary cooling was initiated. This was designed to cool the concrete to the mean ambient temperature of 15.6^{o} C. Cold river water was utilized during the winter months. The cooling program was remarkably successful in preventing unduly high concrete temperatures and in preventing cracks in the completed structure. Cooling was usually continuous until the concrete was at or near the mean ambient temperature.

The capacity to produce and place concrete was entirely adequate for the job. With the opening of the river bottom in November 1953, the concrete placement began in earnest, and continued on a normal schedule for 14 months until the project was nearly complete. In January 1954, slightly over 2400 cubic meters were placed in just less than 24 hours; this established a new placing record in Japan at that time. After this, hourly rates often exceeded 100 cubic meters per hour. The total volume of concrete as measured at the batch plant was 381,357.5 cubic meters. This volume was placed in the dam, thrust block, cutoff and spillways.

Contraction Joint Grouting

In order to obtain a monolithic structure, it was necessary to thoroughly grout the vertical contraction joints. These joints were divided into 20-meter grout lifts. Each lift was enclosed by Z-shaped copper seals anchored in the concrete of the adjacent blocks. Systems of embedded tubing with grout outlet boxes in the joint were installed to inject the grout into the joints. The outlet boxes were spaced 3 meters vertically and the riser tubes 2 meters apart. The vent groove at the top of each lift was not connected to the tubing system.

The grout lifts were established by placing horizontal grout stops at the foundation rock and at elevations 390, 410, 430, 450, 472 and 482 meters. In the spring of 1954, the elevation 430 to 450 grout lift was divided into two portions by adding a stop at elevation 440. This permitted joint grouting as high as possible before closing the dam in September, as was then scheduled. Reinjectable grout systems imported from France were installed in addition to the regular systems above elevation 460 and were the only systems installed above elevation 472.

In the spring of 1954, grouting of concrete-rock contact systems installed on the lower portions of the foundation was begun. These systems were washed for several hours, pressure tested at 1 psi for each 1-ft height of concrete above the system, then grouted at the same pressure using a grout mix with a W/C ratio of 4 by volume, or thicker when it was desirable. Grouting was continued until refusal; pressure was maintained on the system for two more hours. Water was circulated in adjacent ungrouted contact and joint grouting systems to protect them from damage from plugging by grout intrusions from the system being grouted.

All contact systems located below the elevation of the top of a grout lift were completed before joint grouting was started in that lift. All contact grouting was delayed until the concrete had cooled to 15.6° C. Grout takes were generally small, although a few systems had relatively high takes; these latter were usually on the steeper foundation surfaces. A summary of the contact grouting is shown on Table 2.

The contraction joint grouting, begun in May 1954, was the first such program in Japan. The majority of the lifts of the joints were grouted individually; there were cases where it was necessary or desirable to grout two joints in one lift or two lifts in one joint simultaneously. All joints were satisfactorily grouted up to elevation 472 by the end of April 1955. Above elevation 460, concrete temperatures were above the mean ambient temperature of 15.6° C when the joints were grouted. The reinjectable systems were installed for just such a situation and were used in the spring of 1956 for regrouting.

In scheduling the joint grouting, temperature of the concrete was the prime consideration. Mean ambient temperature or lower was considered necessary for maximum concrete shrinkage, maximum joint opening and maximum effectiveness of grouting. At times it was necessary to grout with concrete temperatures nearly one degree higher. Temperatures were determined by the embedded thermometers, stress meters, strain meters and joint meters. These readings were intermittently checked by filling selected cooling coils with water and capping them for as long as 48 hours. The water was then slowly forced out as its temperature was measured. The average temperature of such water corresponded very closely with the temperature readings of the embedded instruments.

Once a lift was ready for grouting, the joints were given a preliminary water test to locate any serious leaks. Leaks of some magnitude existed in nearly every joint; all face leaks were caulked before grouting began. Specifications required that adjacent ungrouted joints be filled with water and the headers capped; this was done to minimize block movement and prevent the adjacent joints from being closed by grouting pressures. Experience showed that few of the joints were sufficiently watertight to accomplish this; therefore, it was necessary to use flowing water in the adjacent ungrouted joints. Flowing water was also maintained in the lift of the joint above the lift being grouted. In cases of known open leaks, this water was maintained under enough pressure to eliminate the leakage past the horizontal grout stop.

During all water testing and grouting operations, dial gages were mounted on the downstream face of the dam across the joint or joints being tested or grouted, and also placed across those adjacent joints in which water was circulated. Testing and grouting were done at pressures at the upper vent headers that produced a maximum movement of 0.8 millimeter or at a maximum of 60 psi.

Grouting was done by following a routine procedure. The grout was introduced to the lower supply header, circulated through it and returned to the mixer. The return header valve was then closed and the grout forced into and upward through the joint and out the open upper vent header valves. These valves were then closed and the pressure in the joint controlled by properly throttling the lower return header valve. The upper vent header valves were cracked as often as necessary to bleed off excess water and thin grout. Grout was circulated continuously through the lower supply-return header to replace the water lost through bleeding. This was continued until the joint was completely grouted. Pressure was then held on the joint for an additional two hours to allow the grout to set.

Type I cement was used for all joint grouting. To remove lumps and any possible foreign matter the cement was screened through a # 30 screen immediately before use. The W/C ratio of the grout mix used was 1 by volume. The initial batch of each operation usually had a W/C ratio of 2 or 4 by volume. The initial joint openings as indicated by the embedded joint meters ranged from a minimum of 0.5 millimeter to a maximum of slightly over 4 millimeters.

After completion of this work, several core holes were drilled into joints which had been grouted to sample the grout deposits. In general, these check holes revealed that the joint grouting had been very effective.

Table 3 is a tabulation summarizing the contraction joint grouting.

Closure of Dam

The plan for closure of the dam was to follow a sequence as follows:

1) A vertical shaft, approximately 1-1/2 meters in diameter, had been excavated from the heel of block 4 to the location chosen for the concrete plug in the diversion tunnel.
2) Two holes had been cut in the concrete arch cofferdam. Timber flap gates to cover the holes were installed on the upstream face of the cofferdam and secured in the open position.
3) The diversion tunnel was closed by setting stop logs in the intake portal of the tunnel thus diverting the river through the openings in the cofferdam and into the construction sluiceway through the arch dam.
4) After the diversion tunnel completely drained, keyways were excavated in the tunnel walls for a plug 10 meters in length, the plug area cleaned, and radial grout holes drilled for grouting after the plug was completed.
5) Concrete for the tunnel plug was placed by using spouts suspended down the shaft; two sets of cooling coils were embedded in the plug; the vertical shaft was completely backfilled with concrete.
6) After the plug was completed and grouted the flap gates were dropped closing the holes in the cofferdam. This allowed a short period of time (approximately 3 hours) for the water to drain through the sluiceway, exposing the closure gate seats. The seats were cleaned and the gate lowered into place sealing the sluiceway.
7) The sluiceway was then backfilled with concrete and the plug sealed by grouting.

By March 1955, the first five steps of the closure plan had been completed and all joints up to elevation 450 had been grouted. River discharge was low, making this an opportune time to complete closure of the dam. However, in view of the possibility of early spring floods raising reservoir levels quickly to the spillway crest, decision was reached that all joints should be grouted up to elevation 472 and that the curtain grouting should be completed under all portions of the dam before closing. Allowing time for this work, as well as for other operations, April 16th was set as the closure date. Closure was delayed by 8-1/2 in. of rain and a small flood which occurred April 15. On

April 27, 1955, closure was accomplished. One flap gate temporarily failed to close, which caused momentary difficulty and anxiety; nevertheless, the sluiceway gate was successfully sealed, but just 2 minutes before the cofferdam was overtopped.

With the closure of the dam, reservoir level rose quickly and on May 26, 1955 reached a level that permitted the power plant to go into operation. Later, on June 25, 1955, the Kamishiiba Hydro-Electric Development was formally dedicated. And so, after 30 months of hard work, in spite of extremely difficult topographic conditions and the adversities of nature, the combined efforts of Japanese and American engineers were able to successfully complete the first arch dam to be constructed in the Far East.

TABLE 1

AVERAGE 91-DAY COMPRESSIVE STRENGTHS OF 6" x 12" TEST CYLINDERS

(Kg/cm^2)

Period	"A" Mix	"B" Mix	"C" Mix
June-October 1953	385	333	-
November	397	358	-
December 1953	390	360	-
January 1954	375	347	-
February	406	350	-
March	435	384	347
April	375	373	281
May	410	345	304
June	400	-	318
July	410*	390	321
August	388	357	312
September	390*	355	287
October	412	389	337
November	444	-	363
December 1954	440*	427	338

* Approximate

TABLE 2

SUMMARY OF CONTACT GROUTING

Operation Number	Location	Foundation Slope	Area M^2	Outlets	Area/ Outlet	Estimated Net Take 50 kg Sacks
1	Bl 7	40^o	390	39	10.0	1.4
2	Bl 10 Lower	35^o	155	32	4.8	2.3
3	Bl 8-9	25^o	55	11	5.0	1.8
4	Bl 10 Upper	45^o	380	54	7.0	13.5
5	Bl 9	40^o	160	25	6.4	2.0
6	Bl 11 Lower	40^o	100	16	6.2	0.3
7	Bl 11 Upper	35^o	220	23	9.6	7.6
8	Bl 6 Lower	35^o	75	23	3.2	2.9
9	Bl 6 Upper	40^o	340	24	14.1	8.7
10	Bl 12	90^o	90	28	3.2	10.0
11	Bl 13	45^o	320	37	8.7	1.1
12	Bl 5	45^o	204	30	6.8	1.4
13	Bl 3	55^o	238	37	6.4	5.2
14	Bl 15	55^o	140	28	5.0	2.9
15	Bl 2	50^o	100	18	5.5	9.6
16	Bl 1	40^o	210	22	9.5	1.1
17	Bl 16-17	90^o	30	6	5.0	85.1*
18	Diversion Tunnel Plug - Take not determined					

* System in end of old foundation exploration tunnel

TABLE 3

SUMMARY OF CONTRACTION JOINT GROUTING

Grouting Operation	Joint	(Elev. in Meters) Lift	Sq. Meters Area	Grouting Pressure psi At Top of Lift	Opening Movement Recorded mm 1/	Est. Avg. Joint Opening Under Grouting Pressure mm	Estimated Cement Required 50-kg Sacks	Total Cement Accounted 50-kg Sacks 2/	Liters/Min. Leakage Indicated by Water Pressure Test
1	9-10	Rock-390	362	60	0.4	1.4	20.4	41	Trace
2	(8-9	Rock-390	448	60	)	1.3	65.8	163	23
	(8-9	390-410	642	35	0.7)				
3	7-8	Rock-390	250	60	0.1	0.8	7.2	22	13
4	(6-7	Rock-410	413	60	0.5	2.2)	76.1	157	(30
	(7-8	390-410	532	60	0.7	1.6)			(60
5	(10-11	Rock-410	402	60	0.2)	1.3	19.4	39	12
	(11-12	Rock-410	90	60	)				2
6	9-10	390-410	542	60	0.5	1.4	34.6	70	
7	9-10	410-430	487	40	0.8	2.1	43.3	239	75
8	7-8	410-430	473	30	0.7	2.2	43.9	164	110
9	8-9	410-430	486	50	0.5	1.9	39.7	97	47
10	10-11	410-430	483	60	0.6	1.8	37.8	25	16
11	6-7	410-430	484	40	0.8	2.5	50.1	111	
12	(11-12	410-430	483	60	0.0	1.1)	31.2	46	10
	(12-13	Rock-430	233	60	0.0	0.3)			10
13	5-6	Rock-430	413	60	0.4	2.2	38.3	130	26
14	4-5	Rock-430	99	60	0.2	2.0	8.4	26	58
15	8-9	430-440	204	60	0.3	2.5	21.0	20	8
16	(10-11	430-440	204	60	0.6	1.7)	28.3	53	(8
	(11-12	430-440	204	60	0.3	1.4)			(8
17	6-7	430-440	204	60	0.8	3.4	31.3	50	12
18	5-6	430-440	204	60	0.7	2.7	22.5	32	36
19	7-8	430-440	204	60	0.5	2.9	24.0	35	9
20	9-10	430-440	204	55	0.4	2.5	21.1	40	25
21	(12-13	430-442	233	60	0.3	1.0)	21.2	32	(6
	(13-14	Rock-442	172	60	0.3	1.2)			(18
22	3-4	Rock-440	178	60	0.6	2.3	17.1	24	8
23	4-5	430-440	204	60	0.4	2.2	18.9	146	47
24	8-9	440-450	178	60	0.3	4.0	28.0	23	5
25	9-10	440-450	178	50	0.4	3.0	21.6	40	3
26	11-12	440-450	178	60	0.4	1.7	13.3	37	8
27	6-7	440-450	178	50	0.8	4.7	32.5	58	6
28	5-6	440-450	178	50	0.6	3.6	25.5	59	31
29	13-14	442-450	132	60	0.5	1.6	9.4	16	12
30	14-15	Rock-450	188	60	0.3	1.5	12.7	20	11
31	3-4	440-450	178	60	0.4	2.6	19.1	21	12
32	2-3	Rock-450	58	60	0.4	2.2	5.4	14	6
33	4-5	440-450	178	35	0.6	2.7	19.7	31	43
34	7-8	440-450	178	50	1.0	4.5	31.2	35	37
35	12-13	442-450	136	60	0.4	1.3	8.2	36	90
36	10-11	440-450	178	30	0.8	3.2	31.9	53	38
37	7-8	450-470	268	20	0.7	4.0	42.2	34	5
38	5-6	450-470	268	20	0.9	3.8	40.2	28	4
39	4-5	450-472	280	25	0.7	3.2	36.0	30	22
40	8-9	450-470	268	30	0.9	3.6	38.3	19	5
41	6-7	450-470	268	50	0.8	4.3	45.0	24	5
42	11-12	450-470	268	30	0.7	2.8	30.6	30	8
43	9-10	450-470	268	25	0.7	3.8	40.2	22	4
44	13-14	450-472	283	25	0.7	2.1	25.2	21	17
45	10-11	450-470	268	30	0.8	4.0	42.2	52	22
46	12-13	450-470	268	30	0.7	2.0	22.9	70	20
47	3-4	450-472	283	25	0.6	2.3	27.2	13	19
48	1-2	Rock-470	170	60	0.2	1.1	9.0	17	28
49	2-3	450-470	270	60	0.5	1.3	16.2	61	24
50	14-15	450-472	282	40	0.6	1.8	22.1	21	21
51	15-16	Rock-470	202	60	0.2	1.6	14.3	92	42
52	16-17	Rock-470	128	60	0.1	0.6	4.5	12	8

1/ Opening movement is that indicated by dial gauges at D/S face by grouting pressure tabulated.

2/ Total cement accounted includes all waste, bleeding and grout to fill external lines, as well as cement in joint and embedded system.

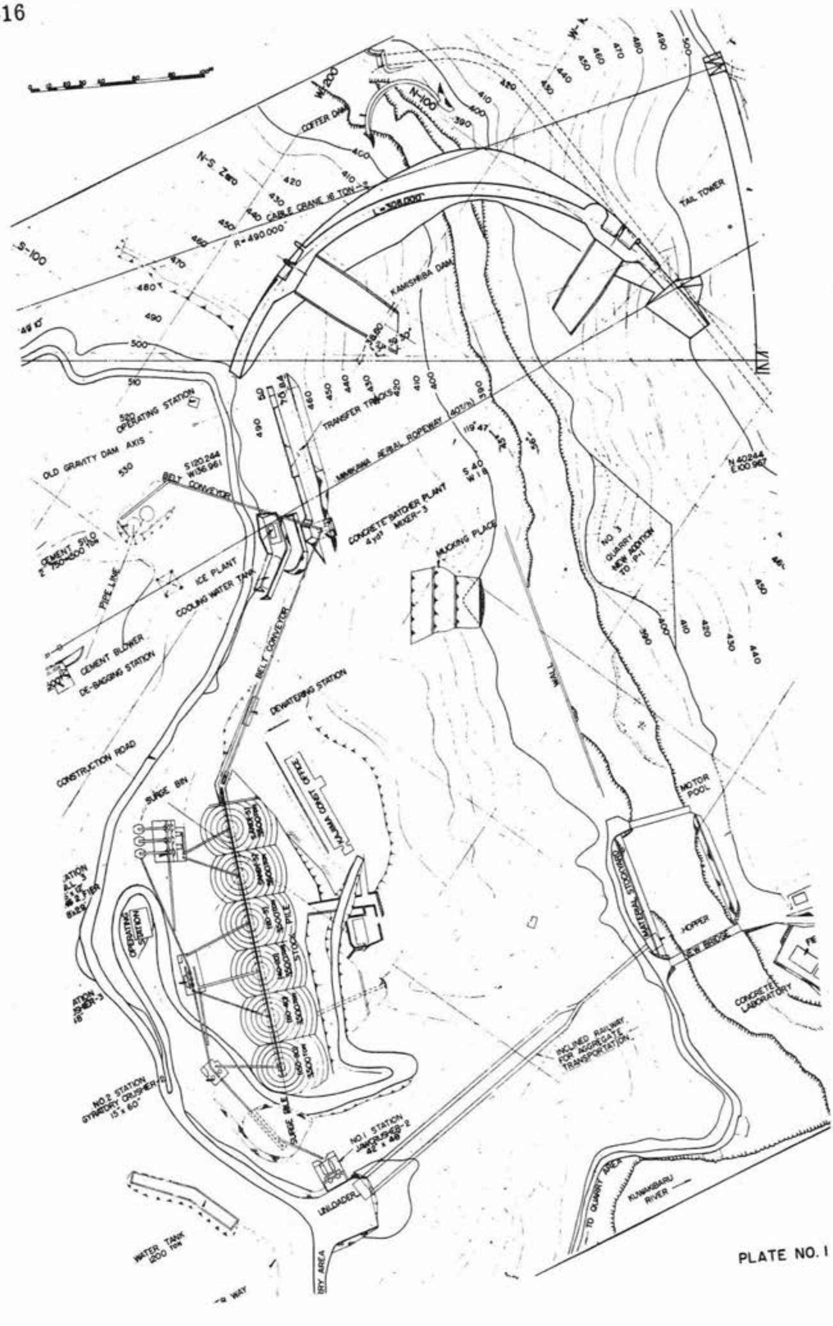
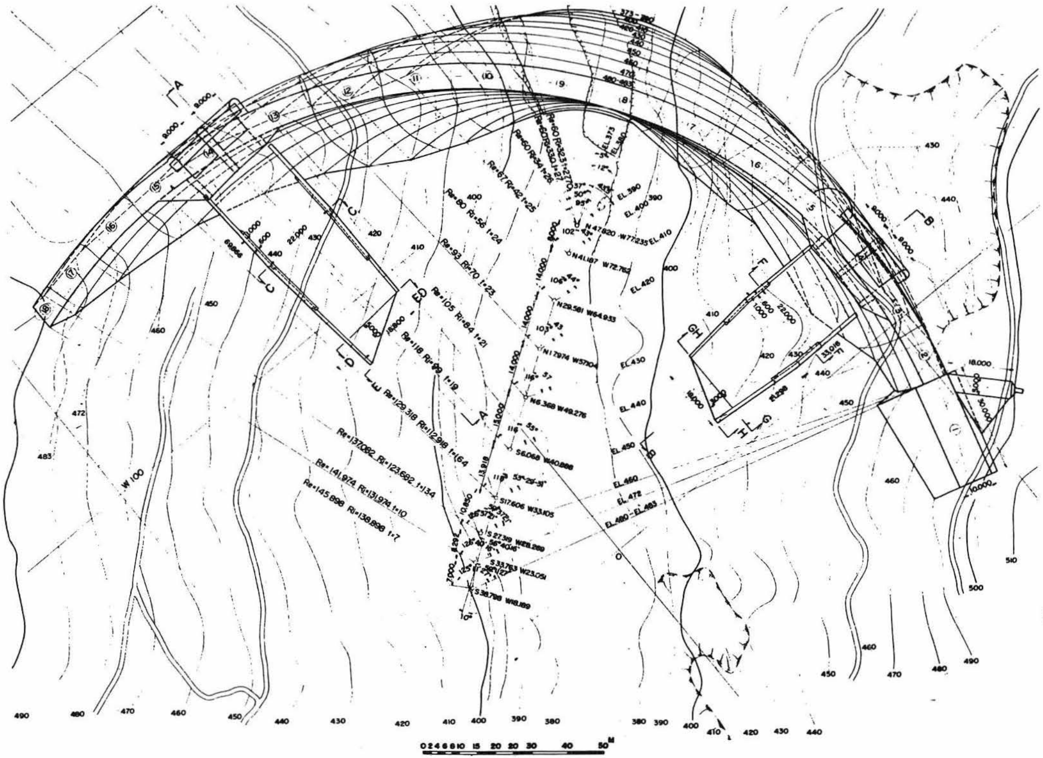

COFFER DAM
N-S Zero
CABLE CRANE 16 TON
R=490.000
L=308.000
KAMISHIBA DAM
TAIL TOWER
S-100
N-100
W-200
OPERATING STATION
OLD GRAVITY DAM AXIS
BELT CONVEYOR
TRANSFER TRACKS
AERIAL ROPEWAY
CONCRETE BATCHER PLANT
4yd³ MIXER-3
MUCKING PLACE
CEMENT SILO
PIPE LINE
ICE PLANT
COOLING WATER TANK
CEMENT BLOWER
DE-BAGGING STATION
BELT CONVEYOR
DEWATERING STATION
CONSTRUCTION ROAD
SURGE BIN
STOCK PILE
NO. 3 QUARRY
NEW ADDITION TO P-1
MOTOR POOL
MATERIAL STOCKYARD
HOPPER
NEW BRIDGE
CONCRETE LABORATORY
INCLINED RAILWAY FOR AGGREGATE TRANSPORTATION
NO. 2 STATION GYRATORY CRUSHER-2 15" x 60"
SURGE PILE
NO. 1 STATION JAWCRUSHER-2 42" x 48"
UNLOADER
WATER TANK 1200 TON
TO QUARRY AREA
KUWAKIBARU RIVER

PLATE NO. 1



FOR SECTIONS SEE
PLATE NO. 4

PLATE NO. 2

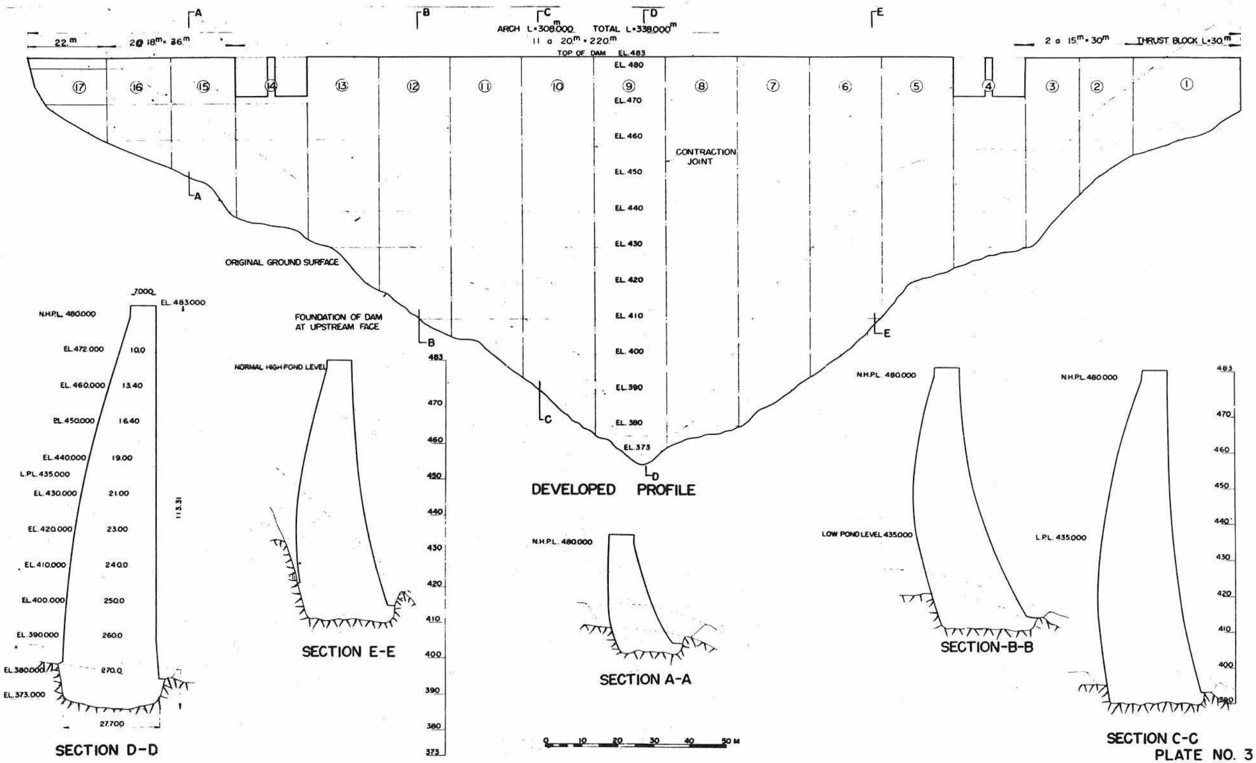
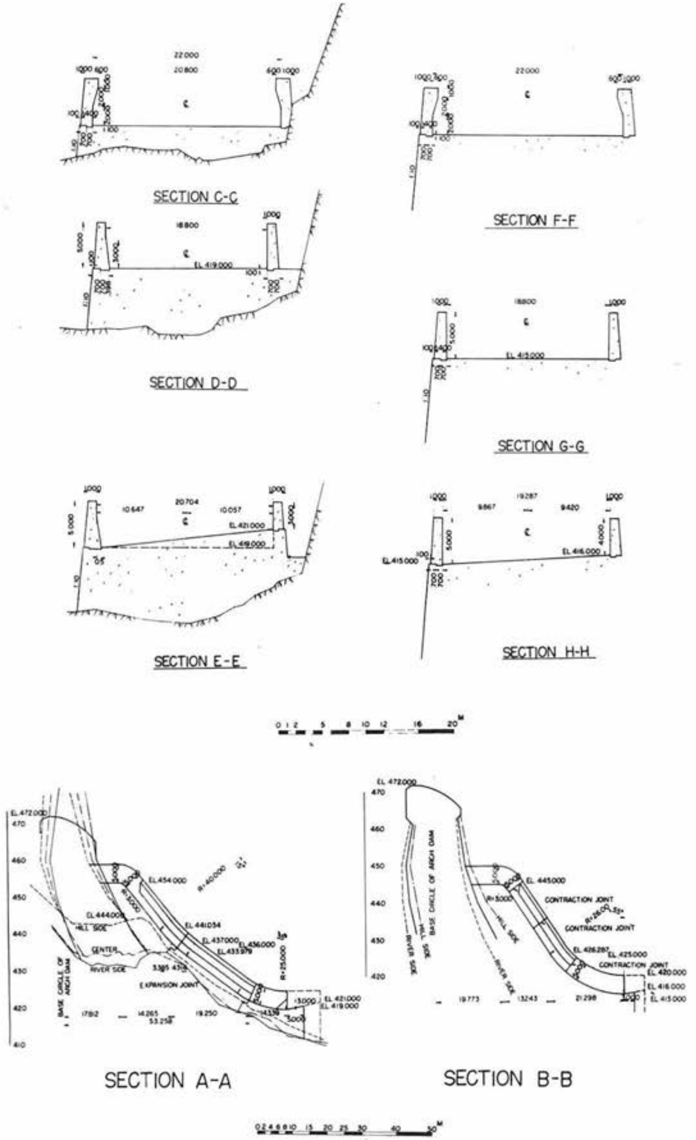

A
B
C
D
E
ARCH L=308.000 m
TOTAL L=338.000 m
22 m
2@18 m = 36 m
11 a 20 m = 220 m
2 a 15 m = 30 m
THRUST BLOCK L=30 m
TOP OF DAM EL.483
EL. 480
EL. 470
EL. 460
EL. 450
EL. 440
EL. 430
EL. 420
EL. 410
EL. 400
EL. 390
EL. 380
EL. 373
CONTRACTION JOINT
ORIGINAL GROUND SURFACE
FOUNDATION OF DAM AT UPSTREAM FACE
DEVELOPED PROFILE
N.H.P.L. 480.000
EL. 483.000
7.000
EL. 472.000
EL. 460.000
EL. 450.000
EL. 440.000
L.P.L. 435.000
EL. 430.000
EL. 420.000
EL. 410.000
EL. 400.000
EL. 390.000
EL. 380.000
EL. 373.000
10.0
13.40
16.40
19.00
21.00
23.00
24.00
25.0
26.0
27.0
113.31
27.700
SECTION D-D
NORMAL HIGH POND LEVEL
SECTION E-E
SECTION A-A
LOW POND LEVEL 435.000
SECTION-B-B
SECTION C-C
0 10 20 30 40 50 M

PLATE NO. 3



FOR LOCATION OF SECTIONS
SEE PLATE NO. 2

PLATE NO. 4

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: REVIEW OF EXPERIENCE

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(Proc. Paper 1217)

SYNOPSIS

The review presents records of the performance of some of the older arch dams to learn how well they had served the purposes of their designers. The records are presented in tabular form, supplemented by notes and pictures.

FOREWORD

This paper is one of a group presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams (April, 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time, it is not known exactly how many papers will be printed from the Symposium. So far, sixteen papers have been approved: "Arch Dams: Their Philosophy," by Andre Coyne (Proc. Paper 959); "Arch Dams: Trial Load Studies for Hungry Horse Dam," by R. E. Glover and Merlin D. Copen (Proc. Paper 960); "Arch Dams: Portuguese Experience with Overflow Arch Dams," by A. C. Xerez (Proc. Paper 990); "Arch Dams: Theory, Methods, and Details of Joint Grouting," by A. Warren Simonds (Proc. Paper 991); "Arch Dams: Santa Giustina Single-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 992); "Arch Dams: Measurements and Studies on Santa Giustina Dam," by Claudio Marcello (Proc. Paper 993); "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 994); "Arch Dams: Isolato Double-Curvature Arch Dams," by Claudio Marcello (Proc. Paper 995); "Arch Dams: Rio Freddo Dam with Gravity

Note: Discussion open until September 1, 1957. Paper 1217 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 2, April, 1957.

1. Research Engr., Bureau of Reclamation, U.S. Dept. of the Interior, Denver, Colo.

Abutments and Cut-offs," by Claudio Marcello (Proc. Paper 996); "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira (Proc. Paper 997); "Arch Dams: Development in Italy," by Carlo Semenza (Proc. Paper 1017); "Arch Dams: Design of the Kamishiiba Arch Dam," by C. C. Bonin and H. W. Stuber (Proc. Paper 1018); "Arch Dams: Observed Behavior of Several Italian Arch Dams," by Dino Tonini (Proc. Paper 1134); "Arch Dams: Measurements and Studies of Behavior of Kamishiiba Dam," by H. Kimishima and C. C. Bonin (Proc. Paper 1182); "Arch Dams: Construction of the Kamishiiba Arch Dam," by K. M. Mathisen and C. C. Bonin (Proc. Paper 1183); and "Arch Dams: Review of Experience," by Robert E. Glover (Proc. Paper 1217).

Records

A Review of Experience with the older Arch Dams was made a special feature of the Symposium on Arch Dams which was held at Knoxville, Tennessee during the week of June 4-8, 1956. It was the purpose of this review to present records of the performance of some of the older Arch Dams to learn how well they had served the purposes of their designers and to profit from these experiences to the end that new designs might be improved. Records were contributed generously for this review by Engineers from many countries. Due to the kindly assistance received from the New Zealand Institution of Engineers and from the Institution of Engineers of Australia, the New Zealand, Australia, and Tasmania area is exceptionally well represented. This is fortunate since some of the longest records of all came from there.

As might be expected, there are many items of interest and value in these records. Among them are the records of the venerable dams from Italy and France. The experience with the "peripheral joint" in Italy. The records of two dams in Germany which were bombed out during World War II, records of the use of a curtain wall to protect thin dams from damage by frost as reported from Norway, experiences with alkali-aggregate expansion from the United States, records of arch dams with overflow crests from many countries, the records for two arch dams which are alleged to have failed, records of methods of design, of resistance to earthquake and many other items.

Presentation of Data

In order to present the many records in a concise and usable way as many items as possible have been tabulated. These are supplemented by extracts from the notes accompanying the records. The basic order of presentation is a geographical one which begins with Italy and ends with Tasmania. Each dam has been given a number in the table and this number appears also on the notes and on the pictures of the dams.

Summary

With records coming from so many countries it should be expected that there would be some language difficulties. It is not surprising therefore that Engineers in many cases interpreted the request for an experience record as a request for data on measurements. Although a number of dams, for this reason, lack a specific statement covering their behavior over the years it seems certain that they have served satisfactorily, since black and white photographs were sent in nearly all cases and these were commonly supplemented with recently prepared color slides for use during the Symposium meetings. One can not study these records without being impressed with the excellent performance of Arch Dams. All types designed by all methods have served well. In three cases they have continued to stand even after an abutment or a part of the foundation was lost. By so doing they have given a convincing demonstration of the ability of the arch type of dam to cope with adverse conditions.

The original records have been placed on file in the Society Library, where they may be consulted for additional details, if desired.

NOTES

1. Ponte Alto- Ownership—Italian Government
2. Madruzza

For description see Paper 1017 by Carlo Semenza. "During the big flood of 1882, the dam was overflowed by a 5 m head of water; it suffered some damages, especially at the toe. For this reason, in 1884-1886, another dam was built 180 m downstream; the arch-dam of Madruzza, 40 m high, built of big concrete elements."

3. Osiglietta - Accizierie E Ferriere Lombarde Falck-Milan, Italy
4. Rochetta
5. Giaredo
6. Ganda
7. Publino
8. Venina
9. Valla
10. Moledana

Reported by—Dr. Ing. Mario Scalabrini—Milano, Italy. (Director of Iron and Steel Company Falck—Hydroelectric Division) Corso Matteotti 6-Milano. 6 February 1956.

11. Corfino Ownership—Societa Elettrica Selt-Valdarno—Milano, Italy
12. Turrite Di Gallicano

An earthquake in 1920 completely destroyed the village of Villa-Collemandina but did not damage the Corfino Dam.

Stability of the Turrute Di Gallicano Dam has been excellent and no repairs have been necessary. The structure withstood several earthquakes, the most severe in 1920.

These reports compiled from the 1952 records of Associazione Nazionale Imprese Distributrici di Energia Elettrica—(ANIDEL).—by Mr. R. A. Sutherland.

13. Zola—Maintained and operated by the "Service hydraulique des Bouches du Rhone—France.

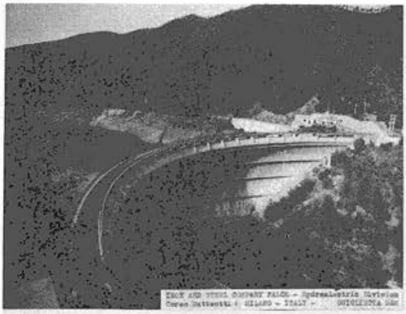
The Zola Dam is a masonry arch dam whose construction was commenced in 1850 and completed in 1854. The dam is constructed of selected dressed stone with the upstream face and downstream face carefully jointed, the

RECORDS OF EXPERIENCE

Number	1	2	3
Name	Ponte-Alto	Madruzza	Osiglietta
Location	Italy	Italy	Italy
Height	46.3 M	40.6 M	76.8 M
Top Thickness	4.0 M	3.2 M	5.8 M
Base Thickness	2.0 M	5.5 M	10.74 M
Crest Length	12.0 M	15.0 M	224.0 M
Completion Date	1611-1887	1886	1939
Temperature control during construction	.....	.....	.....
Final Closure	.....	.....	Grouting
Spillway	Overflow Crest	Overflow Crest	Overflow Crest and Spillway on L. Bank.
Abutment Rock	Limestone Red Scale	Limestone Red Scale	Porphiritic Gneiss
Method of Design	.....	.....	Elastic Arch and Models.
Purpose	.....	.....	Power
Ownership	Italian Government	Italian Government	Falck
Remarks	.....	.....	Dome Type with Cushion.

RECORDS OF EXPERIENCE

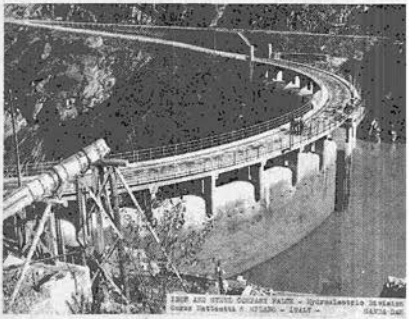
Number	4	5	6
Name	Rochetta	Giardo	Ganda
Location	Italy	Italy	Italy
Height	76.0 M	27.5 M	30.0 M
Top Thickness	3.5 M	3.0 M	3.0 M
Base Thickness	12.8 M	2.0 M	3.15 M
Crest Length	136.3 M	37.43 M	90.55 M
Completion Date	1937	1941	1947
Temperature control during construction	.....	.....	.....
Final Closure	Grouting	Grouting	Grouting
Spillway	2-6x5 M Sluices	Overflow Crest	Overflow Crest
Abutment Rock	Siliceous Sandstone	Old Red Sandstone	Phyllite
Method of Design	Elastic Arch and Models.	Elastic Arch	Elastic Arch
Purpose	Power	Power	Power
Ownership	Falck	Falck	Falck
Remarks	Single Curvature Arch.	Single Curvature with Cushion.	Single Curvature with Cushion.



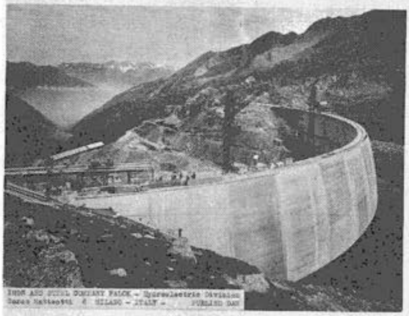
3 OSIGLIETTA

4 ROCCHETTA

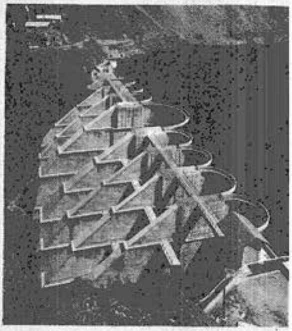
5 GIAREDO



6 GANDA



7 PUBLINO



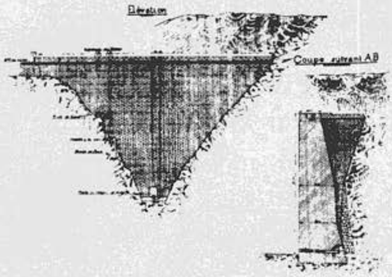
8 VENINA

RECORDS OF EXPERIENCE

Number	7	8	9
Name	Publino	Venina Lake	Valla
Location	Italy	Italy	Italy
Height	42.0 M	49.5 M	47.0 M
Top Thickness	2.8 M	0.95 M Cent. Arch 0.60 M Side Arch	1.0 M
Base Thickness	12.1 M	3.50 M Cent. Arch 1.50 M Side Arch	15.72 M
Crest Length	205.59 M	175.0 M	113.80 M
Completion Date	1951	1926	1925
Temperature control during construction	.....	.....	.....
Final Closure	Grouting	.....	None
Spillway	Glory Hole	Overflow Sill with Gates.	Overflow Crest
Abutment Rock	Gneiss	Quartziferous Phyllites and Crystalline Schists.	Serpentine
Method of Design	Tolke and Models	Elastic Arch	Elastic Arch
Purpose	Power	Power	Power
Ownership	Falck	Falck	Falck
Remarks	Double Curvature Arch - Gravity with Cushion.	Multiple Arch	Single Curvature Arch

9 VALLA

10 MOLEDANA



13 ZOLA

14 FURAN

15 MÖHNE

RECORDS OF EXPERIENCE

Number	10	11	12
Name	Moledana	Corfino	Turrite Di Gallicano
Location	Italy	Italy	Italy
Height	42.6 M	37.5 M	42.0 M
Top Thickness	2.0 M	1.5 M	3.20 M
Base Thickness	7.61 M	7.0 M	13.50 M
Crest Length	64.3 M	65.0 M	58.5 M
Completion Date	1931	1914	1916
Temperature control during construction	.....	.....	.....
Final Closure	Grouting	No joints	.....
Spillway	Overflow Crest	Siphons	Overflow Crest
Abutment Rock	Crystalline Schists	Serpentine	Grey Limestone
Method of Design	Elastic Arch	Separate Arches	Separate Arches
Purpose	Power	Power	Power
Ownership	Falck	Societa Elettrica Selt-Valdarno	Societa Elettrica Selt-Valdarno
Remarks	Single Curvature Arch	Earthquake Record. See Notes.	Earthquake Record. See Notes.

RECORDS OF EXPERIENCE

Number	13	14	15
Name	Zola	Furan	Mohne
Location	France	France	Germany
Height	140.0 ft.	208.0 ft.	40.3 M
Top Thickness	19.5 ft.	9.0 ft.	6.25 M
Base Thickness	43.0 ft.	174.0 ft.	34.2 M
Crest Length	282.0 ft.	354.0 ft.	650.0 M
Completion Date	1854	1866	1918
Temperature control during construction	.....	.....	.....
Final Closure	.....	.....	.....
Spillway	Channel at Side	Channel at Side	Overflow Weir
Abutment Rock	Hard Limestone	Compact Granite	Shale
Method of Design	Not Known	.....	Gravity Action Alone
Purpose	Water Supply	Water Supply	Water Supply and Power
Ownership	See Notes	Service Des Ponts and Chausees De La Loire	Rurtalsperrenverein
Remarks	Built of Selected Dressed Stone. Reported in perfect condition in 1933.	Constructed of undressed stone with dressed stone faces - bedded in lime mortar. Seepage has decreased from 1.1 to 0.03 gallons per second.	Bombed in 1943 - repaired in 4 months time. Twelve years successful service since. See notes for additional detaile.

upstream face is vertical while the slope of the downstream face is variable. The foundations are a good quality hard limestone and the dam was constructed without cutoff, drains or impervious curtains in the foundations. In consequence there are certain seepage losses under the foundations estimated to be of the order of 10 gallons/sec. when the reservoir is full.

The spillway is a side channel 26' long cut in the right bank abutment. The discharge channel has an extremely steep slope. With a freeboard of only 4 ft. the spillway capacity is insufficient and severe floods sometimes discharge over the crest of the dam.

The intake for the Zola canal is on the right bank 49' below crest level and is controlled by a penstock gate 2' 9" x 3' 0" hand operated with great difficulty by a direct acting winch. Scouring was provided for by two scour valves, one discharging through the right bank abutment 16' 6" below the canal intake, the other through a 19" scour pipe located in a scour gallery situated on the axis of the dam at the base of the wall. Neither of these valves is however serviceable today, as the reservoir has silted up to about 7 feet below the intake level.

Two reports on the dam are available and are quoted below:

1. Note of M. le Sous-Ingeniour des Ponts et Chaussees Coneste on the Zola Dam dated 18th May 1918 (Records of the Department of Ponts et Chaussees, et Aix).

"The Zola Dam continues to behave well. During my last visit I noticed serious streaking over part of the face of the dam due to seepage through cracks and when the dam is full some seepage is visible on the downstream face in the neighbourhood of the scour gallery, which can no longer be used. I have sunk a gauging pit downstream of the structure and this has enabled me to estimate the losses for the whole basin at approximately 15 gallons/sec. but it must be added that the greater part of this takes place through the foundation rock. Not more than 3/4 gal/sec. should pass through the wall itself and there is nothing to indicate that this does not take place at the abutments.

This structure dates from 1853; it does not appear to have suffered any deformation due to temperature effects or to overtopping during severe storms."

2. Report dated May 1933 for the International Commission on Large Dams.

"The dam is still in perfect condition. The losses mentioned in the above note do not seem to have the importance indicated. The dam was emptied during the year 1921 and the upstream face was found to be everywhere in good condition. The intake gate was repaired at that time but should have been replaced by one of a modern type. At the same time a series of scales were placed on the wall to enable water depths to be read.

All in all in spite of its thin section the Zola Dam, doubtless because of its arch form and the solid support given by foundations and abutments, is thoroughly stable and this is confirmed by 80 years of perfect functioning without incident."

Contributed by Mr. J. Martin, Member of the Program Committee—18 November 1955.

14. Furan

"The Furan Dam is a masonry gravity dam constructed in a gorge on the Furan known as the Gulf of Hell, with which name the dam is often associated. It was commenced in 1863, completed in 1866 and is used as a reservoir for supplying water to the town of St. Etienne in the Department of the Loire. It is maintained and operated by the "Service des Ponts et Chaussees" of the "Department de la Loire."

The dam is constructed of undressed stone extracted from the deviation canal and bedded in lime mortar. The upstream and downstream faces are constructed with variable slopes out of dressed stone. The dam is founded on a hard compact granite. A deviation canal, 5560 ft long, with a capacity of 3510 cusecs commences upstream of the basin of the dam and follows generally the line of top water-level, discharging into the river below the wall. A system of control gates situated in this canal enables floods to be discharged either into the dam or into the river downstream. Additional control measures consist of a discharge tunnel driven in the right bank abutment rock controlled by a valve discharging into the discharge canal, and a right bank side spillway 65 ft long. Intake and scour are provided for by two 15 3/4" pipes and one 8 1/2" pipe drawing off 26.2 ft above the reservoir bottom. The name of the dam, "the Gulf of Hell," has apparently no connection with the possible ultimate destination of its engineers as it has always behaved admirably since its construction. Slight seepage occurs through the downstream face but these losses which were originally 1.1 gals/sec have now dropped to 0.03 gals/sec when the reservoir is full."

Contributed by Mr. J. Martin, Member of the Program Committee—18 November 1955.

15. Mohne—Ownership—Rurtalsperrenverein—Germany
16. Eder—Ownership—Wasserstrombauverwaltung—Germany

The Eder and Mohne dams were bombed in 1943 with the result that a block of about 20 meters in height and 70 m in length with parabolic shape was completely destroyed and swept away in the middle part of the dams. On both sides of the destroyed parts many cracks and fissures appeared.

In the Eder Dam it was evident that there was some arch action before the block was swept away. This could be proved by the displacements in the region of the crest of the dam. Both dams were reconstructed within 4 months by a masonry plug. It was necessary, of course, to remove on both sides masonry masses in very bad state, so that the breadth of the plugs exceeded by 20 meters that of the hole. The many cracks and fissures remaining were artificially closed and grouted. Both dams were in full service since 12 years and proved practically safe and water-tight.

Contributed by Dr. Ing F. Tolke, Direktor Otto-Graf-Institutes—Stuttgart, Germany.—April 4, 1956.

17. Langli Dam—Ownership—Oslo Vann-og Kloakkvesen—Norway.

The dam has been in service only for 15 years. No sign of damage has occurred on the dam during these years. Some small cracks which occurred in the limit between rock and concrete in the beginning and where we got some moisture on the downstream side, have tightened during the years. In the left abutment we very soon got a vertical crack. This crack has never closed. The dam at the crack is about 16 m high. There is air on both sides of this part of the abutment so there is no leakage through the crack. It shows however that this part of the abutment does not act as an abutment at all. Downstream 0.8 m from the arch is placed the insulating wall to protect the dam against frost damage in winter when the temp. may go down to -25 to 30°C. The insulating wall is 15 cm (6") thick and is made of reinforced concrete which has, of course, very small insulating ability. The important thing is, that we are getting a space between the dam and this wall with still air and it prevents frost damage. Without this wall we certainly would have got frost damage. This is clearly shown on a lot of small intakes and other constructions where the insulating wall is neglected. The rock is tight and no leakage is observed.

The oldest of the arch dams in Norway is the dam at Nedre Skjerkavann which serves as intake dam for the Skjerka Hydro Electric Power Plant. The dam has a height of 16 m over the whole length. The radius is 54.5 m. The dam is made of reinforced concrete and completed 1932. Insulating wall of reinforced concrete 15 cm thick. The dam site is on the level 605 meters. There is no frost damage on the dam but for the plaster on the overflow which has had to be renewed in some places. Experience shows that plaster is not satisfactory on dams or other water constructions. Other arch dams, from the years before and after the Second World War, give the same experience. The dams are poured with a concrete mix of 350-400 kg cement to each m3 concrete. In the insulating walls we only use 300 kg cement. On account of the low temperature during the winter and the acid water in the Norwegian rivers it is not advisable to have lower cement content in our dams without getting bad damage."

Contributed by Ingenior Chr. F. Groner, Oslo, Norway—November 26, 1955.

18. Walters—Ownership—Carolina Power and Light Company.

"Spillway performance has been remarkably satisfactory. There has been little concrete maintenance to date. However, downstream face has been damaged by freezing and thawing action. Also, there are signs of concrete growth; this will receive more study in the near future."

Contributed by Mr. R. W. Gunwaldsen, Hydraulic Engineer, Ebasco Services. March 3, 1955.

19. Angostura—Ownership—Property of the Nation—Mexico.
20. Calles
21. Pabellon

Reports prepared by Mr. Mariano Trejo, Civ. Eng., Assistant of the Engineer in Chief of Irrigation—October 1955.

22. Colimilla—Onwership—New Chapala Electric Company, Mexico.

Report prepared by Eng. Eduardo Rojas G.—October 1955.

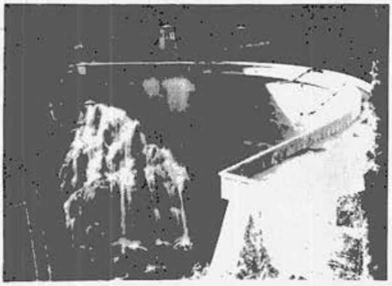
These Mexican records were submitted by Mr. Aurelio Benassini, Engeniero en Jefe de Irrigaçion y Control de Rios—Secretaria de Recursos Hidraulicos.—Through Mr. Julian Hinds, Member of the Program Committee.

23. Pacoima—Ownership—Los Angeles County Flood Control District.

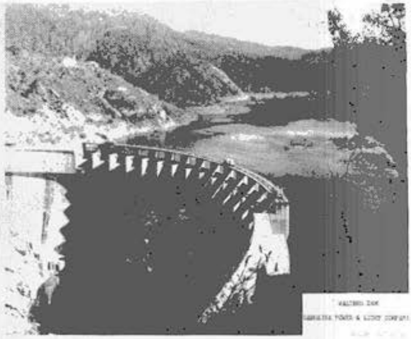
"Open steel penstocks installed in dam during construction, gates installed on penstocks at completion. Central section concrete pours always lower than mean elevation of finished concrete arch during construction. Reservoir has been full to 11.7 feet above lower spillway crest. Some measurable seepage has occurred at abutments. Abutments have been grouted to some extent on 9 different occasions. No evidence of cracks or deterioration of

RECORDS OF EXPERIENCE

Number	16	17	18
Name	Eder	Langli-Dam	Walters
Location	Germany	Norway	U. S.
Height	48.0 M	37.1 M	207.0 ft.
Top Thickness	5.8 M	0.70 M	16.0 ft.
Base Thickness	36.5 M	1.70 M	40.5 ft.
Crest Length	400.0 M	65.0 M	800.0 ft.
Completion Date	1914	1940	1930
Temperature control during construction	.....	.....	.....
Final Closure	.....	.....	.....
Spillway	Overflow Weir	Overflow	Overflow Crest
Abutment Rock	Shale	Granite	Quartzite
Method of Design	Gravity Action Alone	Trial-Load	Separate Arches
Purpose	Flood Control - Water Supply - Navigation Power	Water Supply	Power
Ownership	Wassestrombauver-Waltung	Oslo Van-Og Kloakkvesen	Carolina Power & Light Company
Remarks	Bombed in 1943 - repaired in 4 months time. Twelve years successful service since. See notes for additional details.	Protected against frost damage by an insulating wall enclosing an air space. No sign of damage in 15 years of service.	Spillway performance remarkably satisfactory. Little concrete maintenance. Some frost damage and signs of concrete growth. See notes.

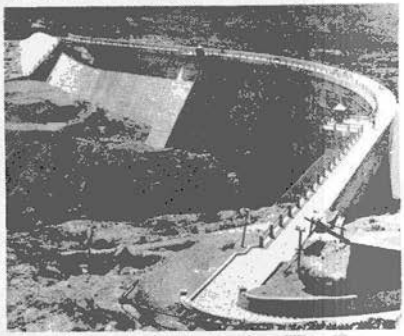


17 LANGLI

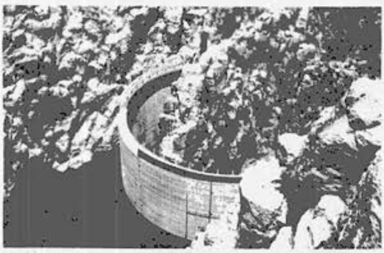


18. WALTERS

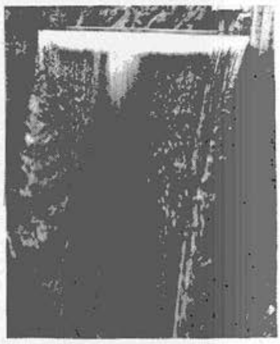
19 LA ANGOSTURA



20 CALLES



21 PABELLON



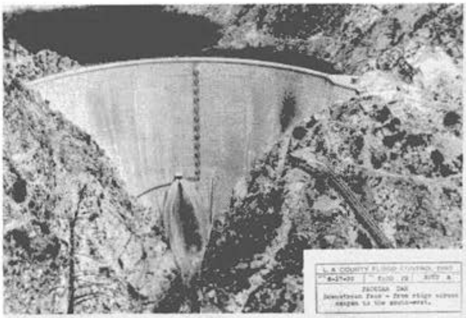
22 COLIMILLA

RECORDS OF EXPERIENCE

Number	19	20	21
Name	Angostura	Calles	Pabellon
Location	Mexico	Mexico	Mexico
Height	302.0 ft.	220.0 ft.	105.0 ft.
Top Thickness	11.5 ft.	9.8 ft.	3.94 ft.
Base Thickness	101.7 ft.	36.0 ft.	8.20 ft.
Crest Length	584.0 ft.	918.0 ft.	246.0 ft.
Completion Date	1942	1931	1932
Temperature control during construction	Low Heat Cement- Embedded Pipe Cooling	.....	.....
Final Closure	Grouting	.....	.....
Spillway	Overflow	Overflow	Overflow Crest
Abutment Rock	Riolita	Riolita	Riolita
Method of Design	Trial-Loads	Trial-Loads	.....
Purpose	Flood Control, Power, Irrigation.	Power, Irrigation.	See remarks.
Ownership	Property of the Nation.	Property of the Nation.	Property of the Nation.
Remarks	.....	.....	Purpose - Deviation for the Calles Dam.

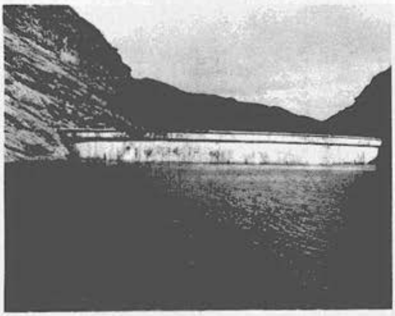
RECORDS OF EXPERIENCE

Number	22	23	24
Name	Colimilla	Pacoima	Gene Wash
Location	Mexico	U. S.	U. S. Calif.
Height	321.0 ft.	372.0 ft.	138.0 ft.
Top Thickness	11.5 ft.	10.4 ft.	5.0 ft.
Base Thickness	32.8 ft.	99.2 ft.	26.5 ft.
Crest Length	.....	640.0 ft.	430.0 ft.
Completion Date	1949	1929	1938
Temperature control during construction	None	None	Yes
Final Closure	Grouting in upper parts - Closure plug in lower parts.	.....	Grouting
Spillway	Overflow Crest	Tunnel with chute	Off Channel
Abutment Rock	Riolita	Grano-Diorite	Red Sandstone
Method of Design	Separate Arches	Trial-Load	Trial-Load
Purpose	Power	Flood Control, Water Conservation.	Water Supply
Ownership	New Chapala Electric Company	Los Angeles County Flood Control District	Metropolitan Water District of Southern California
Remarks	.....	Some seepage at abutments. Abutments grouted on 9 occasions. No evidence of cracks or deterioration of concrete in structure.	Service experience similar to Copper-Basin but effects of expansion somewhat more moderate. No significant structural impairment.



23 PACOIMA

24 GENE WASH



25 COPPER BASIN

26 ROOSEVELT

27 HORSE MESA

28 MORMAN FLAT

RECORDS OF EXPERIENCE

Number	25	26	27
Name	Copper Basin	Roosevelt	Horse Mesa
Location	U. S. Calif.	U. S. Arizona	U. S. Arizona
Height	210.0 ft.	280.0 ft.	300.0 ft.
Top Thickness	5.0 ft.	16.0 ft.	8.0 ft.
Base Thickness	35.0 ft.	184.0 ft.	45.0 ft.
Crest Length	254.0 ft.	723.0 ft.	660.0 ft.
Completion Date	1938	1910	1927
Temperature control during construction	Yes	None	None
Final Closure	Grouting	.....	.....
Spillway	Concrete Ogee	Side Channel	Two Side Channels and One Tunnel.
Abutment Rock	Red Sandstone	Sandstone	Rhyolite
Method of Design	Trial-Load	Gravity	Trial-Load
Purpose	Water Supply	Power, Irrigation	Power, Irrigation
Ownership	Metropolitan Water District of Southern California	Salt River Project - Arizona	Salt River Project - Arizona
Remarks	Alkali-aggregate expansion Upstream movement of 5.5 inches at top center of dam. Expansion has ceased. Concrete strength good. No significant structural impairment.	This dam has required practically no maintenance.	Service experience good.

concrete in structure. All construction joints are tight."

Report prepared by Mr. Paul Baumann, Assistant Chief Engineer, Los Angeles County Flood Control District.—February 6, 1956.

24. Gene Wash--Ownership: The Metropolitan Water District of Southern California.

25. Copper Basin

"The service experience of Gene Wash dam is generally similar to that of Copper Basin dam, although the effects of expansion in the concrete appear to be somewhat more moderate.

Observations of deflection and elevational changes have been made on Gene Wash dam, but have not been continued as systematically as in the case of Copper Basin dam. For information relating to performance of concrete in Gene Wash dam, reference may be made to the reports listed in the supplementary statement for Copper Basin dam.

As in the case of Copper Basin dam, no significant structural impairment of Gene Wash dam has been detected.

The aggregates used in the concrete in Copper Basin dam were derived from deposits in the bed of the Bill Williams River, in common with the aggregates for Parker and Gene Wash dams. Subsequent developments including comprehensive laboratory investigations, disclosed that these aggregates contained a significant proportion of material reactive to cement, or more particularly to the alkali in cement. The cement used in these dams was purchased under specifications which did not include special limitation of the alkali content, and was moderately high in alkali. The reactivity of the aggregates resulted in the formation of a silica gel wherever there was sufficient residual moisture in the concrete, accompanied by a slow but persistent increase in volume of the interior mass of concrete. The dryer concrete at and just below the surfaces above water was disrupted by the expansion of the interior mass, giving rise to the familiar random-pattern cracking. These cracks, a few of which were up to 1/2 of an inch in width at the surface, rarely extended more than a foot or so into the concrete.

Placement of concrete in Copper Basin dam was completed May 10, 1938, grouting of the vertical joints between the blocks was completed December 3, 1938, and filling of the reservoir with water was started January 29, 1939. Immediately following completion of the dam a point, designated P-2, was set on the parapet near the center of block C, and near the center of the uppermost arch, and a line of sight designated reference Line P, was established between points on the rock near each abutment.

The maximum downstream position of point P-2 was observed March 3, 1939, after the reservoir was full. The radial movement of point P-2 during the reservoir filling was 0.22 of an inch, which is consistent with the estimated deflection of the top arch due to water loading on the dam.

Sporadic observations on point P-2 were continued, and an unexpected upstream movement was recorded, which by October 1942 had attained a magnitude of 2.35 inches from the farthest downstream position under water load as observed March 3, 1939. During 1942 a plumb line was erected on the downstream face of the dam and points were set in the face for measurement of deflections at elevations 1024, 1000, 980, and 960, as well as at the parapet at elevation 1041.5. Access to the plumb line is by a ladder erected on the face of the dam. The plumb line weight is suspended in a receptacle full of oil, to dampen vibration. A comprehensive survey of reference points was made between October 21 and 26, 1942. Systematic observations have been made at regular intervals beginning October 1942.

The total movement of point P-2 from its maximum downstream position under water load on March 3, 1939, to its maximum upstream position during recent summers, amounts to 5.5 inches. The annual increment was practically constant from 1939 to 1946 at 0.54 of an inch per year. From 1946 to 1952 annual increment decreased, and there has been no cumulative movement whatever since 1952.

Tests of cores cut from Copper Basin and Gene Wash dams during the period between 1940 and 1950 show that no appreciable retrogression in strength or elastic modulus of the concrete has occurred in the two dams. No evidence of significant structural impairment has been discovered."

Contributed by Mr. Robert B. Diemer, General Manager & Chief Engineer, The Metropolitan Water District of Southern California—September 15, 1955.

26. Roosevelt—Salt River Project Agricultural Improvement and Power District.—Arizona
27. Horse Mesa
28. Mormon Flat
29. Stewart Mountain
30. Bartlett

"Roosevelt dam has required practically no maintenance.

There have been no problems experienced with Horse Mesa dam. There is some evidence of alkali aggregate reaction, but there has been no appreciable movement.

There have been no structural problems with Mormon Flat dam.

Alkali aggregate reaction has occurred in the concrete of Stewart Mountain dam. Expansion of the concrete has caused the top of arch in the central portion to move 5.5" upstream. The dam has separated from the powerhouse. An additional mass of concrete has been placed on the downstream face of one abutment to provide reinforcement. Additional grouting of the dam has been done to seal up cracks and construction joints to reduce flow of water past the face of the dam. The object was to reduce the amount of water in the mass and thus the rate of alkali aggregate reaction. Rate of growth has decreased during the last few years.

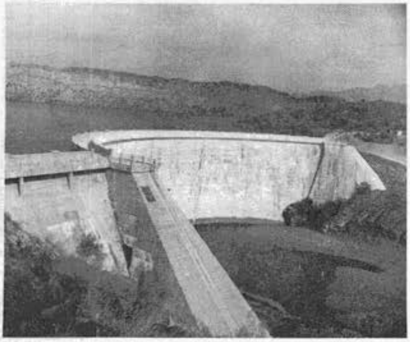
Reports by Mr. C. H. Whalin—Supervisory Engineer, Civil Division—Salt River Project Agricultural Improvement and Power District.—February 10, 1956.—Transmitted by Mr. T. M. Morong, Chief Engineer to Mr. Julian Hinds, Member of Program Committee.

31. Arrowrock—Ownership—United States Government.
32. Buffalo Bill (Shoshone)
33. Clear Creek
34. Deadwood
35. Gerber
36. Gibson
37. Hoover
38. Owyhee
39. Parker
40. Pathfinder

Arrowrock dam was built of sand-cement concrete which had very low frost resistance qualities. Downstream face was badly damaged by frost

RECORDS OF EXPERIENCE

Number	28	29	30
Name	Mormon Flat	Stewart Mountain	Bartlett
Location	U. S. Arizona	U. S. Arizona	U. S. Arizona
Height	224.0 ft.	207.0 ft.	283.0 ft.
Top Thickness	8.0 ft.	8.0 ft.	2.34 ft.
Base Thickness	20.0 ft.	36.0 ft.	7.0 ft.
Crest Length	380.0 ft.	1260.0 ft.	800.0 ft.
Completion Date	1925	1930	1939
Temperature control during construction	None	None	Spray Cooling
Final Closure	.....	Grouting	.....
Spillway	Side Channel and Chute	Side Channel	Chute
Abutment Rock	Rhyolite	Gravity Buttresses	Concrete Buttresses
Method of Design	Separate Arches	Trial-Load	.....
Purpose	Power, Irrigation	Power, Irrigation	Irrigation
Ownership	Salt River Project - Arizona	Salt River Project - Arizona	Salt River Project Arizona
Remarks	Service experience good.	Alkali-aggregate expansion. Upstream movement of 5.5 inches at top center of dam. Repairs by grouting and strengthening of buttress. Rate of growth has decreased in last few years.	Multiple arch. No leakage. No maintenance.

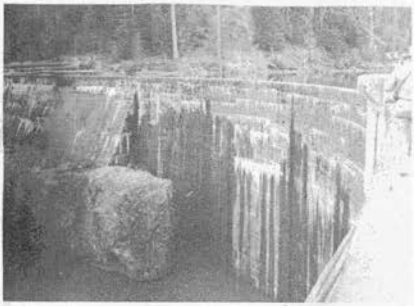


29 STEWART MTN

30 BARTLETT

31 ARROWROCK

32 BUFFALO BILL



33 CLEAR CK.

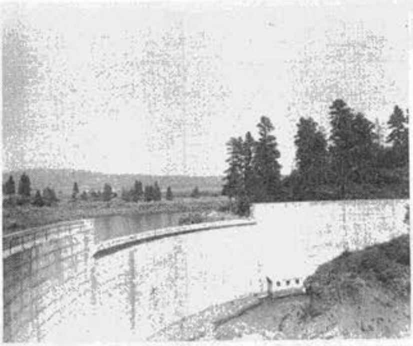
34 DEADWOOD

RECORDS OF EXPERIENCE

Number	31	32	33
Name	Arrowrock	Buffalo Bill (Shoshone)	Clear Creek
Location	U. S. Idaho	U. S. Wyoming	U. S. Washington
Height	354.0 ft.	325.0 ft.	80.0 ft.
Top Thickness	16.0 ft.	10.0 ft.	3.0 ft.
Base Thickness	223.0 ft.	108.0 ft.	10.0 ft.
Crest Length	1150.0 ft.	200.0 ft.	404.0 ft.
Completion Date	1915	1910	1914
Temperature control during construction	None	None	None
Final Closure	Concreted Shafts	.....	.....
Spillway	Side Channel	Concrete Weir and Tunnel	Overflow Rock Cut Channel
Abutment Rock	Granite	Granite	Igneous Rock and Shale
Method of Design	.....	Trial-Load	Cylinder Formula
Purpose	Water Supply	Water Supply and Power	.....
Ownership	U. S. Government	U. S. Government	U. S. Government
Remarks	Built of sand-cement concrete. Frost damage. New face slab constructed 1937 and dam raised 5 feet. Face slab cracked in 1950.	Concrete in excellent condition in 1948.	Built to height of 50 feet in 1914. Increased 21 feet in 1918. Dam leaks through construction joints. Concrete is sound.

RECORDS OF EXPERIENCE

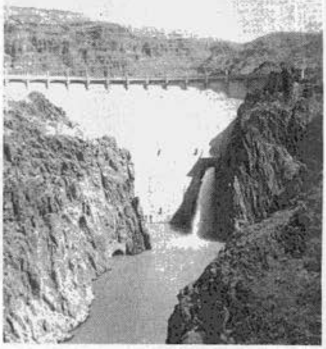
Number	34	35	36
Name	Deadwood	Gerber	Gibson
Location	U. S. Idaho	U. S. Oregon	U. S. Montana
Height	165.0 ft.	88.0 ft.	195.5 ft.
Top Thickness	9.0 ft.	5.0 ft.	15.0 ft.
Base Thickness	62.0 ft.	17.85 ft.	87.0 ft.
Crest Length	749.0 ft.	485.0 ft.	960.0 ft.
Completion Date	1931	1925	1929
Temperature control during construction	.....	None	None
Final Closure	Grouting	Closure Plugs	Grouting
Spillway	Overflow Section	Overflow Weir at Center	Glory Hole
Abutment Rock	Granite	Lava Flows and Conglomerate	Limestone
Method of Design	Trial-Load	.....	Trial-Load
Purpose	Water Supply	Water Supply	Water Supply
Ownership	U. S. Government	U. S. Government	U. S. Government
Remarks	Some grout headers frozen at time of grouting in March 1931. Joints only partially grouted. Some leakage at joints. Frost damage. Spillway lip damaged by icicles.	Frost action on seepage water caused deterioration of concrete. One closure plug shows disintegration of concrete.	Dam in good condition in 1948. Minor seepage through a few joints.

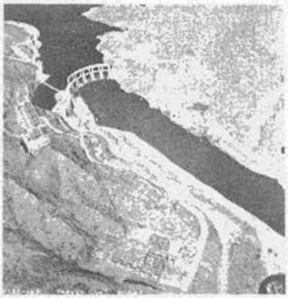


35 GERBER

GIBSON

HOOVER





Number	37	38	39
Name	Hoover	Owyhee	Parker
Location	U. S. Nevada-Arizona	U. S. Oregon	U. S. Arizona-California
Height	726.4 ft.	417.0 ft.	320.0 ft.
Top Thickness	45.0 ft.	30.0 ft.	39.5 ft.
Base Thickness	660.0 ft.	265.0 ft.	100.0 ft.
Crest Length	1244.0 ft.	833.0 ft.	856.0 ft.
Completion Date	1934	1932	1938
Temperature control during construction	Embedded Pipe Cooling System.	None	Embedded Pipe Cooling System.
Final Closure	Grouting	Grouting	Grouting
Spillway	Side Channel	Glory Hole	Overflow Crest
Abutment Rock	Andesite and Latite Breccia	Felsitic, Rhyolite and Glassy Lavas	Gneiss Granite
Method of Design	Trial-Load and Models	Trial-Load	Trial-Load
Purpose	Flood Control, Water Supply, Power	Water Supply	Flood Control, Water Supply, Power
Ownership	U. S. Government	U. S. Government	U. S. Government.
Remarks	Dam in good condition.	Minor weathering at crest. Evidence of alkali reaction. Mass concrete reported good.	Alkali-aggregate action present. Slab of concrete broken off during foundation grouting operations. Concrete badly cracked in some places.

action from freezing spray from outlet valves. New face was built in 1937 when height of dam was increased 5 feet. The inspection in 1950 disclosed that the new face slab was cracked badly in some places.

Buffalo Bill dam was built during the period 1905-1910, of cyclopean rubble concrete. This dam and Pathfinder dam were designed by trial-load procedures. From a structural standpoint there is nothing in the history of the dam to indicate any weakness or abnormality of behavior of the arch. Rock slides have damaged some of the appurtenant structures but no harm has been done to the dam.

In 1928 the downstream face of the dam was cleaned of scale, and the concrete was reported to be in excellent condition at that time. Concrete in dam is in excellent condition (1948 report).

Clear Creek dam was built to height of 50 feet in 1914. Additional 21 feet added in 1918. No record of joint treatment. Photographs in Report of April 6, 1925, by W. L. Rowe show joints between arch and thrust blocks but no joints in arch section. Dam leaks through cold joints. Leakage has increased between 1925 and 1948, as shown by photographs.

Recent inspections of the dam disclose that the concrete is fairly sound. There are numerous leaks at the horizontal construction joints and the downstream face has weathered badly in places. The seepage is worst at the construction joint at the original crest.

Deadwood dam was constructed during the period from September 1929 to June 1931. At the time the structure was completed there was an unusually light fall of snow and since it was desirable to catch as much of the early runoff as possible, water was stored several months earlier in the season than had been planned originally. This required that the grouting be performed during February and March of 1931, when the weather was unduly severe. Some of the headers to the grouting system were frozen and attempts to thaw them open were only partially successful. The result was that contraction joint grouting was incomplete. Leaks developed through some of the radial contraction joints and also through some of the horizontal construction joints. In 1938 an attempt was made to stop the seepage by regrouting the contraction joints.

The concrete in Deadwood dam has suffered from frost action. The curbs on top of the dam and the concrete in the wet areas have been badly damaged by frost action. The spillway lip has been damaged by the formation of massive icicles, which increase in weight until they spall large areas of concrete. The roof of the valve house has been damaged similarly. Repairs to the curbs on top of the dam and to the spillway lip were made in 1947 and again in 1953.

Gerber dam was built during the period 1923-1925. Because of the difficulty of obtaining adequate natural materials for concrete, it was necessary to manufacture sand. The use of manufactured sand resulted in concrete which was of low strength in certain portions of the dam.

After the dam was completed and placed in service, seepage developed through horizontal construction joints. This seepage, coupled with freezing weather during winter months, caused considerable disintegration of the concrete. In the fall of 1951, the water surface in the reservoir was lowered and the upstream face of the dam was covered with a waterproof membrane, which eliminated most of the seepage.

The dam was built with closure plugs between the arch and gravity sections. The concrete in the plug at the left end of the arch shows considerable deterioration in places.

Gibson dam was constructed during the period 1926-1927.

The condition of the dam is good. There are only a few minor seeps. There are no structural deficiencies.

Hoover dam was constructed during the period 1930-1935.

Observations for uplift pressure on the base of the dam were made periodically as the reservoir filled. These observations indicated that the uplift pressure beneath the Nevada side of the dam was increasing and that it exceeded that used as a basis for design. Corrective measures were started in 1939 by grouting a deeper and more extensive cut-off curtain slightly downstream from the axis of the dam. This work was followed by the establishing of a new system of foundation drains downstream from the additional grout curtain. Since the program of additional grouting and drainage was performed, there have been no further developments at Hoover dam affecting the behavior of the arch structure."

Owyhee dam was built during the period 1928-1932.

Shortly after the reservoir was filled, considerable leakage developed. The total flow was about 15 cubic feet per second, with most of this being through the left abutment and emerging downstream quite close to the dam. A program of supplemental grouting was started in April 1936 and completed in October 1937. The present flow from seepage is between 2 and 3 cubic feet per second.

Since the completion of the foundation grouting, there has been no further work at Owyhee dam. From a structural standpoint, the arch is in good condition although there is considerable cracking of the concrete. In the upper elevations recent inspections have disclosed that there is considerable evidence of alkali-aggregate reaction, which, combined with the effects of freezing and thawing, has caused disintegration of the curbs, walkways, parapets, and lamp post bases.

Parker dam was built during the years 1934-1938.

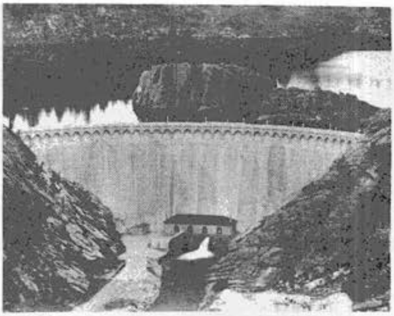
While grouting operations were in progress during the construction of the dam, a large slab of concrete, up to 18 inches thick, cracked loose from the face of the dam at the right abutment. This slab was removed and 183 cubic yards of new concrete was placed in the defective area. Dowels were placed in the older concrete to anchor the new concrete. Since that time numerous cracks have developed at various places in the structure, which have been attributed to the reaction of the high alkali cement with the aggregates.

The severely cracked condition of the exposed surface of the dam has been discussed quite extensively in technical writings on concrete design and structural behavior. Surface cracking became noticeable and of serious concern within a relatively short time after completion of the dam. An extensive investigation of the crack development was made in 1940 and 1941 and from this it was concluded that the cracking, although extensive, did not extend very deep. Cores drilled from the concrete when stored in fog room exuded gels, and when tested for compressive strength and modulus of elasticity gave somewhat lower values than expected. Samples of the concrete subjected to cycles of wetting and drying showed great expansive properties.

The alkali-aggregate reaction has caused sufficient expansion of the concrete to make operating the gates on top of the dam difficult, and to require considerable readjustment in order to obtain sufficient clearance. This work was completed in 1942. An inspection made in 1954 revealed no further difficulties with the arch structure except for some minor leaks.

Pathfinder dam is a masonry arch dam built during the period 1905-1909.

41 CHEESEMAN

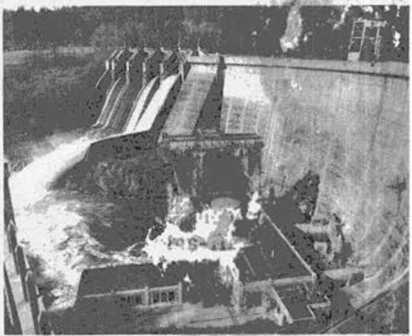


42 ELEVEN MI. CANY

Showing the unsupported end of arch dam and damaged spillway section.

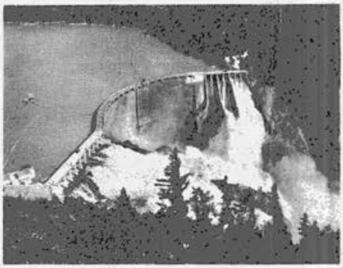
Taken up-stream from dam showing arch dam and depth to which spillway section and east abutment were washed away.

43 MOYIE



45 MERWIN

46 BAKER R.



47. DIABLO

RECORDS OF EXPERIENCE

Number	40	41	42
Name	Pathfinder	Cheesman	Eleven Mile Canyon
Location	U. S. Wyoming	[illegible]. S. Colorado	U. S. Colorado
Height	214.0 ft.	[illegible].0 ft.	142.0 ft.
Top Thickness	10.9 ft.	18.0 ft.	15.0 ft.
Base Thickness	96.5 ft.	167.0 ft.	53.1 ft.
Crest Length	432.0 ft.	634.0 ft.	496.0 ft.
Completion Date	1909	1904	1932
Temperature control during construction	None	None	None
Final Closure	.....	None	Grouting
Spillway	Weir at North End of Dam	Around End	Around End
Abutment Rock	Granite	Granite	Biotite Granite
Method of Design	Trial-Load	Gravity	Trial-Load
Purpose	Water Supply	Water Supply	Water Supply
Ownership	U. S. Government	City and County of Denver	City and County of Denver
Remarks	Dam in good condition. Faces of Ashlar Masonry. Cyclopean Masonry in the Interior.	No cracks-no leaks-no repairs. Masonry construction. Loads carried by Arch Action were estimated.	No cracks-no leaks-no repairs.

RECORDS OF EXPERIENCE

Number	43	44	45
Name	Moyie	Lake Lanier	Merwin (Ariel)
Location	U. S. Idaho	U. S. North Carolina	U. S. Oregon
Height	53.0 ft.	62.0 ft.	313.0 ft.
Top Thickness	2.0 ft.	2.0 ft.	19.4 ft.
Base Thickness	5.33 ft.	12.0 ft.	93.0 ft.
Crest Length	154.0 ft.	236.0 ft.	728.0 ft.
Completion Date	1924	1925	1931
Temperature control during construction	.....	.....	Cooling water in vertical holes.
Final Closure	.....	.....	Closure Plugs
Spillway	Timber lined cut to side stream.	Overflow Crest	Separate Overflow Section.
Abutment Rock	Stratified Quartzite dipping 30° to 45° downstream.	Folded Granitic Rock	Andesite and Basalt
Method of Design	.....	.....	Trial-Load
Purpose	Power and Milling	Impoundment for a pleasure lake.	Power
Ownership	Abandoned	.....	Pacific Power & Light Company
Remarks	Unprecedented flood destroyed spillway and destroyed east abutment. Dam still standing in 1954. Damage was never repaired. Project in state of abandonment.	Undetected foundation weakness caused loss of west abutment. Arch did not fail. Repaired by addition of gravity abutment, spillway section and addition of earth fills converting it to core type dam. It has successfully impounded water since its repair.	Dam remarkably tight at both vertical and horizontal construction joints.

RECORDS OF EXPERIENCE

Number	46	47	48
Name	Baker River	Diablo	Ross
Location	U. S. Washington	U. S. Washington	U. S. Washington
Height	285.0 ft.	389.0 ft.	540.0 ft.
Top Thickness	12.0 ft.	16.0 ft.	33.0 ft.
Base Thickness	120.0 ft.	146.0 ft.	208.0 ft.
Crest Length	450.0 ft.	1180.0 ft.	1300.0 ft.
Completion Date	1925	1930	1949
Temperature Control during construction	None	None	See notes.
Final Closure	Closure Plug	Grouting	Grouting
Spillway	Overflow Crest	Overflow Abutment Gravity Section	Overflow Crest with Deflector Hoods
Abutment Rock	Limestone	Granite Gneiss	Granite Gneiss
Method of Design	Trial-Load	Separate Arches	Trial-Load
Purpose	Power	Power	Flood Control, Power
Ownership	Puget Sound Power & Light Company	City of Seattle	City of Seattle
Remarks	Overflow spillway passed 21,712 cfs. in 1949. Spillway designed for 36,000 cfs.	In continuous service since completion except during construction joint grouting in 1950-1951.	In continuous service since completion. (This dam built in steps - See notes.)

The dam was constructed of cyclopean, coarse grained granite stones with a maximum size of 10 tons. These stones were bedded in cement mortar and the vertical joints filled with granite spalls. The ashlar facing stones were laid in horizontal courses with 2-inch maximum joints. As built, the dam contains 48 percent rock, 39 percent concrete, and 13 percent mortar. The stone for the masonry was quarried from the granite of the canyon walls and concrete aggregate was obtained from the river bed. The upper 27 feet of the dam was reinforced horizontally with steel. There is nothing in the history of Pathfinder dam which indicates any abnormality or deficiency in structural behavior. Repeated inspections of the structure have disclosed that the masonry is in excellent condition, with no leakage showing on the downstream face.

During the winter seasons, the operating personnel have always been bothered by accumulations of ice on the canyon walls downstream from the dam due to freezing of seepage water passing through the abutments. In 1949 a program of grouting the abutments was undertaken with the result that while the seepage was reduced it was not entirely eliminated. The grouting was successful for the reservoir elevation at the time the grouting was done, but the reservoir elevation increased in 1950 and new leaks appeared at the higher elevations in both abutments. No additional grouting has been done since 1949."

Data on Bureau of Reclamation dams supplied by Mr. L. N. McClellan, Assistant Commissioner and Chief Engineer.

Extract from a letter of April 2, 1956 from Mr. Wayne A. Perkins—Consulting Engineer.

Clear Creek dam was designed as a variable radius type, vertical at crown at the intrados, and vertical at the theoretical abutment at the extrados, this point being the contact between the arch and the gravity abutments. This criterion of design was set up to avoid overhang at any point. Stresses were determined by the cylinder formula. Limiting stresses set by the consulting board were 25 tons compression and 50 lbs tension.

After we moved in to start construction we discovered that the prepared design could not be used as the abutments were not adequate, necessitating a new design at a site farther upstream, which was made by the field crew on the ground and under pressure in order not to hold up the construction crew. When this dam was built the science of arch dam analysis was not well developed, and most of the earlier arches were designed by the cylinder formula which was used by Mr. D. C. Henny who supervised the first design and was a consultant throughout the construction.

41. Cheesman—Ownership—City and County of Denver.
42. Eleven Mile Canyon

Lake Cheesman dam was completed in 1904.

There have been no cracks, no leaks, no repairs except to outlet valves. There has always been a small seep through rock formation in canyon wall near end of dam.

Eleven Mile Canyon dam was completed on October 30, 1932. The contraction joints were grouted later. There have been no cracks, no leaks, no repairs, but should have new roadway paving and repair on face of highline due to frost action. There has always been a small seep through rock formation in canyon wall near end of dam.

Contributed by Mr. D. D. Gross, Consulting Engineer.

43. Moyie—Present ownership not determined.

Construction practically complete in 1924.

The abutment rock was stratified quartzite dipping sharply (30^0-45^0) downstream. Left abutment was in a sharp narrow ridge between the Moyie River and a side stream. The spillway was a timber lined cut 24 feet wide, 12 feet deep and 50 feet long discharging into the side stream on the left abutment.

According to a report by the U.S. Forest Service information reached their office about May 1, 1925 that three small dams had failed on the Moyie River in Canada due to high water. The resulting flood took out the highway bridge at Meadow Creek above the Cynide Gold Mining Company Project. The debris and flood water apparently tore out the timber lining of the spillway. Water cut out the entire spillway section and entirely destroyed the abutment at the east, left, end of the concrete arch dam. According to the report the arch section of the dam was apparently undamaged except for a crack near the east abutment.

Report prepared by Mr. John F. Mangan, Civil Engineer, Spokane, Washington.

44. Lake Lanier

Lake Lanier dam was completed in March 1925.

Extract from a letter of January 21, 1955, from Mr. Shannon Meriwether, Tryon, North Carolina.

"The dam was never a power dam; it simply impounded the water for a pleasure lake. It has successfully impounded the water since its repair.

Local inquiry has given me this information:

About two thirds of the original dam served as a spillway—there was no separate spillway. Failure was due to undermining at one end. It is said that this foundation failure resulted from the fact that this section rested on a large boulder—say forty feet across, instead of on hard bed-rock. Reports are that there were only a few borings. But even if borings had been carefully and thoroughly made, the boulder and its surrounding clay or detritus might not have shown up as such. In this locality the rock is very ancient, all granitic, and never stratified. Its folding, fracturing, and metamorphosis is almost beyond belief. At any rate, pressure from the impounded water loosened the material around the boulder and it moved, letting the section of the dam go out. I cannot determine how it moved, whether it sunk and overturned or just what did happen. Repair was made by dewatering that end of the dam and removing everything to a more certain bed-rock, and then building a new heavy gravity section, designed as a spillway, with a bridge over it. Then, as an added precaution, earth fill was added on both sides of that portion of the dam which had not failed, until the dam became in effect a core type dam."

Reference:

The October 14, 1926 issue of Engineering News-Record contains an account of the abutment failures at both the Moyie and Lake Lanier sites.

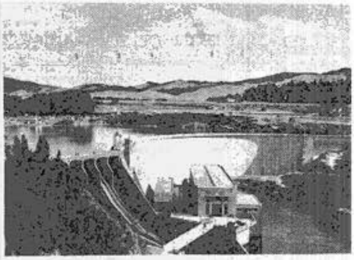
45. Merwin (Ariel)—Pacific Power and Light Company.

Merwin dam was completed on April 18, 1931.

"The dam was and is remarkably tight at both horizontal and vertical construction joints."

Report prepared by Mr. E. Robert de Luccia, Vice President and Chief Engineer, Portland, Oregon—March 7, 1955.

48 [illegible]



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50 MAR[illegible]

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53 MANORBURN

54 [illegible]

#1

RECORDS OF EXPERIENCE

Number	49	50	51
Name	Karipiro	Maraetai	Poolburn
Location	New Zealand	New Zealand	New Zealand
Height	170.0 ft.	284.0 ft.	100.0 ft.
Top Thickness	8.0 ft.	10.0 ft.	4.0 ft.
Base Thickness	50.0 ft.	50.0 ft.	30.0 ft.
Crest Length	576.0 ft.	436.0 ft.	400.0 ft.
Completion Date	1947	1952	1931
Temperature control during construction	Open slots	Embedded cooling pipes	4 ft. wide keyed slots
Final Closure	Slots filled	Grouting	Slots filled
Spillway	Separate Overflow Crest	Tunnel	Overflow Crest
Abutment Rock	Greywacke	Ignimbrite	Mica-Schist
Method of Design	Trial-Load	Separate Arches and Trial-Load	Cylindrical Arch
Purpose	Power	Power	Irrigation
Ownership	New Zealand Government	New Zealand Government	New Zealand Government
Remarks	Service experience - satisfactory.	Service experience - satisfactory.	.....

RECORDS OF EXPERIENCE

Number	52	53	54
Name	Fraser	Manorburn	Upper Cordeaux No. 1
Location	New Zealand	New Zealand	Australia, N.S.W.
Height	107.0 ft.	90.0 ft.	46.5 ft.
Top Thickness	2.5 ft.	3.0 ft.	3.5 ft.
Base Thickness	20.0 ft.	25.0 ft.	11.6 ft.
Crest Length	450.0 ft.	380.0 ft.	540.0 ft.
Completion Date	1937	1915	1902
Temperature control during construction	.....	None	.....
Final Closure	.....	.....	.....
Spillway	Overflow Crest	Overflow	Overflow
Abutment Rock	Slabby-Schist	Schist	Basalt
Method of Design	Cylindrical Arch	Cylindrical Arch	Thin Cylinder
Purpose	Irrigation	Irrigation	Water Supply
Ownership	New Zealand Government	New Zealand Government	Metropolitan Water, Sewerage & Drainage Board - Sydney.
Remarks	Service experience - satisfactory.	No details available.	No major repairs in 54 years of service.

46. Baker River—Puget Sound Power and Light Company.

Baker River dam was completed in 1925.

"The spillway of this arch type dam was designed with a discharge capacity of 36,000 c.f.s. It has been found to be adequate for any flood which has occurred during the past 25 years. The worst flood on record for this dam occurred on November 27, 1949 when 21,712 C.F.S. was recorded."

This report was prepared by Mr. A. L. Pollard, Operating Manager—Seattle, Washington.

47. Diablo—City of Seattle
48. Ross

Diablo dam was completed November 1930.

This dam has been in continuous service since completion except during construction joint grouting. Construction joint and foundation grouting and spillway paving to stop rock erosion was part of rehabilitation and improvement program in 1950 and 1951. Original design was based on the assumption that the arch rings carried the entire water load to the abutments with no transfer of load to foundation by cantilever or vertical beam action. A trial load analysis was made in 1949.

Ross dam was completed in June, 1949.

This dam was designed for construction in four steps. The First Step from foundation rock at elevation 1075 to elevation 1365. The Second Step to elevation 1550. The Third Step to elevation 1615. The Ultimate Height to elevation 1733. In construction the second and third steps were carried through in one continuous operation.

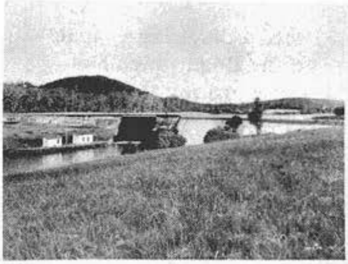
Report submitted by Mr. C. E. Shevling, Skagit Project Engineer, Seattle, Washington.

49. Karapiro—New Zealand Government.
50. Maraetai
51. Poolburn
52. Frazer
53. Manorburn

Karapiro dam was completed in April, 1947.
Service experience: Satisfactory—nothing unusual.
Maraetai dam was completed in December, 1952.
Service experience: Satisfactory, nothing to report.
Poolburn dam was completed in 1931.
Service experience: No details available.
Frazer dam was completed in 1937.
Service experience: Satisfactory.
Manorburn dam was completed in 1915.
Service experience: No details available.

Reports prepared by Mr. F. R. Askin, Chief Designing Engineer (Hydro). Hydro-electric Design Office, Ministry of Works, Wellington, New Zealand. 31.12.55.

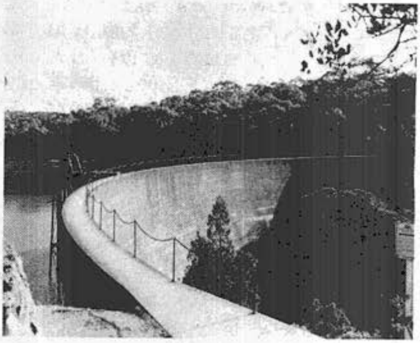
54. Upper Cordeaux No. 1. Metropolitan Water, Sewerage & Drainage Board, Sydney, N.S.W.
55. Upper Cordeaux No. 2.
56. Warragamba
57. Bargo.



55 UPPER CORDEAUX #2

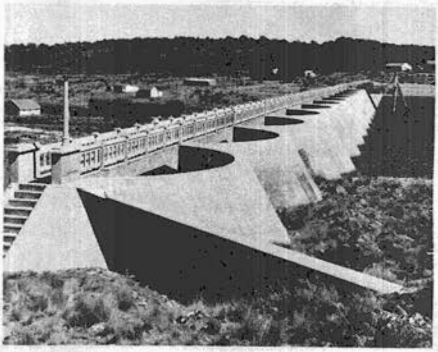
56 WARRAGAMBA

57 BARGO



58 BAROSSA

59. MT BOLD



71 MIENA

RECORDS OF EXPERIENCE

Number	55	56	57
Name	Upper Cordeaux No. 2	Warragamba Balance Reservoir	Bargo
Location	Australia, N.S.W.	Australia, N.S.W.	Australia, N.S.W.
Height	70.0 ft.	54.0 ft.	43.0 ft.
Top Thickness	3.5 ft.	3.0 ft.	4.0 ft.
Base Thickness	38.0 ft.	14.25 ft.	13.6 ft.
Crest Length	815.0 ft.	358.8 ft.	137.0 ft.
Completion Date	1915	1940	1897
Temperature control during construction	.....	None	.....
Final Closure	.....	Closure Plug	.....
Spillway	Overflow Spillway	Separate Side Spillway	Overflow
Abutment Rock	Sandstone	Sandstone	Sandstone
Method of Design	Thin Cylinder	Thin Cylinder	Thin Cylinder
Purpose	Water Supply	Water Supply	Water Supply
Ownership	Metropolitan Water, Sewerage & Drainage Board - Sydney.	Metropolitan Water, Sewerage & Drainage Board - Sydney.	Metropolitan Water, Sewerage & Drainage Board - Sydney.
Remarks	Structure sound. No major repairs have been necessary.	Slight seepages at joints. No remedial measures have been required.	In continuous use for 59 years. Some cracking and seepage in evidence but no major repairs are considered necessary.

RECORDS OF EXPERIENCE

Number	58	59	60	61
Name	Barossa	Mount Bold	Boreenore Creek	Puddle Dock Creek
Location	Australia South Australia	Australia South Australia	Australia, N.S.W.	Australia, N.S.W.
Height	112.75 ft.	177.0 ft.	53.5 ft.	61.0 ft.
Top Thickness	4.5 ft.	12.5, 22.0 & 25.0 ft.	3.5 ft.	3.5 ft.
Base Thickness	41.0 ft.	102.0 ft.	25.5 ft.	24.8 ft.
Crest Length	472.5 ft.	717.5 ft.	405.0 ft.	269.6 ft.
Completion Date	1903	1938	1928	1928
Temperature control during construction	No	None	.....	.....
Final Closure	No Contraction Joints	Grouting	.....	.....
Spillway	None	Dam Overshot	.....	.....
Abutment Rock	See Notes	See Notes	.....	.....
Method of Design	Separate Arches	Separate Arches	.....	.....
Purpose	Water Supply	Water Supply	.....	.....
Ownership	South Australia Government	South Australia Government	Department of Public Works, N.S.W.	Department of Public Works, N.S.W.
Remarks	Dam has given satisfactory service.	Dam has given satisfactory service.	Dam has given satisfactory service and is now in good condition.	Dam has given satisfactory service and is now in good condition.

RECORDS OF EXPERIENCE

Number	62	63	64	65
Name	Nattai Creek	Coeypoly Creek	Beardy River	Connors Creek
Location	Australia, N.S.W.	Australia, N.S.W.	Australia, N.S.W.	Australia, N.S.W.
Height	25.1 ft.	62.6 ft.	28.0 ft.	36.75 ft.
Top Thickness	3.5 ft.	3.5 ft.	4.0 ft.	3.5 ft.
Base Thickness	11.1 ft.	23.2 ft.	9.1 ft.	12.3 ft.
Crest Length	421.5 ft.	327.3 ft.	140.0 ft.	365.0 ft.
Completion Date	1931	1932	1932	1934
Temperature control during construction	.....	.....	.....	.....
Final Closure	.....	......	.....	.....
Spillway	.....	.....	.....	.....
Abutment Rock	Sandstone	Sedimentary	Granite	Diorite
Method of Design	.....	.....	.....	.....
Purpose	.....	.....	.....	.....
Ownership	Department of Public Works, N.S.W.	Department of Public Works, N.S.W.	Department of Public Works, N.S.W.	Department of Public Works, N.S.W.
Remarks	Dam has given satisfactory service and is now in good condition.	Dam has given satisfactory service and is now in good condition.	Dam has given satisfactory service and is now in good condition.	Dam has given satisfactory service and is now in good condition.

RECORDS OF EXPERIENCE

Number	66	67	68	69
Name	Flat Rock Creek	Mooney Creek	Back Creek	Greaves Creek
Location	Australia, N.S.W.	Australia, N.S.W.	Australia, N.S.W.	Australia, N.S.W.
Height	49.43 ft.	40.5 ft.	44.8 ft.	57.0 ft.
Top Thickness	3.5 ft.	3.5 ft.	3.5 ft.	3.5 ft.
Base Thickness	11.9 ft.	11.0 ft.	15.7 ft.	16.1 ft.
Crest Length	270.0 ft.	158.5 ft.	365.0 ft.	220.0 ft.
Completion Date	1935	1937	1937	1942
Temperature control during construction	.....	.....	.....	.....
Final Closure	.....	.....	.....	.....
Spillway	.....	.....	.....	.....
Abutment Rock	Sandstone	Shale and Sandstone	Shale	Sandstone
Method of Design	.....	.....	.....	.....
Purpose	.....	.....	.....	.....
Ownership	Department of Public Works, N.S.W.	Department of Public Works, N.S.W.	Department of Public Works, N.S.W.	Department of Public Works, N.S.W.
Remarks	Dam has given satisfactory service and is now in good condition.	Dam has given satisfactory service and is now in good condition.	Dam has given satisfactory service and is now in good condition.	Dam has given satisfactory service and is now in good condition.

Upper Cordeaux dam No. 1 was completed in 1902.

Apart from some leaching of mortar at the overflow section and calcium carbonate deposits at the non-overflow section, this dam is in very good condition after 54 years of service. No major repairs have been carried out. The reservoir is normally kept full.

Upper Cordeaux dam No. 2 was completed in 1915.

The reservoir is normally kept full. There is some leaching out of mortar on upstream face and at overflow section on downstream face; also some calcium carbonate deposits, but structure is quite sound and no major repairs have been found necessary.

Warragamba dam was completed in 1940.

This dam has been in service, mainly as a balance reservoir, on a 48-in. rising main for 16 years. Grouting of joints was provided for but was found to be unnecessary. There are some slight seepages through vertical and horizontal construction joints but no remedial measures have been required.

Bargo dam—The original completion date was 1897; raised 8-ft. 1910; raised further 7-ft. 1947. This dam has been in continuous use for 59 years. Some cracking and seepage are in evidence, but no major repairs are considered necessary.

Reports submitted by Metropolitan Water, Sewerage & Drainage Board, Sydney, N.S.W., Australia. March 1956.

58. Barossa—South Australia Government
59. Mount Bold

Barossa dam was completed in February, 1903.

"The Barossa dam is of particular interest as it was one of the earliest of the thin arch structures in Australia. The abutment rock is argillaceous and arenaceous laminated rock with micaceous shale joints. It was constructed without contraction joints. The dam has given satisfactory service."

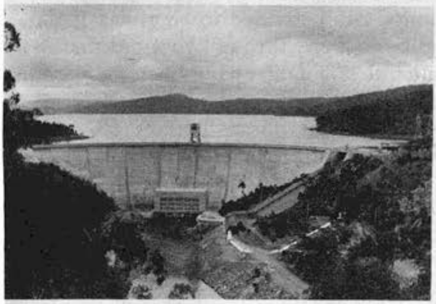
Mount Bold dam was completed in 1938.

"The abutment rock is Hard Phyllite with Limestone bands. The dam is of the constant radius arch type with gravity abutments. Puddled clay cut off walls extended from the concrete abutments into the river side slopes for a distance of 250 ft. on the south-easterly side and 150 ft. on the north-westerly side. Foundations grouted. The dam has given satisfactory service."

Report prepared by Mr. J. R. Dridan, C.M.G., B.E., A.M.I.E. Aust., Engineer-in-Chief, Engineering and Water Supply Dept., S.A. Govt., South Australia. May 28, 1956.

60. Boreenore Creek—Department of Public Works, N.S.W. Sydney, Australia.
61. Puddle Dock Creek
62. Nattai Creek
63. Coeypolly Creek
64. Beardy River
65. Connors Creek
66. Flat Rock Creek
67. Mooney Creek
68. Back Creek
69. Greaves Creek
70. Cudgegong River

"All these dams have given satisfactory service and are now in good condition and look like remaining in service for an indefinite future period.



72 CLARK

73 RIDGEWAY

RECORDS OF EXPERIENCE

Number	70	71	72	73
Name	Cudgegong River	Miena	Clark	Ridgeway
Location	Australia, N.S.W.	Tasmania	Tasmania	Tasmania
Height	56.6 ft.	40.0 ft.	199.0 ft.	195.0 ft.
Top Thickness	3.8 ft.	1.0 ft.	15.0 ft.	6.0 ft.
Base Thickness	25.0 ft.	1.9 ft.	80.0 ft.	53.75 ft.
Crest Length	465.0 ft.	1080.0 ft.	1110.0 ft.	232.75 ft. Arch 293 & 203 ft. Abutment
Completion Date	1953	1922	1949	1919
Temperature control during construction	.....	Nil	Cooling with River Water.	.....
Final Closure	.....	.....	Grouting	Closure Plug
Spillway	.....	Nil	Ski-Jump	Weir and Tunnel
Abutment Rock	Granite	Dolerite	Dolerite	Dolerite
Method of Design	.....	Separate Arches	Trial-Load	Not Known
Purpose	.....	Power	Power	Water Supply
Ownership	Department of Public Works, N.S.W.	Hydroelectric Commission of Tasmania.	Hydroelectric Commission of Tasmania.	Corporation of City of Hobart.
Remarks	Dam has given satisfactory service and is now in good condition.	Multiple arch dam. 27 arches, 40 foot span. Few slight leaks sealed by mortar on upstream face during low water. Concrete still very sound.	Minor seepage through a few horizontal construction joints and grout pipes.	Minor leakage on arch face repaired by plugging visual faults on pressure fact.

Some scour has occurred at the downstream toe of some of these dams and, in one case, a plug of clayey material was washed from underneath the foundation of an arch dam, thus emptying the storage. However, all these minor defects have been repaired effectively at little cost."

Reports submitted by Mr. J. M. Main, Director of Public Works—May 23, 1956.

71. Miena—The Hydro-Electric Commission, Tasmania.
72. Clark

Miena dam was completed in 1922.

"Few slight leaks at horizontal construction joints, sealed by mortar on upstream face during low water. Concrete still very sound."

Clark dam was completed in 1949.

"Minor seepage through a few horizontal construction joints and grout pipes. Leakage into gallery through foundation drains less than 10 Imp. gallons per minute with reservoir full."

Report prepared by Mr. G. T. Colebatch, Chief Civil Engineer, The Hydro-Electric Commission, Hobart, Tasmania—5th April, 1956.

73. Ridgeway—Corporation of the City of Hobart, Tasmania.

Ridgeway dam was completed in 1919.

"Minor leakage since construction has persisted at a number of points on the arch face. Reduced by plugging visual faults on pressure face."

Report prepared by Mr. Kenneth V. Harris, Designing Engineer, Hobart City Council, Tasmania.

CIVIL ENGINEERING

The Magazine of Engineered Construction

ARCH DAMS: THIN ARCH DAMS FAVORED IN NORWAY[a]

C. F. Groner[1]

In Norway for the past fifty years the development of hydroelectric power has been a vital factor in the economy of the country. Dam construction, for special reasons, has followed a different path from that in most other countries. Norwegian dam sites are located in varying climates and at different elevations ranging from sea level up to 4,000 ft (1,200 m). Most dams are slender reinforced concrete structures for the following reasons:

1. As the transportation of building materials in the mountains is expensive, and the yearly construction season at high altitudes is only five to seven months, the total amount of material that has to be transported for a dam must be reduced as much as possible.
2. The water in our lakes and rivers is acid (pH from 5.6 to 6.7) and exceptionally clean. Therefore the water seeping through joints and cracks has no sealing effect.
3. In a thin dam it is easy to locate any subsequent leakage and to trace it to its source. This is considerably more difficult in heavy construction where the water may take an untraceable course through the dam.

Thanks to narrow valleys and good rock for foundations, there are a number of excellent arch-dam sites in Norway. Our first reinforced concrete arch dams were built about 1930. Since that time many such dams have been constructed (Fig. 1), up to a height of 164 ft (50 m). At present, one 230 ft (70 m) high is under construction.

Shape of Dam Sites

Arch dams are used at sites where the length-to-height ratio of the dam is as much as 7:1. The usual ratio is 3:1 or 4:1. The sites fall naturally into five categories:

1. U-shaped dam sites where the sides of the valley serve as abutments for the arch for the whole height of the dam.
2. U-shaped dam sites where the river has eroded weak material out of the river bottom. Normally this part has to be concreted first to form the foundation for the arch. See Fig. 2. In cases where there is a fault in the middle of the site, sheetpiling may be used to tighten it in special cases.
3. U-shaped valleys which broaden out considerably at the top.

a. Reprinted from Civil Engineering, April, 1957, p. 246-250
1. Consulting Hydroelectric Power Plant Engineer, Oslo, Norway.

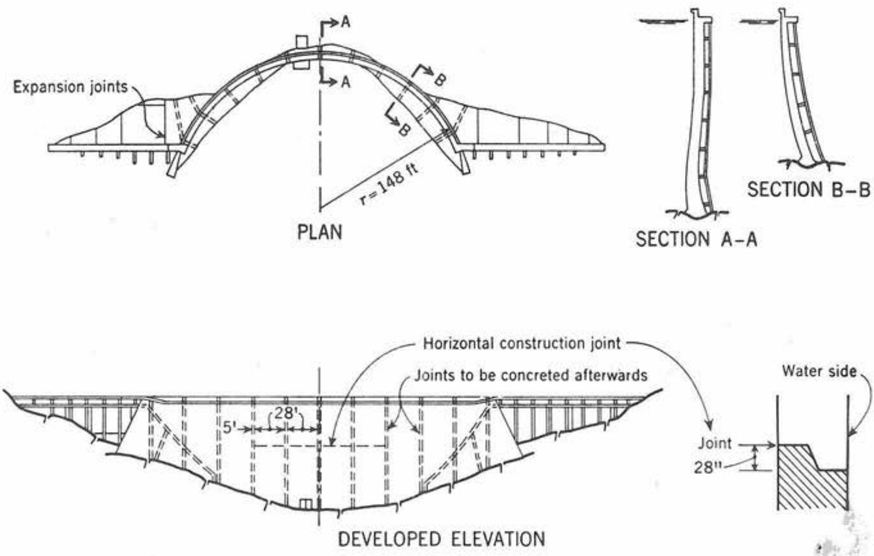
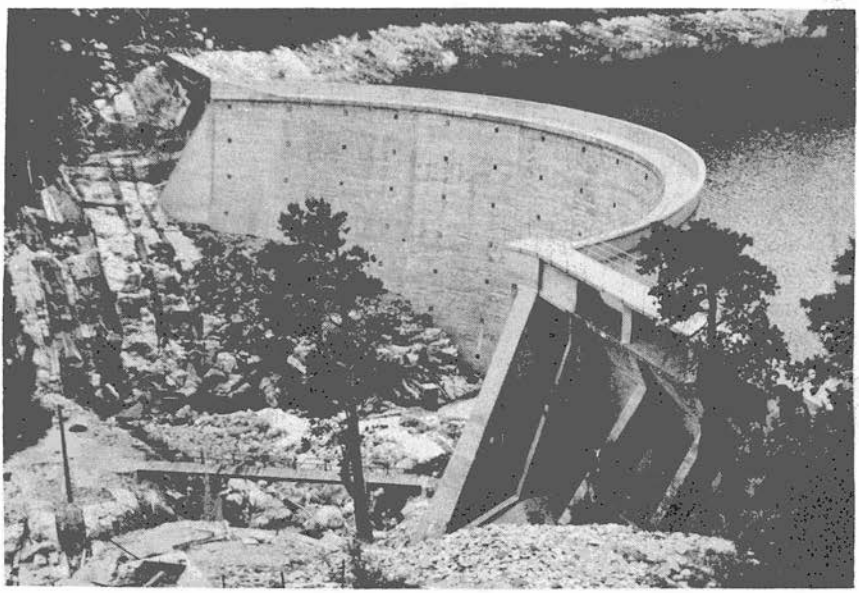


FIG. 1. Typical Norwegian concrete arch dam for U-shaped valley is Vatnedal, 98 ft (30 m) high. Note central arch with short constant-angle radius and side sections of flat-slab and buttress construction. In photo, note windowed insulating wall on downstream face, also buttress supports for side section in foreground.



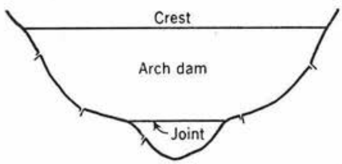


FIG. 2. At sites where bottom of U-shaped valley has been eroded, section of dam to fill this area is poured first.

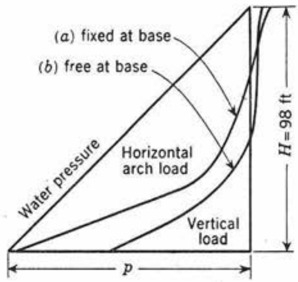


FIG. 3. Loads carried by arch and cantilever action in Vatnedal Dam are shown for two conditions: (a), when vertical section is assumed fixed at base, and (b), when vertical section is assumed hinged at base.

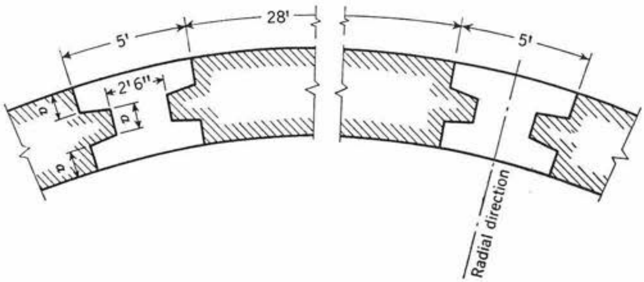


FIG. 4. Dentated contraction joints, without water stop, are poured in cold weather after alternate 28-ft (8.5 m) sections have reached full height and have attained nearly freezing temperature.

When an arch dam is selected for this type of site, the usual procedure is to construct the arch in the center of the profile and to place, at one or both ends, artificial abutments connected to flat-slab buttress dams or earth-fill dams. These combination dams complicate construction somewhat but have proved satisfactory. By avoiding the long, relatively flat arch dam, a considerably shorter radius is required, which in turn permits a less heavy dam section. This represents the most common type of arch dam in Norway. The Vatnedal Dam, Fig. 1, is an example.

4. The long, relatively low sites where multiple arch dams are employed. Only three such dams have so far been constructed in Norway and this type is not considered in this article.

5. Canyon dam sites, which are not common in Norway.

Principles of Construction

Smaller arch dams are designed mainly on the principle of constant radius, but large dams are designed often with a constant angle and variable radius (Fig. 1). The cylinder formula is used for the design of the smaller dams.

For larger dams, the trial-load method usually is employed using a series of horizontal arch sections and only one vertical cantilever section. In special cases, three vertical sections are calculated. Usually the vertical section is not considered fixed at the base. Fixity at the base means that a disproportionately large part of the water load is assumed to be taken by the vertical section. In an arch dam most of the load ought to be taken by the arch, that is, the arch should be the primary bearer. For the Vatnedal Dam, Fig. 3 shows the load carried in the horizontal and vertical direction, when (a), the vertical section is assumed to be fixed at the base, and (b), when this section is assumed hinged at the base. In certain cases a construction joint is formed at the base but normally the arch sections are concreted directly against the rock. Harmful cracks do not occur on the upstream face.

The Norwegian Water Course and Electricity Board has determined that the allowable concrete stresses should not exceed about 710 psi (50 kg per cm^2), and the buckling safety factor should be about 8. The dam is designed for a temperature rise and fall, when empty, of about 36 deg F (20 deg C) at the crest and 18 deg F (10 deg C), at the base; and when full, for a variation of 18 deg (10 deg C) at the crest and 9 deg F (5 deg C) at the base.

As a rule, the quality of the abutment rock is good except for the center part which comprises the actual river bed, where the rock may often be of rather poor quality. Rock conditions have required occasional grouting but the base usually is grouted after the dam is completed and while the reservoir is being filled. Any leakage in the rock can then be located, and grouting can be applied to stop it.

The dams are cast in vertical sections having a width of about 28 ft (8.5 m). Between sections there is a 5-ft (1.5-m) dentated contraction joint which remains open until the completion of the construction period (Fig. 4). Each section is calculated to resist maximum wind pressure. In the case of arches of constant angle, there is a considerable overhang at the quarter points, and reinforced concrete columns are used as temporary supports for the section.

By reducing the number of joints in a dam, the risk of seepage is reduced to a minimum. Each section is cast continuously from the bottom to the top, except where a section has a greater height than about 75 ft, when the pour is divided with a horizontal construction joint like that shown in Fig. 1. No

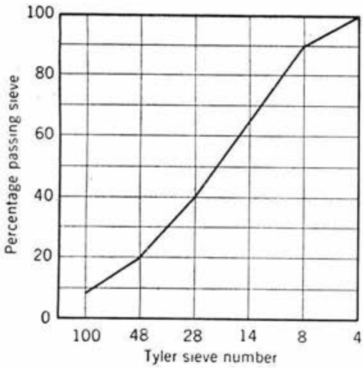


FIG. 5. This grading curve for sand is followed as closely as possible. Natural sand and gravel are used when available.

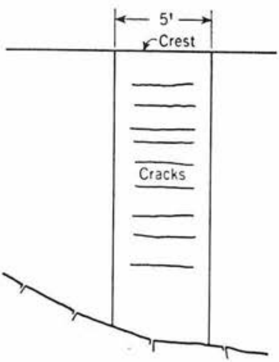


FIG. 6. Horizontal cracks may be caused by heat of setting when contraction joints are poured too rapidly or at too high an air temperature.

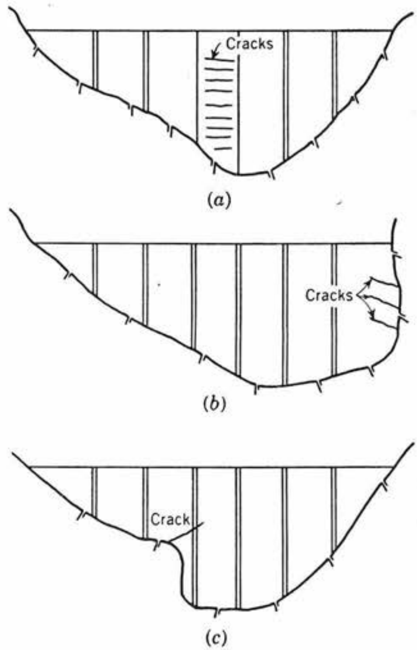


FIG. 7. Norwegian experience with arch dams in U-shaped valleys shows three construction procedures to be avoided: (*a*) pouring a section without contraction joints, (*b*) pouring a section against vertical rock face, and (*c*) allowing foundation rock to project into dam.

sealing strips are used for any of these contraction or construction joints. All cast surfaces are very thoroughly roughened and washed before pouring is continued. Great care is taken to prevent seepage.

Placing Concrete

The concrete mix normally employed has about 25 lb of cement per cu ft of concrete (350-400 kg per cu m). A minimum concrete cylinder strength of 3,300 psi (232 kg per cm^2) after 28 days is required. Great importance is attached to the correct grading of sand. To the extent possible, the sand curve shown in Fig. 5 is followed.

Natural sand and gravel are employed when available, but often crushed sand and stone must be used. Air entraining agent is added to the concrete to provide a maximum air volume of 3 to 4 percent. Internal vibrators are used during casting. The sections are poured at a rate of a 12-in. to 20-in. (30- to 50-cm) rise per hour, this slow rate being necessary to avoid a large temperature rise during construction.

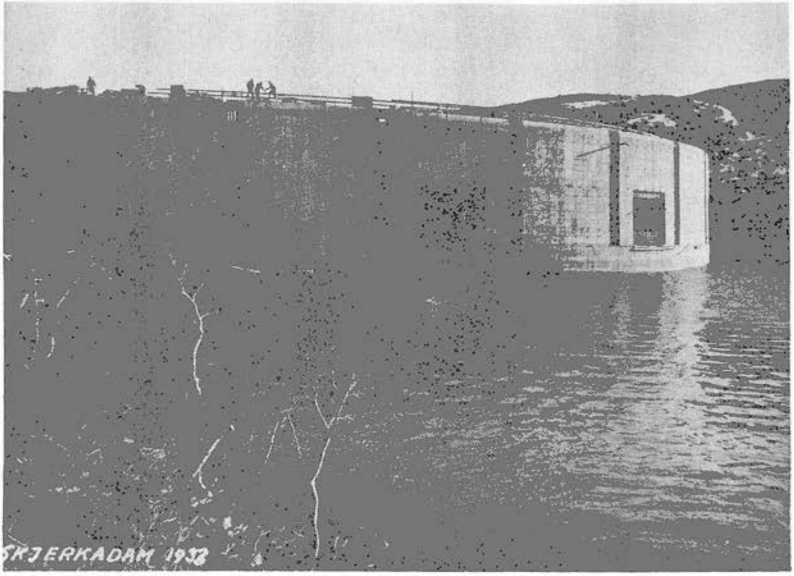
Contraction joints are concreted at the rate of about 12 in. per hour. It is extremely important that this rate be not exceeded in order to avoid cracking due to the chemical heat developed by the setting of the concrete. Contraction joints are concreted when the air temperature is about 32 deg F and after the adjoining sections have been cast at least three or four months. By this time the completed concrete has contracted as a result of shrinkage and temperature drop. If contraction joints are concreted at too high an air temperature, or if the pouring is done too rapidly, enough heat develops to cause horizontal cracks, as shown in Fig. 6.

The side sections usually are cold enough for pouring of the contraction joints between them when the air temperature has dropped to 32 deg F (0 deg C). It is important that the form-work be tight to prevent the concrete surfaces from cooling below the freezing point. Artificial heating is often necessary before concreting begins. Electricity is preferred for this heating.

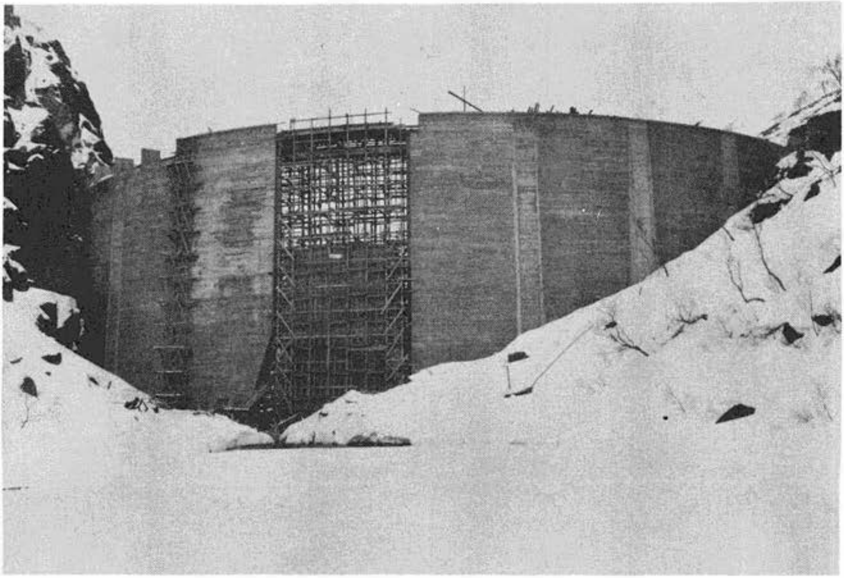
If the temperature is below freezing for a long time, the formwork must be insulated. A movable enclosure ought to be erected 12 to 15 ft (4 to 5 m) above the rock surface in the contraction joint in order to heat the lowest part of the joint. When concreting has begun, the setting temperature will normally maintain the air and side surface of the sections above the freezing point. As concreting proceeds the roof is gradually moved upwards.

To avoid involuntary joints, concrete must be poured in the form at least three times per hour. Under these circumstances a concrete mixer would not be fully employed, and consequently several joints are usually concreted at the same time. During concreting of the contraction joints, vibrators are the contraction joints varies greatly. In the middle of the joint the temperature will be considerably higher than at the edges adjoining the cold sections. I have taken temperature readings in the concrete of a contraction joint of an arch dam 98 ft (30 m) high, poured while the air temperature remained below freezing by as much as 10 deg F (−12 deg C) (in November and December), and found the temperature in the joint to be above freezing.

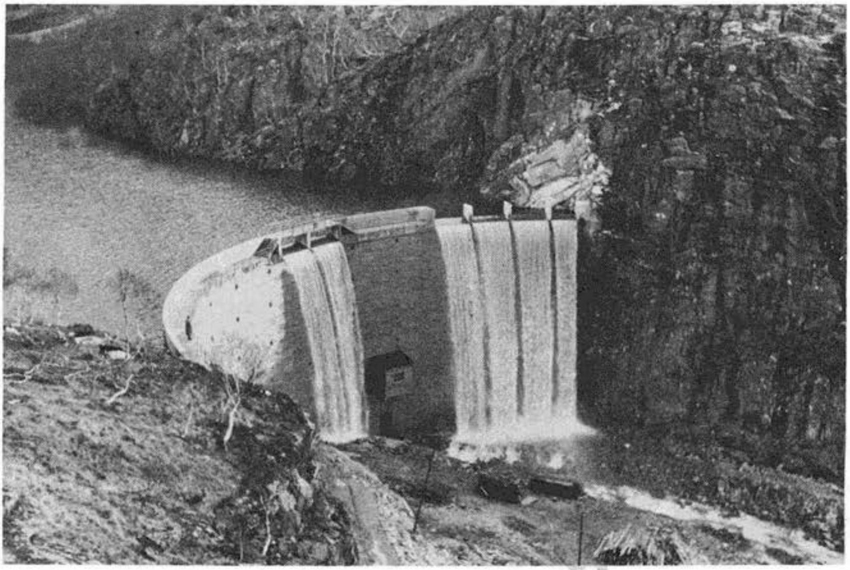
In certain cases the center section of a dam has been concreted without contraction joints but as a rule this has resulted in a certain amount of small horizontal cracking due to the large difference in temperature between the middle and the sides of the section. The same conditions prevail when concreting against a vertical rock face. Slightly diagonal cracks will develop

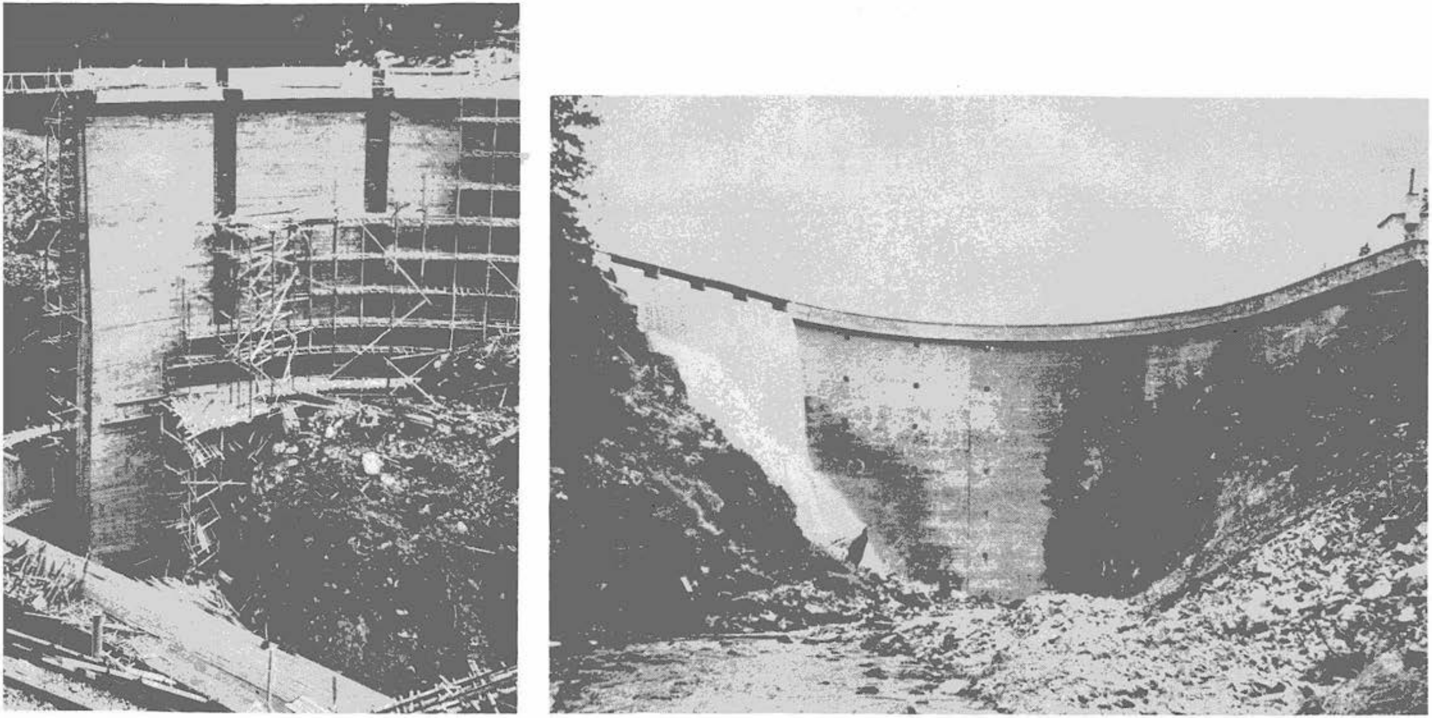


Construction in flat valley, Skerkavann Dam has ratio of length of 7:1. In upstream view contraction joints are being poured. Note two temporary openings for river flow. Completed dam is seen below.



Bergdal Dam is seen during pouring of last section (above) and after completion (below). In view above note contraction joint pours, and in lower view, over-flow sections.





Langli Dam, 155 ft (47 m) high, is seen (above, left) during pouring of main sections of arch, and (at right) completed, with water flowing over spillway. In view at left note gaps for contraction joints, to be poured later, and above, note windows in insulating wall.

against the rock face on account of the temperature difference. Sharp changes in height are carefully avoided at the base of the dam. It is necessary to level out the base; otherwise experience has shown that diagonal cracks will develop as shown in Fig. 7.

Formwork must not be removed before the temperature in the concrete has fallen approximately to air temperature. This procedure will avoid temperature cracks caused when the warm surface of the concrete comes in contact with the colder air. Water curing is required for at least three weeks after concreting.

Formwork is attached by means of bolts which at the upstream face are fitted with welded steel collars to reduce water seepage along them. These bolts have proved satisfactory in the main sections but in contraction joints no bolts are permitted to pass through the concrete. The outer parts of the bolts are unscrewed when the formwork is removed. The holes in the concrete surface, about 2 in. deep, are then cleaned and filled with motar.

Permanent Insulating Wall

Because of the slenderness of these dams, a certain amount of dampness will remain in the concrete as free water even close to the downstream face, no matter how compact or well proportioned the concrete may be. In the winter, with low and varying temperatures, frost damage will occur in the course of time. To avoid this, a permanent insulating wall is built to protect and insulate the downstream face against frost damage. This wall is normally of reinforced concrete 4 to 6 in. (10 to 15 cm) thick, placed at least 31 in. (80 cm) from the downstream face of the arch, as shown in the cross sections A-A and B-B of Fig. 1.

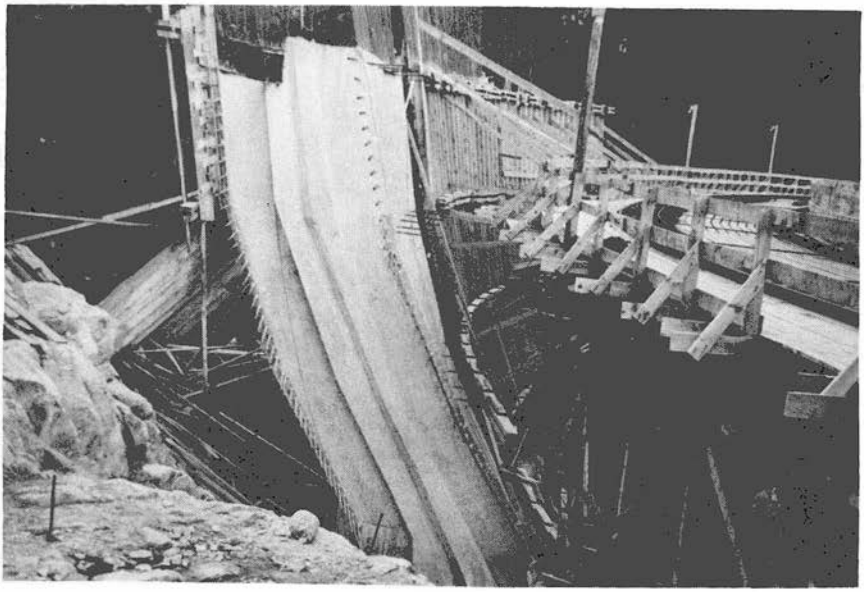
The insulating wall is supported on the arch itself by short pillars spaced about 13 ft (4 m) apart both vertically and horizontally. Walkways erected on these pillars facilitate inspection and control. This wall has no great insulating properties of itself; its main object is to provide still air on the downstream side of the dam. As a result, the temperature of the dam itself during the winter normally will be above freezing. The lowest temperature observed behind these insulating walls is about 19 deg F above zero (—7 deg C) even when the air temperature outside was as low as 20 deg F below zero (—30 deg C). The insulating wall is sometimes constructed of impregnated woodwork. It is fitted with windows, and in some cases facilities are provided for heating the air between the arch and the insulating wall.

The oldes arch dams in Norway are now more than 25 years old, and no damage of consequence has been observed on them. However, without an insulating wall there is no doubt that frost damage would occur. This has been clearly shown in a number of small intakes where an insulating wall has not been considered worthwhile. It is not only the low temperatures that result in frost damage. On the coast where the winter temperature is relatively higher, frost damage occurs because of repeated freezing and thawing during this season.

Cement plaster is applied only to the crest of the spillways as experience has shown that plaster elsewhere on the dam is unsatisfactory in a cold climate. Our experience with other types of waterpower construction shows that concrete with cement-plaster rarely is satisfactory.

For the larger dams the average construction cost ranges from \$45 to \$55 (U. S.) per cu yd of concrete; for the smaller dams, this cost may be \$55 to \$75 (U. S.) per cu yd, depending on the location of the site.

To sum up, Norwegian arch dams are constructed as slender reinforced concrete dams. They are built with comparatively few construction joints, and such joints are dentated to stop seepage. The downstream face is protected with insulating walls to avoid frost damage. Our experience with reinforced concrete arch dams has been excellent, and we consider these dams, when the site is suitable, to be superior to all other types both because of their safety factor and because of their statically correct force distribution.



Juvann Dam, constant-angle arch 160 ft high, overhangs at quarter points enough to require temporary support during construction by reinforced conrete columns at left. Note dentated edge in foreground, provided for con-:action joint. A horizontal construction joint also was required. Neither 'ater stops nor sealing strips are used.

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ARCH DAMS: STRESS STUDIES FOR ROSS AND DIABLO DAMS

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(Proc. Paper 1267)

SYNOPSIS

This paper presents representative results of a long-time study of stress from strain meters at Ross Dam. Included is an outline of the method followed in deriving stress from embedded strain meters. Also presented are the preliminary results obtained from a stress study on the Diablo Dam by strain relief measurements.

FOREWORD

This paper is one of a group presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams (April, 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time, it is not known exactly how many papers will be printed from the Symposium. So far, seventeen papers have been approved: "Arch Dams: Their Philosophy," by Andre Coyne (Proc. Paper 959); "Arch Dams: Trial Load Studies for Hungry Horse Dam," by R. E. Glover and Merlin D. Copen (Proc. Paper 960); "Arch Dams: Portuguese Experience with Overflow Arch Dams," by A. C. Xerez (Proc. Paper 990); "Arch Dams: Theory, Methods, and Details of Joint Grouting," by A. Warren Simonds (Proc. Paper 991); "Arch Dams: Santa Giustina Single-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 992); "Arch Dams: Measurements and Studies on Santa Giustina Dam," by Claudio Marcello (Proc. Paper 993); "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam," by Claudio Marcello (Proc.

Note: Discussion open until November 1, 1957. Paper 1267 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 3, June, 1957.

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Paper 994); "Arch Dams: Isolato Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 995); "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-Offs," by Claudio Marcello (Proc. Paper 996); "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira (Proc. Paper 997); "Arch Dams: Development in Italy," by Carolo Semenza (Proc. Paper 1017); "Arch Dams: Design of the Kamishiiba Arch Dam," by C. C. Bonin and H. W. Stuber (Proc. Paper 1018); "Arch Dams: Observed Behavior of Several Italian Arch Dams," by Dino Tonini (Proc. Paper 1134); "Arch Dams: Measurements and Studies of Behavior of Kamishiiba Dam," by H. Kimishima and C. C. Bonin (Proc. Paper 1182); "Arch Dams: Construction of the Kamishiiba Arch Dam," by K. M. Mathisen and C. C. Bonin (Proc. Paper 1183); "Arch Dams: Review of Experience," by Robert E. Glover (Proc. Paper 1217); and "Arch Dams: Stress Studies for Ross and Diablo Dams," by Joe T. Richardson and Owen J. Olsen (Proc. Paper 1267).

INTRODUCTION

As a part of this symposium on arch dams there are presented representative results of the long-time study of stress from strain measurements made at Ross Dam. The measurements were obtained by Carlson elastic-wire strain meters embedded in the mass concrete of the structure. The study of the stress behavior of this massive structure has been a joint undertaking between the City of Seattle, Department of Lighting, and the Bureau of Reclamation. The former organization contributed instrumental equipment and their installation in the dam, materials for required laboratory creep determination tests, and the program of systematically obtaining data. The latter organization contributed technical assistance during instrument installation, conducted laboratory tests for the determination of creep and other concrete properties, and performed the complex computations required to derive stress from values of measured data combined with the laboratory-derived function of creep. The results of the stress study were to be used[1] as would materially benefit both organizations.

Ross Dam

Ross Dam shown in Figure 1, is a variable radius arched structured located on the Skagit River in Northern Washington and is ranked as one of the world's highest arch dams. It has a height of 540 feet, a crest length of 1,300 feet, and contains 905,000 cubic yards of concrete. The dam is constructed in 26 sections, separated by contraction joints. Each section is 50 feet wide at the axis of the dam. During construction, the sections were raised systematically in a manner that provided for alternate low, intermediate, and high sections.

The dam was constructed in two major stages. The first stage was entirely completed in 1940, with the top of the dam at elevation 1365 except for sections at each abutment that had been left lower to serve as spillways. The later stage was completed between 1943 and 1949, with the top of the dam

1. Simonds, A. Warren, "The Determination of Stresses in an Arch Dam from Observed Strains," RILEM Paper 4 U.S.A., Themes 1b et 3a October 1955.

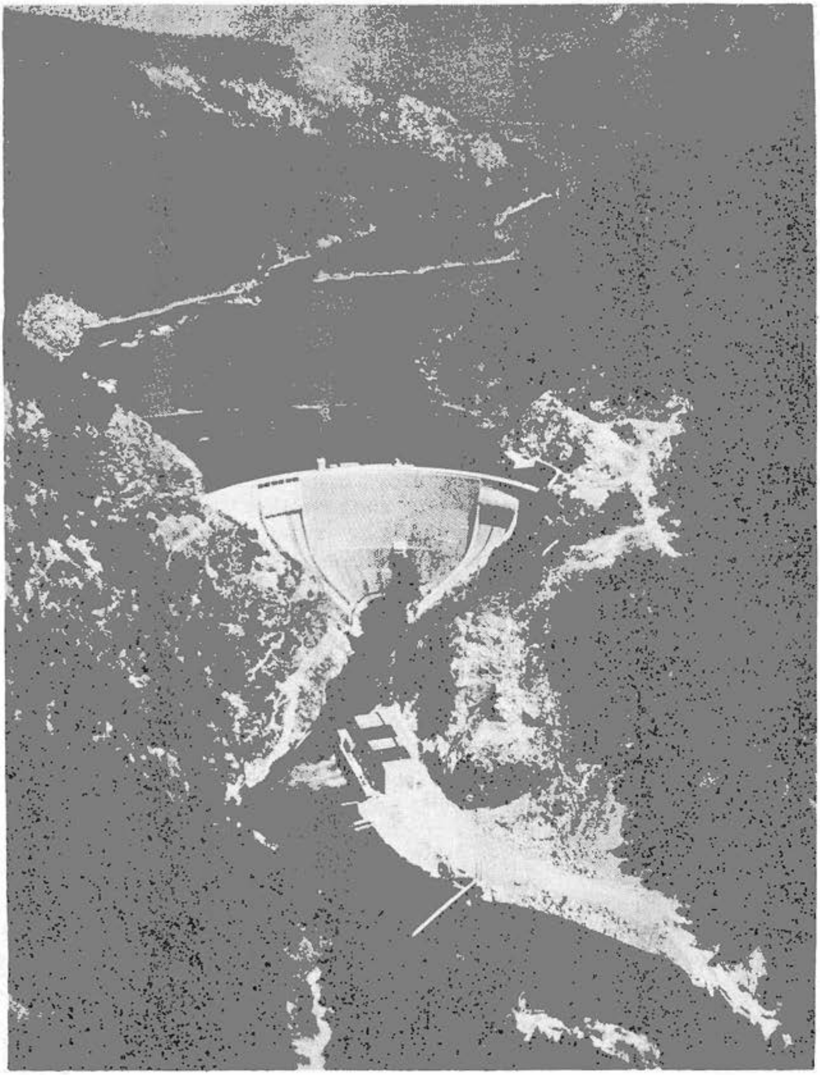


Figure 1. Ross Dam and Reservoir.

being at its present height, elevation 1615. No artificial cooling was done in the first stage concrete; artificial cooling was done in the later stage concrete.

The sectional construction of the dam provided for keyed contraction joints which were radial to the upstream face and approximately radial to the downstream face, the tangents being connected by the arc of a circle.

Ross Dam is unusual in some of its structural features. These include the provision for additional construction beyond the present stage to raise the dam to its ultimate height, elevation 1733.0. This contemplated additional construction will require thickening the entire dam, beginning at the base. To form a keyed bond between the old and the new concrete sections, patterns of square and vertical keys were cast in the concrete at the downstream face.

Other unusual features are the two hooded spillways incorporated in the ends of the dam. Their design was determined from experimental scale models built in the Bureau of Reclamation Laboratories in Denver. The flows of water routed down the two spillways collide in midstream, partially dissipating their energy against each other. The spillway deflector hoods are built of concrete, heavily reinforced with steel.

Construction progress and reservoir water surfaces elevation for the structure are shown on Figure 2.

Strain Meter Layout

Groups of strain meters were installed at 98 selected locations in the mass concrete of the dam as shown in Figure 3.

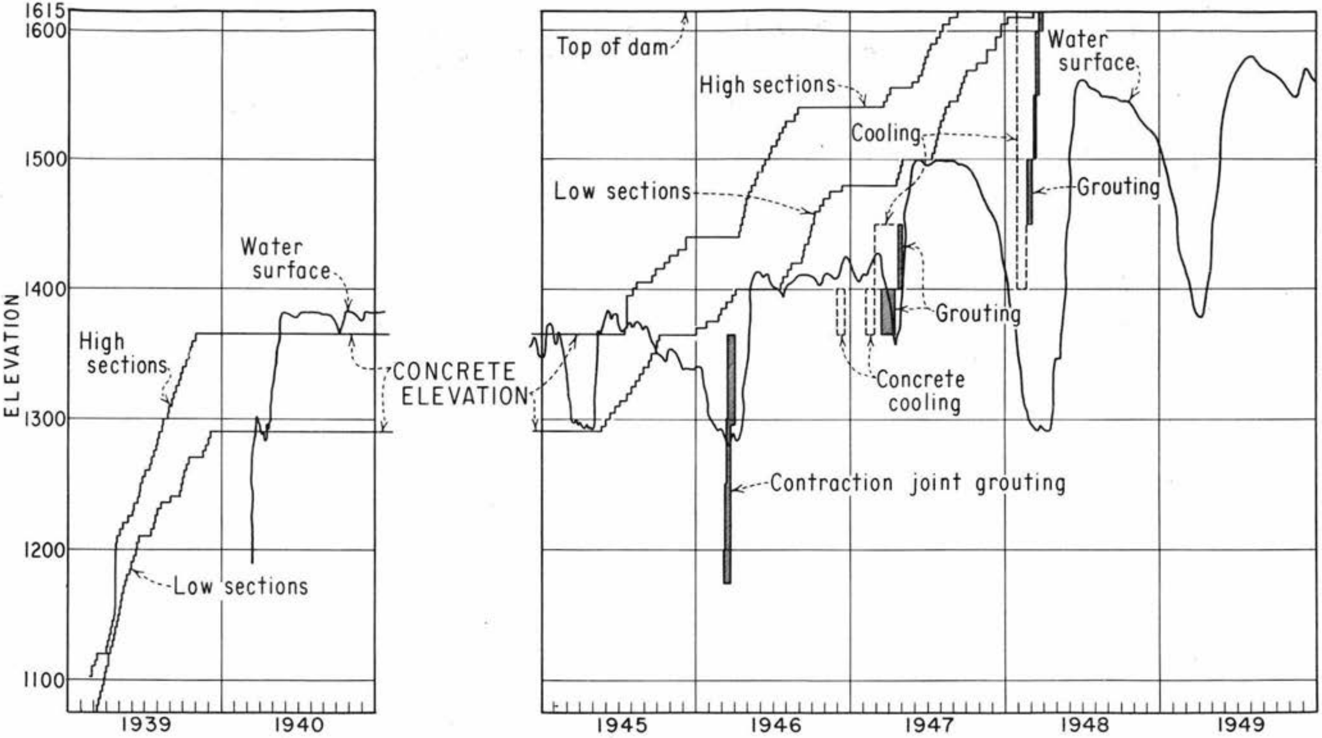
The groups of strain meters were placed in the planes of 11 arches, spaced 50 feet vertically between elevation 1100 and elevation 1600; additional groups of strain meters were placed in the planes or arches at elevations 1225 and 1275. In the planes of arches between elevations 1150 and 1350, the strain meter groups were generally located on lines parallel to the joint near the crown and parallel to the foundation at the abutment; in the planes of arches between elevations 1400 and 1600, strain meter groups were included on two to four intermediate lines, in addition to the lines near the crown and abutment. Each line of strain meter groups included a group near each face of the dam; most lines included one or more groups at the interior of the section; few lines contained only face groups.

Each group of strain meters is composed of several instruments oriented in the directions indicated in the detail on the lower part of Figure 3. Duplicate meters were not included.

With many groups of strain meters, a single additional strain meter was included to furnish data for determining growth or shrinkage in the mass concrete. The instrument was cast into a concrete block contained in a cardboard-lined wooden box. The whole assembly was embedded in the mass concrete of the dam with a strain meter group. This single strain meter thus served as a stress-free instrument contained in the mass concrete of the dam.

On several sections of the upper arches, stress meters were embedded to determine stress in the direction of arch thrust. A total of 21 stress meters was embedded in the structure. Results from 5 of these instruments are included with the stress results.

Since the dam is approximately symmetrical about the line of centers, strain meters were installed only in the right half.



CONSTRUCTION PROGRESS AND RESERVOIR OPERATION

Figure 2. Construction Progress and Reservoir Operation.

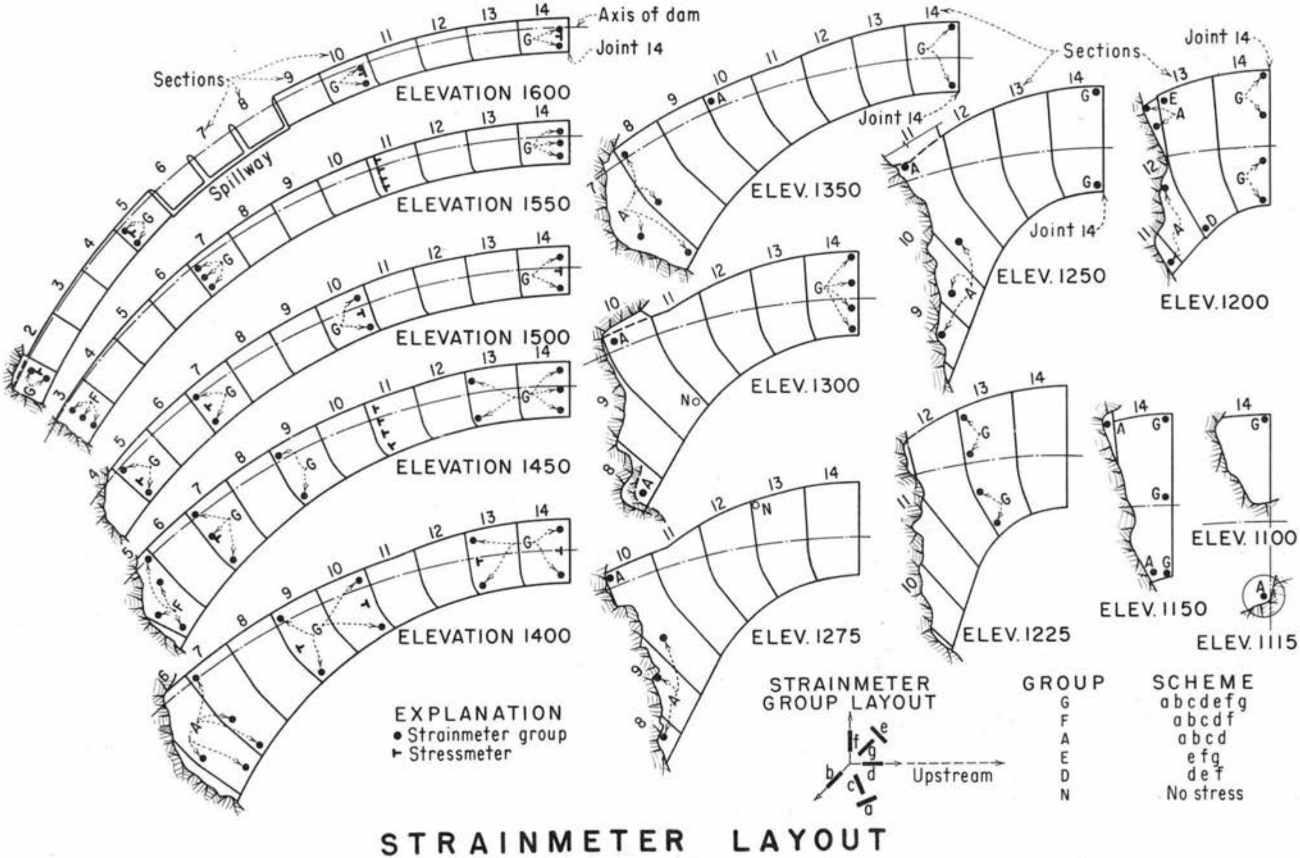


Figure 3. Strain Meter Layout.

Laboratory Tests

The derivation of stress from the collection of strain meter observations made at the dam required a knowledge of the properties of the concrete used in the structure. Accordingly, during the later stages that measurements were made at the dam, tests were conducted on concrete specimens in the Bureau of Reclamation laboratories in Denver.

Creep specimens were made in the laboratory[2] with Ross Dam materials shipped from the project. These specimens of identical concrete comprised 3 series, totaling 21 cylinders, each cylinder containing a Carlson strain meter.

In addition to the creep specimens, 30 cylinders were cast and tested at various ages to determine compression strength, modulus of elasticity, and Poisson's ratio.

All creep cylinders were sealed in lightweight copper jackets to prevent loss of moisture, thus approximating the curing condition of mass concrete. Sustained load of the cylinders was maintained in some cases by heavy coil springs and in others by a hydraulic diaphragm or ram.

A logarithimic function was found from the laboratory data which defined creep in Ross Dam concrete at each age of loading. The equation is as follows:

$$\varepsilon = \frac{1}{E} + F(K)\ \ln(t+1)$$

in which:

ϵ is elastic + creep strain per psi
E is modulus of elasticity, varying with age
F(K) is creep function, a constant for each particular age of loading
ln (t+1) is the natural logarithm of the time after loading + 1 day

The creep parameters for Ross Dam concrete are shown in the following table. By interpolation, values for the parameters could be found for any age of loading.

Loading Age in Days	$\frac{1}{E}$	(F)K
2	.480	.0898
7	.285	.0666
28	.245	.0326
90	.213	.0239
365	.180	.0170
5 year	.150	.0081

2. "Investigations of Creep Characteristics of Ross Dam Concrete," Bureau of Reclamation Concrete Laboratory Report No. C-787, Denver, Colorado, March 16, 1955.

The coefficient of thermal expansion was found from laboratory tests to be 4.6 millionths of an inch per inch per 1^{0} F. Poisson's ratio was found to be 0.20.

Method of Determining Stress

Essentially, stress has been derived from strain determined from the strain meters embedded in the structure combined with creep properties derived from measurements made on loaded cylinders of identical concrete in the laboratory. The strain in the structure in turn is derived from progressive differences of measured length change from each strain meter and the calibration constants of the strain meters.

Mathematically, strain is represented by an equation for a 3-dimensional system. Stress follows through application of a modification of Hooke's law.

Mathematical computations were carried out in the initial stages of the analysis by desk calculators, and in the later stages of analysis, by mechanical punched card computers (IBM).

The program of strain measurements at Ross Dam extended over a long period of time, during which time the concrete properties and the temperatures varied considerably. In the computations, successive corrections[3] were required for temperature in the concrete, for dilatation as measured by two combinations of three orthogonal strain meters, and for the Poisson's ratio effect.

The stress-strain relation can be expressed in the following equation which reflects the stress on orthogonal axes.

$$\sigma_x = E \cdot \left[\frac{1}{(1+\mu)(1-2\mu)} \left((1-\mu)\varepsilon_x + \mu(\varepsilon_y + \varepsilon_z) \right) \right]$$

and in like manner for σ_y and σ_z, in which

- σ is stress
- ϵ is strain
- E is instantaneous elastic modulus
- μ is Poisson's ratio
- x, y, and z are three orthogonal axes

Due to the Poisson's ratio effect in elastic materials, the strain measured on any axis is due not only to the stress acting on that axis, but also to the stress on the orthogonal axes. For the stress computation, it has been found convenient to adjust the strain on a given axis so that only the stress acting along that axis is represented. The adjusted strain is designated ϵ' and is computed from the three orthogonal strains from the equation:

$$\varepsilon'_x = \frac{1}{(1+\mu)(1-2\mu)} \left[(1-\mu)\varepsilon_x + \mu(\varepsilon_y + \varepsilon_z) \right]$$

3. Raphael, Jerome M., "The Development of Stresses in Shasta Dam," Transactions ASCE, Vol. 118, 1953, p. 289.

and in a lime manner for ϵ'_y and ϵ'_z. Stress is then computed from strain and elastic modulus from the equation:

$$\sigma_{xyz} = E\,\varepsilon_{xyz}$$

Since the properties of concrete do not remain constant when subjected to sustained loading and lapse of time, necessity requires modification of the Hooke's law equation to allow for creep in the concrete. Thus:

$$\sigma_{xyz} = E'\varepsilon'_{xyz}$$

where E' represents the average elastic modulus of concrete during the period ϵ' is developing and changes with the age of the concrete.

Some strain meter groups contained instruments only in the plane of the arch. At these locations, synthesis of strain in the vertical direction was required to determine 3-dimensional stress. The synthesized vertical strain was approximated, using the vertical strain from other nearby strain meter groups. The strain meter groups that furnished the comparable vertical strain had horizontal strains of similar trend and magnitude to the horizontal strains at groups that required the synthesized values.

Initially all computations were performed manually using electric desk calculators. This method[4] was a tedious and time-consuming process. Several successive improvements were made in the computational process. A method was devised by which stress computation was considerably accelerated by using IBM punched cards and high-speed digital computers.[5] The machine process at the beginning, merely reproduced mechanically the operations that had formerly been performed by hand. This computation may be represented by the equation:

$$\Delta\sigma = \frac{\Delta\varepsilon}{\frac{1}{E'} + F(K)\ \ln\,(t+1)}$$

Where:

$\Delta\sigma$ is the change in stress during an increment of time

$\Delta\epsilon$ is the total measured strain at the end of the increment of time, minus the elastic strain plus the creep strain at the beginning of the increment of time.

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4. Jones, K., "Calculation of Stress from Strain in Concrete," Bureau of Reclamation, Technical Memorandum No. 653, February 1955.
5. Raphael, Jerome M. and Bruggeman, John R., "Analysis of Strain Measurements in Dams by Use of Punched Card Machines," a paper presented at summer Convention ASCE, Denver, Colorado, June 1952.

$\frac{1}{E} + F(K) \ln(t+1)$ is the function of elastic and creep strain per psi obtained from the laboratory tests.

In the digital computer process, cards were punched for each increment of time for creep function and for total observed strain. The machine computers then integrated for each strain meter and for each increment of time, the elastic strain plus creep strain from all previous stress increments, subtracted that total from the observed strain, and computed the next stress increment. The cards were sorted and the process repeated for the next stress increment, and so on to the end of the study. A later modification of the computational method using an average logarithm concept was devised. This modification made possible the carrying forward in successive computations the summation of total strain and made unnecessary the computations of individual increments of creep for each increment of time.

In the earlier machine computation of stress, prior strain was found as the summation:

$$\varepsilon = \sum_{j=1}^{n-1} \Delta\varepsilon_i \text{, or } \varepsilon = \sum_{i=1}^{n-1} \left[\frac{1}{E'_{i+1}} + F(K)_{i+1} \ln(t+1) \right] \Delta\sigma_i$$

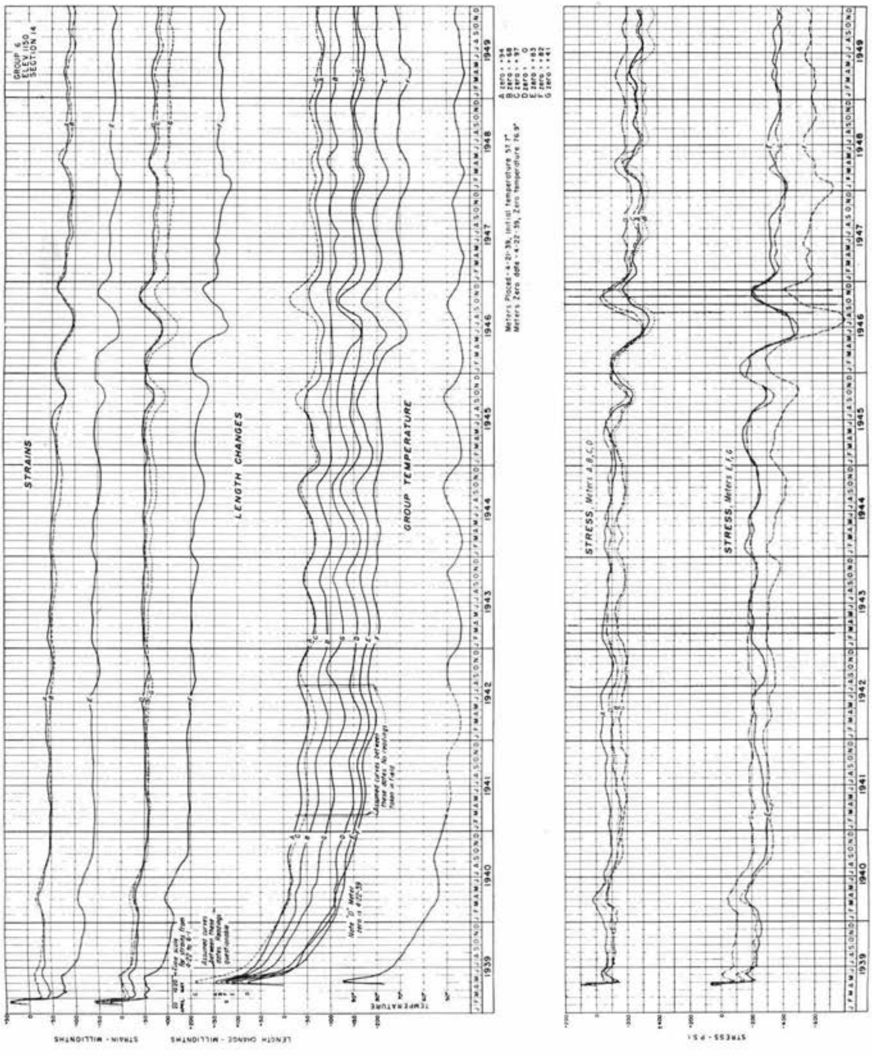
in which the interval i varies from 1 to n, n being the number of intervals. In later machine computation of stress, using the average logarithm concept, prior strain is expressed as:

$$\varepsilon = \sum_{i=1}^{n-1} \frac{1}{E_{i+1}} \Delta\sigma_i - \ln \operatorname{avg}(t+1) \sum_{i=1}^{n-1} F(K)_{i+1} \Delta\sigma_i$$

Stress Results

The stress results for the complete strain meter study cover the period beginning with meter placement, during construction, and extends through 1949, the cutoff year selected for the study. The study included all meters that were functioning in a satisfactory manner. Meter readings are still continuing as it is contemplated that a further possible extension of the analysis may be desirable at a later date. Figure 4 has been included to show as a sample the complete stress history for one group of strain meters.

Stress results have been selected for presentation at this symposium with the view of illustrating for several conditions of loading, the manner in which



ROSS DAM

Figure 4. Sample Stress History from Group of Strain Meters.

vertical stress is developed in the cantilever, and horizontal stress is developed in the plane of the arch in the direction of arch thrust, along with available accompanying conditions of principal stress.

The selected stress results are for two arches located at elevation 1200 and elevation 1450. Sections at these elevations are on the crown cantilever, the arch abutments, and intermediate locations. The loading conditions for these representative stress results are those for January 1 of 1940, 1942, 1948, and 1950. The selected results correspond to conditions of partial load on the initial-stage dam, total load on the initial-stage dam, partial load on the completed dam, and partial normal operating load on the completed dam.

Since the first and second conditions of loading are for the dam completed to elevation 1365, only results from the elevation 1200 groups of strain meters can be included for the initial-stage dam. In the illustrations for the third and fourth conditions of loading, results for both elevation 1200 and elevation 1450 are shown, as the dam had been constructed to its present height, elevation 1615.

The selected stress results are shown on four figures.

Figure 5, shows vertical stress results
Figure 6, shows horizontal arch stress results
Figure 7, shows vertical principal stress results
Figure 8, shows horizontal arch principal stress results

These figures show progressively, for four loading conditions, the variation and increase in stress as developed at the two elevational locations in the crown cantilever, at several locations in one arch, and at two locations in another arch.

Vertical Stress

Figure 5 shows the vertical stress distribution at elevations 1200 and 1450. The major dead and live loads are indicated on the section of the crown cantilever. The height to which concrete had been placed for each selected date and the corresponding reservoir water surface elevation are indicated on this section. Temperature is indicated at each point of stress measurement for each condition shown.

Crown Cantilever

The stress distribution on the crown cantilever for January 1, 1940, at elevation 1200 indicates compressive stress in the central portion of the section, reducing to slight tension at the upstream face and greater tension at the downstream face. Temperatures at the interior of the section were greater than at the faces. Two years later, January 1, 1942, the interior compressive stresses had reduced in magnitude and the tensile stresses near the faces had become compressive. At all points temperature had reduced. For both conditions of loading the joints between blocks were ungrouted. For the later condition the reservoir water surface had increased to elevation 1385, being retained by a timber crib constructed on the top of the initial-stage dam. No cooling of concrete had been done in the initial-stage dam.

On January 1, 1948, after construction of the dam to its present height, elevation 1615, low compressive stress was indicated at the interior of the section, increasing in magnitude at each face. Temperatures at stress points

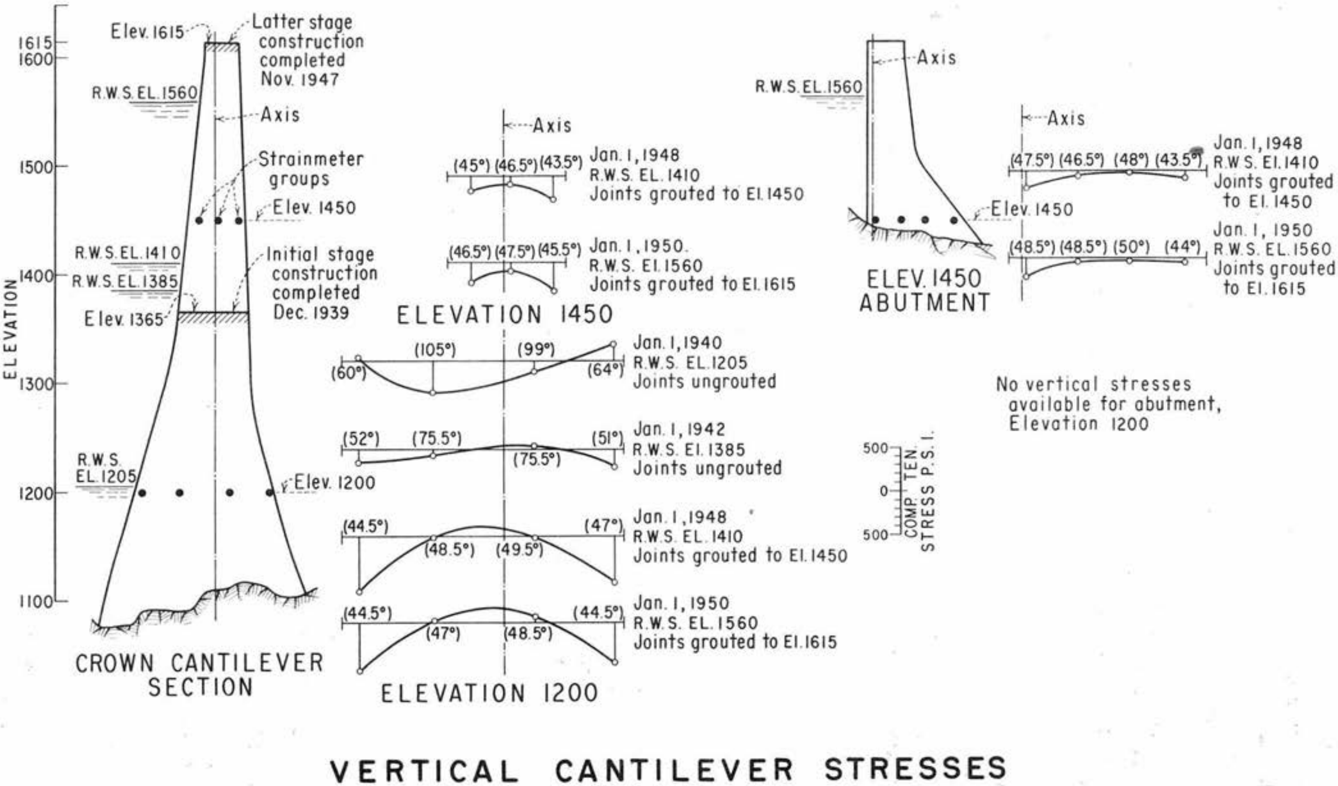


Figure 5. Vertical Cantilever Stresses.

had further reduced from the temperatures shown for January 1, 1942. The joints had been grouted to elevation 1450. The reservoir water surface had increased to elevation 1410.

With elapse of 2 years, the stresses for January 1, 1950, show slight change at the interior. Tension appears at the interior and a slight reduction of compression is indicated near the faces. Temperature at stress points had reduced slightly. Joints had been grouted to the complete height of the dam. Reservoir water surface was at elevation 1560.

At elevation 1450 the vertical stress conditions are shown for the two latter conditions of loading that have been shown for elevation 1200. Cooling of concrete had been done on the later stage of the dam.

The stress distribution for January 1, 1948, indicates lesser compressive stress at the center of the section than near the faces. Temperature was uniform on the section and joints had been grouted to elevation 1450. The reservoir water surface was at elevation 1450, 40 feet below the points of stress measurement. Two years later, January 1, 1950, stresses remained approximately the same. Near the faces stress had increased slightly in compression. Temperature was approximately the same, and the contraction joints had been grouted to the full height of the dam.

Abutment

No vertical stresses are available for the abutment at elevation 1200. At the right abutment of the dam, elevation 1450, stress distribution on January 1, 1948, was compressive, being greatest near the upstream face. Reservoir water surface was below this elevation at that time. Temperature was approximately uniform through the section and grouting had been done to elevation 1450. On January 1, 1950, the stress distribution was approximately the same as previously shown. A slight increase was indicated near the upstream face and slight decreases indicated on the remainder of the section. The temperature within the section was approximately the same as for January 1, 1948. The reservoir water surface had increased to elevation 1560 and the joints had been grouted to the top of the dam.

Horizontal Stress

The horizontal stress distribution for arches at elevations 1200 and 1450 are shown in Figure 6. Arch stresses are greatly influenced by temperature variations, and this effect is strongly shown in the progressive stress history of the several sections through the two arches shown.

Elevation 1200 Arch

The arch stress distribution for the elevation 1200 arch is shown for the crown and the abutment. At the crown on January 1, 1940, compressive stresses were indicated at the interior of the section, while at the faces tension was indicated. Interior temperatures were considerably greater than temperatures near the faces. The reservoir water surface was at elevation 1205 and joints were upgrouted. Two years later the reservoir water surface had increased to elevation 1385, the joints still remaining upgrouted. The stress distribution at the arch crown indicates reduced compression at the interior of the section. One interior stress point had passed from

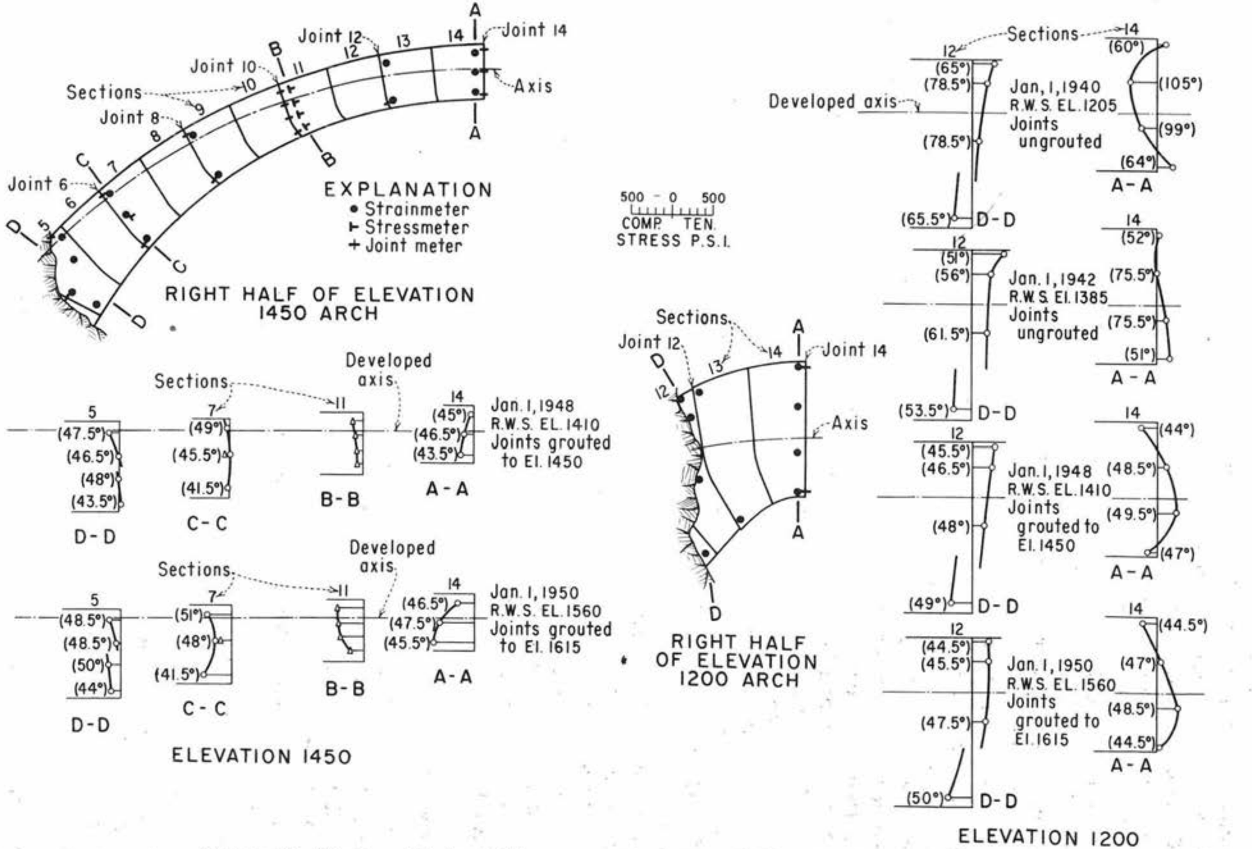


Figure 6. Horizontal Arch Stresses, Elevation 1450 and Elevation 1200.

compression to tension. The upstream portion of the section showed very little stress present, and near the downstream face the stress had reduced in tension. Temperatures at the interior of the section had decreased considerably while at the face the temperature had reduced only slightly.

On January 1, 1948, after completion of the dam to elevation 1615, the reservoir water surface was at elevation 1410. Joints had been grouted to elevation 1450. The stress distribution on the crown section of the elevation 1200 arch showed increased tension at the interior of the section and increased compression near the faces. Temperature on the section indicates an approximate uniform condition, reduced considerably below that shown in 1940 and 1942.

On January 1, 1950, with the reservoir water surface at elevation 1560 and the joints grouted to the top of the dam, the stress distribution indicated tension was still present at the interior of the section. Compressive stress remained at the upstream face and stress at the downstream face had reduced in compression and had become slightly tensile. Temperature conditions had remained very near the same as those shown 2 years earlier.

At the elevation 1200 arch abutment, the stress distribution indicates little change between the dates selected. Early stress distribution indicated tension over the entire upstream two-thirds of the section. The tensile stress increased with time at the point nearest the center of the section, then again decreased slightly. Near the upstream face tensile stress increased between 1940 and 1942, reduced in 1948, and in 1950 a further slight decrease was indicated.

Decreasing temperatures are indicated at all abutment stress points, becoming approximately stable in 1948 and 1950. The stress point nearest the downstream face at the abutment has not been included on the same stress curve as the points at the interior and upstream face of the section, since this stress point is not in the same block with the other three stress points. At this stress point compression has been indicated for the entire record shown here.

Elevation 1450 Arch

The stress distribution at several sections on the elevation 1450 arch are shown for the two latter conditions of loading after the present dam had been completed to its present height, elevation 1615.

On January 1, 1948, stress distribution at the crown indicated all stresses were compressive. Stress is greater at the center and downstream face than at the upstream face. At Section B-B, near the right quarter point of the dam, the stresses are from four stress meters. On January 1, 1948, greatest stress is near the upstream face. At the interior of the section and near the downstream face stress is less. On January 1, 1950, all stresses had increased in magnitude.

At Section C-C of Figure 6, near the abutment, the stress distribution indicates all stresses are compressive though low in magnitude. Near the center of Section C-C the stress from a single stress meter has been included for comparison with stress from the strain meter group. The stress from the stress meter is in good agreement with that from the strain meter group.

At the abutment a stress distribution is shown that is similar in shape to that at Section C-C. The highest compressive stress is indicated near the upstream face. Here again, a break is shown in the stress distribution curve as the four strain meter groups are not on a straight line. Temperature at all sections of measurement indicate the same trend with values being approximately the same.

Two years later on January 1, 1950, the reservoir water surface had increased to elevation 1560 and the joints had been grouted to the top of the dam.

Stress distribution at the crown of the arch showed considerable increase in compression over that of 2 years earlier. The greatest stress was still indicated at the downstream face.

At Section B-B of Figure 6, near the quarter point of the dam, the stress meters indicated stress to be compressive, increasing between 1948 and 1950. The great stress magnitude was near the upstream face of the dam. At Section C-C considerable increase in compressive stress is shown with slightly greater stress near the downstream face than near the upstream face, each face stress being considerably greater than the stress at the center.

At the abutment increased compressive stress is indicated. Temperature conditions for the entire arch show little change over those of 2 years earlier, having remained approximately stable throughout the arch.

Principal Stresses

The orientation and magnitude of the principal stresses in the plane of the crown cantilever, elevations 1200 and 1450, for the selected conditions of loading are shown in Figure 7. Principal stress determination in the vertical planes at the abutments of the elevations 1200 and 1450 arches were not feasible since the abutment strain meter groups contained insufficient meters in the vertical plane to make the required computations.

The orientation and magnitude of the principal stresses in the planes of the elevations 1200 and 1450 arches for the selected conditions of loading are shown in Figure 8. In the plane of the elevation 1450 arch the principal stresses are shown for sections at the crown, the abutment, and several intermediate locations.

CONCLUSIONS

No attempt has been made to completely justify the development of the stresses shown from the study since there is lack of available measured supporting deflection and deformation data. These data are usually required to trace the trend of stress development that would be indicated by correlated external measurements. The stress illustrations herein presented represent through progressive application the effects of known quantities of load comprising concrete, reservoir water, temperature, and grouting effects.

Trial load stress studies made prior to the construction of the completed dam, when compared with the strain meter stress study, show expected differences in stress results. This is not surprising since the study of stress by strain measurements and the study of stress by analytical methods is each on a basis different from the other. Each method reflects the assumptions and conditions for which the particular analysis is made, and thus cannot be expected to show equal final results.

Oftentimes, the analytical study represents only the stress conditions for

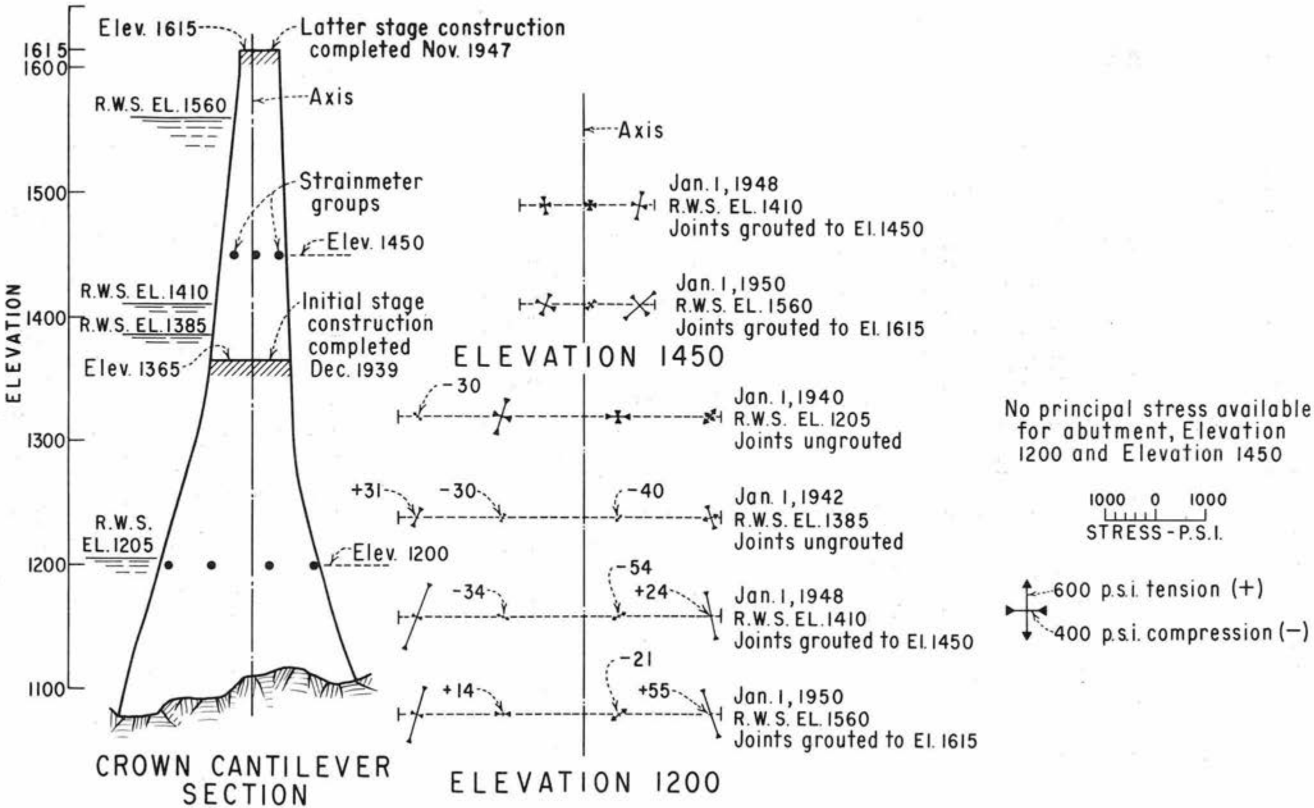


Figure 7. Principal Stresses—Vertical Plane.

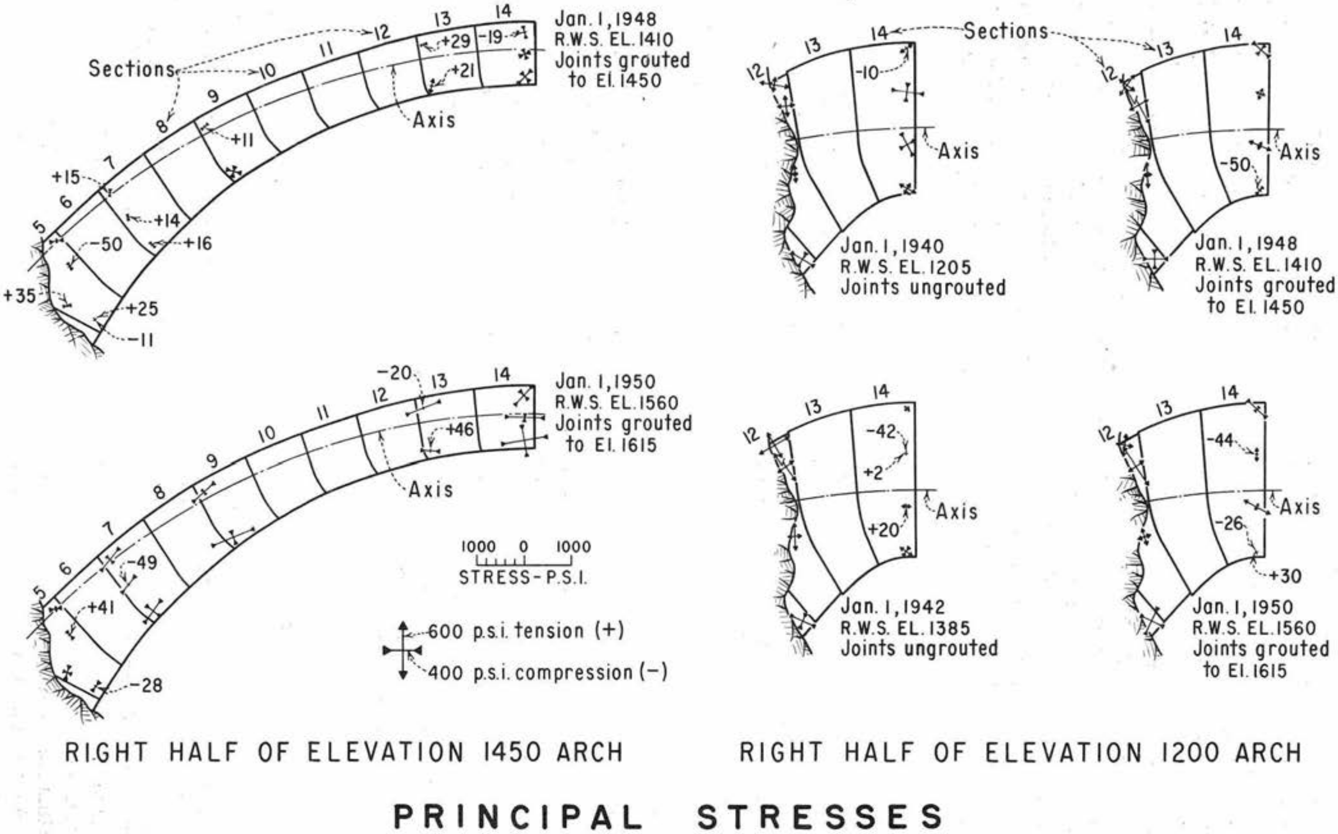


Figure 8. Principal Stresses—Planes of Elevation 1450 and Elevation 1200 Arches.

a structure during the maximum loading or some other fixed operation condition. The inservice conditions may not be comparable, since loading, as represented in the analytical study, may not occur on the prototype for a number of years after completion of the structure, or may never occur. A principal reason for difference between observed and computed stress appears to be due to the transient temperature changes which are not ordinarily included in an analytical stress study. The stresses due to these changes are often of the same order of magnitude as the load stresses. The effect of these transient temperature changes is one of the most striking features brought out by an investigation of this type. Only through the continuing joint efforts of engineers connected with both measurement and analytical studies of structures, can sufficient information be accumulated that will be useful as a means to effect the narrowing of the gap that exists between design and inservice conditions for a structure. The result will be the assurance that the as-built structure will operate stress-wise as anticipated by the design.

INTRODUCTION

As part of the program of determining the behavior of its structures the Department of Lighting conducted strain relief investigations at both Ross and Diablo Dams. The principal investigation was conducted at Diablo Dam where carefully planned test measurements were performed over a period of approximately 2 months.

Diablo Dam

Diablo Dam shown in Figure 9 is located on the Skagit River approximately 4-1/2 miles downstream from Ross Dam. Diablo Lake establishes the tail-water level for the Ross Powerplant. The dam is a constant angle arched structure with a gravity section at each abutment containing the spillways. The plan and sections of the dam as constructed are shown in Figures 10 and 11 respectively. The dam has a maximum height of 389 feet, a crest length of 1,180 feet and a volume of 350,000 cubic yards of concrete. The arch portion of the dam is 588 feet in length. The thickness at the crown varies from 146 feet at the base to 16 feet at the crest. The spillways are designed to discharge 121,000 cubic feet per second through 19 - 20.5- by 19-foot radial gates.

Construction and Service History

Diablo Dam was designed for the city of Seattle by the Constant Angle Arch Dam Company of San Francisco, California. The design of the dam was based on the assumption that the arch rings carried the entire waterload to the abutments with no transfer of load to the foundation by cantilever or vertical beam action. Contraction joints in the dam were spaced approximately 73 feet apart measured on the centerline of the crest. No provision was made for artificial cooling.

Although the dam was designed on the basic assumption that all of the waterload would be carried by arch action, the contraction joints were not grouted when the dam was completed in 1930. A 3-year period was to be

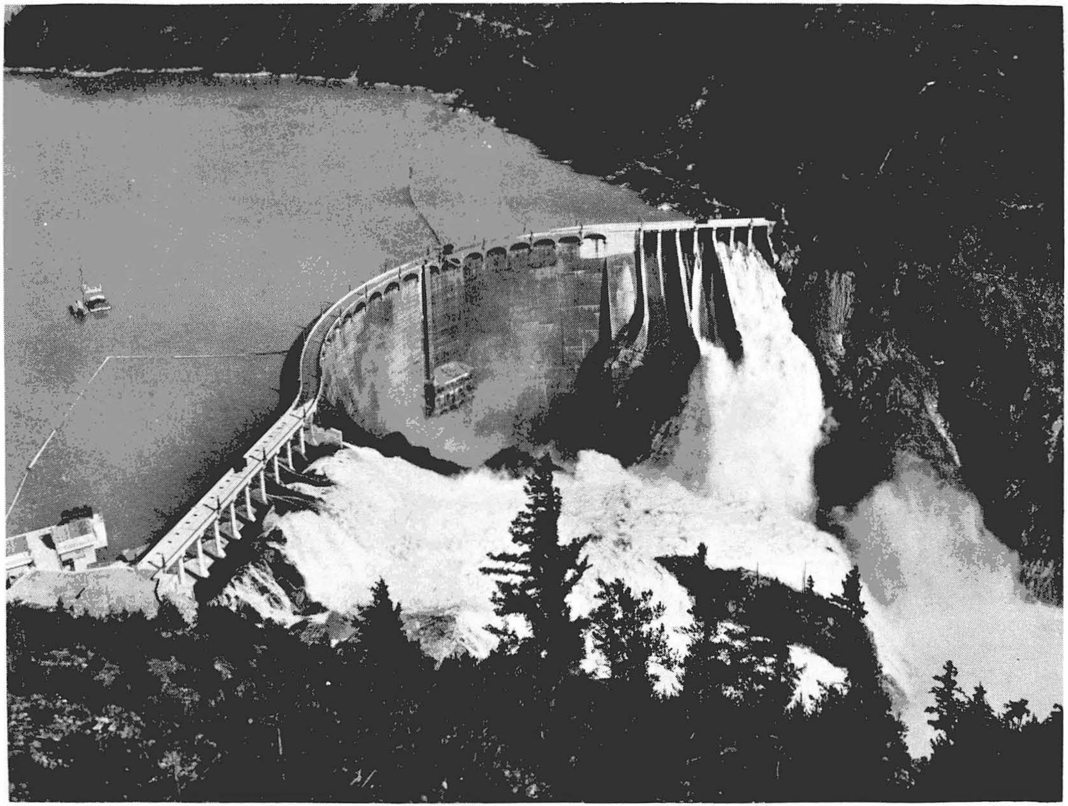


Figure 9. Diablo Dam and Reservoir.

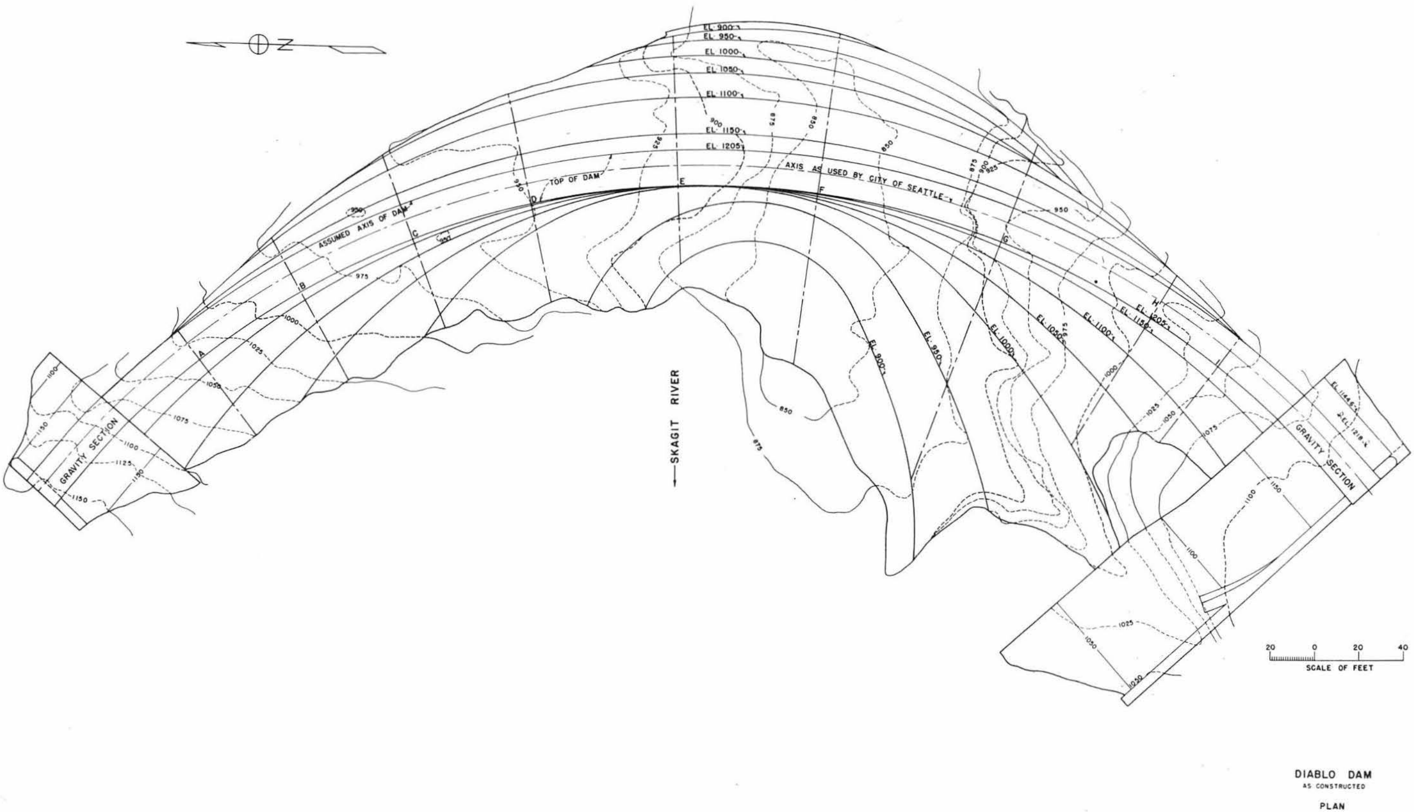


Figure 10. Diablo Dam as Constructed—Plan.

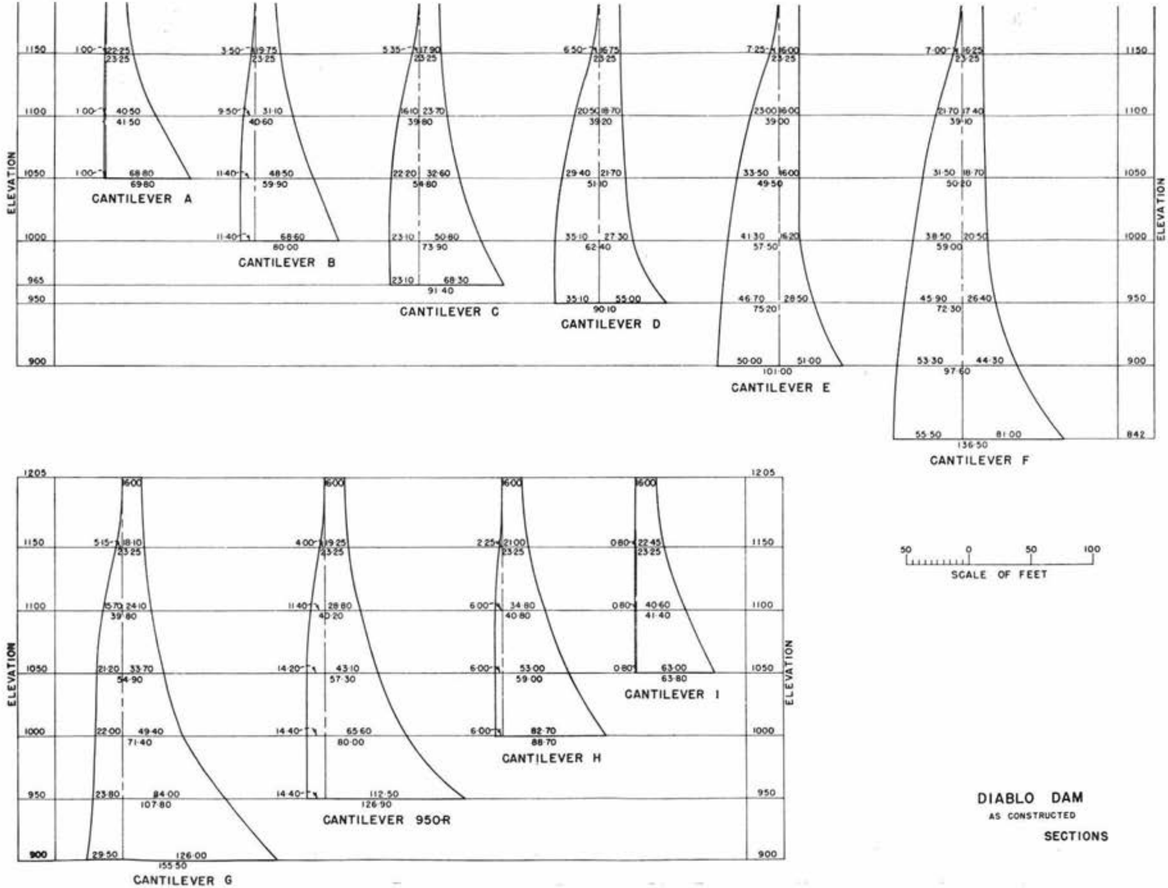


Figure 11. Diablo Dam as Constructed—Sections.

allowed for attainment of thermal stability before grouting the joints. The grouting of the contraction joints did not take place, however, until a program of general rehabilitation and improvements was initiated in 1950. In addition to contraction joint grouting, this work consisted of some foundation grouting, paving areas of exposed rock downstream from both spillways and adding some concrete to the left bank thrust block.

Construction of the dam was begun in October 1927, by the Winston Brothers Construction Company. It was completed in November 1930.

The rehabilitation and improvement work was by Morrison-Knudsen Company in 1950 and 1951.

Procedures

Prior to undertaking the major rehabilitation program for Diablo Dam, a method was sought whereby existing stresses could be evaluated in order to study the dam's structural behavior. The strain relief method was chosen because it had been used successfully to measure residual rock stresses in two tunnels and had been verified in the laboratory.

Using this relaxation method, in which strains are relieved by core drilling around SR-4 gage rosettes attached to the concrete surface, measurements of the existing or residual strain were obtained at 13 locations on the downstream face of Diablo Dam. Measurements were also made at two locations on the downstream face of Ross Dam for comparison. After each test on the surface, the procedure was repeated at the bottom of the core holes thus drilled, 16 inches beneath the surface at these same locations. The tests involved the installation of rock-wool insulating blankets and enclosed work platforms suspended from cables and anchored to the dam for protection from the inclement weather. In addition, strain relief was measured at six surface points on the side walls and top of the sluiceway tunnel through Diablo Dam.

Briefly stated, the strain relief method consists of the following steps: choosing representative locations, preparing the surface, attaching and waterproofing suitable gages, taking strain readings before and after drilling, core drilling to relieve strain, and determining the elastic and creep properties of the material from the cores thus drilled to permit conversion of strain to stress.

After selecting a gage location, an area 10 feet square is covered with a 2-inch-thick rock-wool insulating blanket for at least 48 hours prior to taking strain measurements and this remains in place until the tests are completed. The blanket has a 2-foot-square panel in the center which can be opened to permit access. Through this opening the area on the face of the dam on which the gage will be fastened is chipped and ground smooth. The concrete surface is then dried with heat lamps, thoroughly cleaned, and an SR-4 gage rosette backed with a piece of aluminum foil is cemented in place in the center of a 6-inch-diameter circle marked on the prepared surface. Heat lamps are also used to dry the gage cement. At the end of the drying period the gages are covered with asphalt mastic waterproofing.

With the gage in place and the temperature of the air and the concrete recorded, an electronic bridge is adjusted to balance the resistance in each of the wires of the rosette against the resistance of a dummy gage attached to an unstrained piece of concrete. The gage is then covered with a small metal pie plate, which is sealed in place to protect the gage from water during the ensuing core drilling operation. Using a 6-inch bit, an annular ring is cut around the gage rosette to a depth of 16 inches. The flow of cooling water to the drill is regulated to maintain an outgoing temperature at or slightly above the initial temperature of the concrete. As a result the initial and final gage readings are taken at approximately the same temperature. When the core is broken out the strain in the concrete is relieved. Another bridge reading is taken and the change in resistance, either positive or negative, is interpreted in terms of microinches of residual strain. This is converted to stress after the elastic and creep properties of the concrete cores are determined in the laboratory. When the first core is removed, the whole process is repeated at the bottom of the hole, i.e., the surface at the bottom is prepared, a rosette is placed, readings taken, and a second core 16 inches long removed from the same hole.

Reasons for Use of an Insulating Blanket

If strain measurements for the determination of stresses are to be made at the surface of a dam, it is essential that fluctuations of temperature be eliminated since temperature changes in the concrete will produce stresses which tend to obscure the stress due to loads. The temperature stabilization needed in this case was obtained by the use of an insulating blanket. It was considered necessary to suppress the daily changes over an area having a width which was large compared to the depth of penetration of the daily temperature changes. In unprotected concrete these changes are felt to a depth of about 3 feet, and a width of 10 feet was therefore chosen as being sufficient for this purpose.

An estimate of the thickness of insulation needed to reduce the temperature changes at the face of the dam to tolerable amounts can be made by the methods of the mathematical theory of the conduction of heat in solids.[6] Such estimates indicate that a 1 inch thickness of the ordinary types of insulation will reduce the amplitude of the daily temperature changes of concrete protected by it, to about 6 percent of the amplitudes which would prevail if it were unprotected. This amount of stabilization would give tolerable conditions for strain measurement.

If the insulation is to be effective, it must be protected from rain, and wind must not be allowed to cause air movement between the insulation and the concrete. To protect against these possibilities, 2-inch nominal semi-thick rock-wool batts with vapor barrier were attached to 3/4-inch plywood panels and then protected by an additional covering of waterproof paper and wire mesh which was wrapped around the edges and fastened on top. This entire assembly was then anchor bolted to the concrete at the point where measurements were to be made. Two 4- by 10-foot sheets of plywood with a 2- by 10-foot piece between were used to provide a 10-foot square panel. A portion of the 2-foot-wide strip was removable at the center to give access to the concrete surface.

Incidental to this investigation, temperature recording instruments were installed at suitable locations to obtain continuous observations during the

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6. "Conduction of Heat in Solids," by H. S. Carslaw and J. C. Jaeger, Oxford University Press 1948, Paragraph 24, page 55, Case ii.

course of the field tests. These included concrete surface temperature exposed to the sun, concrete temperature in the shade, and air temperature and humidity in the immediate vicinity of the dam. Reservoir water temperatures were measured at intervals of depth immediately upstream from the dam at the beginning and at the end of the test period.

Subsequent to these observations a joint grouting program was carried through to improve structural action in the dam.

RESULTS

Although the analysis is not yet complete, principal residual strains of the order of magnitude of 130 microinches tension and 120 microinches compression were observed at Diablo Dam. Secant modulus of elasticity and Poisson's ratio at 250 psi stress varied from 2.94 to 6.10 million psi and from 0.01 to 0.18, respectively. Using these elastic constants for converting strain to stress resulted in principal stresses ranging from approximately 450 psi tension to 550 psi compression at Diablo Dam.

Tee-delta rosettes consisting of four gages were used, which furnished one more strain measurement than necessary. This extra gage provided a compatability check on orthogonal strains and was used to compute standard errors by the method of least squares as a means of evaluating accuracy of the data obtained. The average standard error thus computed was $\pm$ 50 psi.

CONCLUSIONS

Although the results of this investigation have not been completely analyzed, it may be stated that the measured values fall within a range considered reasonable for dam structures. The strain relief method has been tested in the laboratory under known stress conditions and found to be reliable. We have complete confidence in the strain measurements obtained by this method and are devoting our present efforts toward a more realistic conversion from strain to stress which will take into account the imperfect elasticity of the concrete.

ACKNOWLEDGMENT

The results presented in this paper represent the combined efforts of many individuals and are by no means entirely the efforts of the authors. Results of the strain-meter investigation represent the combined efforts of teams of workers in the Dams Branch and in the Engineering Laboratories of the Office of the Assistant Commissioner and Chief Engineer, Bureau of Reclamation, Denver, Colorado; and of field and office personnel at Ross Dam and in the office of the City of Seattle Department of Lighting. The efforts of C. R. Hoidal, Designing Engineer, Seattle Department of Lighting, who contributed the description and history of Ross and Diablo Dams, valuable suggestions, and reviewof the paper, and J. M. Raphael, former Bureau of Reclamation engineer and pioneer in the field of embedded instrument measurements, who so ably directed the strain-meter analysis through to completion and organized the complex computations for processing by punched cards and high-speed digital computors, are gratefully acknowledged.

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ARCH DAMS: ECONOMY OF CONCRETE DAMS

Louis G. Puls,[1] A.M. ASCE
(Proc. Paper 1286)

FOREWORD

This paper is one of a group presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams (April, 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time, it is now known exactly how many papers will be printed from the Symposium. So far, eighteen papers have been approved: "Arch Dams: Their Philosophy," by Andre Coyne (Proc. Paper 959); "Arch Dams: Trial Load Studies for Hungry Horse Dam," by R. E. Glover and Merlin D. Copen (Proc. Paper 960); "Arch Dams: Portuguese Experience with Overflow Arch Dams," by A. C. Xerez (Proc. Paper 990); "Arch Dams: Theory, Methods, and Details of Joint Grouting," by A. Warren Simonds (Proc. Paper 991); "Arch Dams: Santa Giustina Single-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 992); "Arch Dams: Measurements and Studies on Santa Giustina Dam," by Claudio Marcello (Proc. Paper 993); "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 994); "Arch Dams: Isolato Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 995); "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-Offs," by Claudio Marcello (Proc. Paper 996); "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira (Proc. Paper 997); "Arch Dams: Development in Italy," by Carlo Semenza (Proc. Paper 1017); "Arch Dams: Design of the Kamishiiba Arch Dam," by C. C. Bonin and H. W. Stuber (Proc. Paper 1018); "Arch Dams: Observed Behavior of Several Italian Arch Dams," by Dino Tonini (Proc. Paper 1134); "Arch Dams: Measurements and Studies of Behavior of Kamishiiba Dam," by H. Kimishima and C. C. Bonin (Proc. Paper 1182); "Arch Dams: Construction of the Kamishiiba Arch Dam," by K. M. Mathisen and C. C. Bonin (Proc. Paper 1183); "Arch Dams: Review of Experience," by Robert E. Glover (Proc. Paper 1217); "Arch Dams: Stress

Note: Discussion open until June 1, 1957. Paper 1286 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 3, June, 1957.

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Studies for Ross and Diablo Dams," by Joe T. Richardson and Owen J. Olsen (Proc. Paper 1267); and "Arch Dams: Economy of Concrete Dams," by Louis G. Puls (Proc. Paper 1286).

SYNOPSIS

Safety and economy are considered in the studies associated in the final selection of type for large concrete dams. Selection of type is further influenced by the arrangement of dam and appurtenant works. The paper concludes with pertinent unit cost data obtained from sources in various countries.

INTRODUCTION

Safety and economy are often considered together in the studies of Cost and Safety Criteria, but generally the two considerations are not inseperably associated in the final selection of type for large concrete dams. The safety of a large dam is the predominant consideration and is satisfied by criteria, which cannot be compromised by economic considerations. Important safety criteria are:

a) Suitable foundation conformation and strength to carry the imposed loads.

b) Stresses within allowable limits due to maximum design loads.

c) Satisfactory stability when subjected to maximum design loads.

d) Control to limit maximum design loads.

These are unalterable criteria, which relate wholly to safety, and there must be funds and benefits available to at least justify the construction of a dam satisfying these criteria or the project should be declared infeasible. The golden rule of dam design and construction is not to sacrifice safety to accommodate a budget.

After satisfying the safety criteria, plenty of latitude remains for the study of economic factors. The important economic considerations relate to structure planning, topography, hydrology, geology, type of dam, materials, accessibility, utility relocations, arrangement of appurtenant works, such as powerplant, spillway and outlets, construction methods, and many other considerations.

It has been stated in technical journals and papers, that American engineers prefer gravity dams and that our friends across the seas are further advanced in the knowledge of building arch dams. It is true that many straight or curved gravity dams have been built by the Corps of Engineers, the Tennessee Valley Authority, the Bureau of Reclamation, and others. These selections cannot be considered as a pronounced aversion to, or an indictment against, arch dams.

It is our view that there is no desire to fan the breeze of competition or ambition in dam building in either continent, and there exists solely cordial relations with the mutual motive to interchange technical knowledge, to consult, and to assist one another to solve local problems and promote good will for the benefit of the profession and welfare of mankind in the development of natural resources.

Contrary to any imaginary belief, that the Americans prefer gravity dams, we subscribe to the selection of an arch dam at sites where this type can be used to advantage. In fact, the arch dam is preferred to the gravity dam because it not only requires less material and is therefore cheaper, but makes a much more effective use of material to provide much greater strength. A gravity dam may be called a monster structure with limited stability and surplus unutilized strength, whereas the arch dam may be considered an efficient embodiment of materials, strength, and beauty.

It was thought that a graphic picture might assist us in this discussion of economics of various sizes and types of concrete dams. The depiction attempt would necessarily be restricted to the important elements and could not be inclusive of all the properties and considerations, so caution must be exercised in the use of the information. The desired graph was obtained by correlating the three properties—canyon shape factor, height of dam, and volume—as shown in Figure 1. The selected canyon profile is a trapezoid for simplification defined by the dimensions, height of dam and developed length of crest, and the average slopes of the abutments. The formula for volume of dam has an approximate analytical basis through the process of integration, but is here considered hypothetical as a convenient means of creating a graph on which to plot data of dams, built or under study. The canyon shape formula resulted as one factor in the volume integration. The lower zone of the graph (small canyon factor) will relate to narrow canyons which would accommodate arch dams. The upper zone of the graph (large canyon factors) will relate to straight gravity dams. The middle zone would be a transition area and would relate to dams of varying curvature. The graph, based upon the full range of canyon shapes, is an attempt to depict by a family of curves, though approximate, the general American practice of concrete dam design for several past decades.

The number of dams over 100 feet in height, constructed or proposed in the world, of the several types within the last century, are approximately as follows:

Gravity	754
Arch	192
Multiple Arch	38
Buttress	39
Rockfill	77
Earthfill	358
Total	1458

Of this total, about 40 per cent have been constructed or proposed in the United States. Of all Nations, which have more than 10 dams, the United States has about 53 per cent of the total number; also, of all dams over 400 feet high in the world, constructed or proposed, about 40 per cent of them are located in the United States. These data have been taken from Mr. R. A. Sutherland's "Statistical Review," A.S.C.E. Proceedings, Separate 355, November 1953.

Comments may arise reflecting in some degree that the American practice is conservative. These comments are probably the result of hasty observations. It is difficult to resolve to a comparable basis the several Countries' practices, because conditions vary—topography, geology, character, quality of materials, and consideration of the damage which might be done downstream of a site if the dam should fail. Cost of materials and labor and efficiency of doing work vary to the extent that hasty comparisons may lead to erroneous conclusions.

It is probably a fairly sound precept to use for guidance, that the evolution of dam design should be a slow process. This must necessarily be so, because a dam is a highly important structure regardless of size.

If the behavior, functions, and operation of a constructed dam conform to design criteria, assumptions, intentions, and expectations, all is well and a great source of satisfaction. A malfunction can cause serious concern and involve large remedial expenditures. A failure is a major disaster resulting probably in inestimable loss of life, property, and benefits many times greater in value than the original investment. To avoid the risk of failure, where consequences may be significant, is conducive to a long term evolution in dam design. The performance record of dams appears good, but a word of caution against overconfidence is urged in attempts to accelerate progress under the guise of mythical economy.

We believe our processes of trial load analysis or model testing for studies of stresses and stability are fully adequate, but we must not abandon continued search and study of theory and prototype behavior. We believe our knowledge of foundations is fully adequate but there may be additional important properties and characteristics of rocks yet to be revealed. We believe the materials are suitable, but frequently mystifying weaknesses do appear, urging further research. We believe our methods of construction are fully adequate but offer a fertile field for improvement to obtain better workmanship at lower costs.

A word of caution is urged in attempts to accelerate progress, in attempts to reduce thickness with increased heights of dam too far ahead of consolidation of gains in theory and firming up of new knowledge. We believe we are cognizant of the essentials of behavior of arch dams, but we cannot be too confident. There may exist, after a few years of duty, the gradual growth of a condition introducing serious deviation of anticipated behavior, which defies investigation either by trial load analysis or by model. Although much has been accomplished to improve the quality of concrete, deficiencies in durability have occurred for reasons unknown. Progress, yes, but cautious advances are advisable.

Structure planning is one of the most fascinating games of economics in dam design. Herein is exercised skillful selection of arrangement of the several structures and facilities to obtain the best operating conditions at the least cost. This process of choosing the most suitable position of the appurtenant works sustains inviolate the basic safety criteria of the principal structure, but does tolerate the risk of some damage under extreme operating conditions to minor elements depending upon the ease of repair, importance of outages, and calculation of long range economy.

Let us select, for discussion, two plans, with their variations, incorporating the gravity and the arch dam. Assuming the rock foundation is competent to support either dam and that the type of dam has been selected ideally based upon the canyon factor, that is, a wide river channel, indicates the selection of the gravity dam and a narrow canyon—the arch dam.

Now, the economic game starts. Our principal consideration is to choose the best arrangement for the spillway, outlet works, and the powerplant. For the gravity dam we find the ideal arrangement is to spill over the dam, directing the flow through a dissipating works located in the river channel at the toe of the dam. The powerplant is located adjacent to the stilling works close to the dam. The river outlets are embedded in the dam and conveniently discharge on the spillway face into the stilling basin. The diversion of the river is accomplished by two stages; first, over blocks in the powerhouse area and, finally, over blocks within the spillway.

For the arch dam plan, the powerhouse is located in the channel as close to the toe of the dam as possible and the spillway is an inclined tunnel joined to a horizontal tunnel and discharges at a slight angle into the river channel at a suitable distance downstream from the powerplant. Diversion of the river is accomplished through a tunnel connecting to the horizontal reach of the spillway tunnel. The river outlets can be embedded in the dam and extended downstream past the powerplant discharging into the channel with maximum amount of dissipation obtainable.

The arrangements of dam and appurtenant works described are generally found to be most economical and provide suitable operating characteristics for the combinations, assuming the dam types have been selected according to their canyon factors. It is a rare occasion when it is possible to base economics upon types of dam only. The combination of all structures, dam, spillway, and powerplant must be considered together and the overall cost determined.

Suppose we are considering whether we should select the gravity or arch type at a specific site. This assumes that the canyon shape is suitable for the arch dam and, if so, it follows that it would probably also accommodate the gravity type. Now we are confronted with the difficulty of finding space to locate the spillway and powerplant for the gravity dam because the canyon is narrow. If the required spillway capacity is large, it will be desirable to discharge over the dam and place the powerplant at some distance downstream from the dam using the space in a natural widening of the channel or, if necessary, the provision for considerable excavation with extended penstock. If the spillway capacity is small, then there is economy in placing the powerplant in the channel at the toe of the dam and locating the spillway in a tunnel, which corresponds to the arrangement generally selected for the arch dam.

Diversion of the river during construction may be important in the selection of type of dam depending upon the magnitude of flow both in peak stage and duration. Shifting the river from one portion of the dam to another involves delays. These costs must be compared to tunnel diversion which permits continuous operations without river handling interruptions.

Comments may be of interest regarding spillway design. A gated or ungated spillway can be set in the crest of a gravity dam without difficulty but the thinness of an arch dam often introduces hydraulic and structural difficulties for an overflow crest arrangement. This difficulty can be overcome by grouting the contraction joints only to the elevation of the spillway crest and cantilevering the blocks above this elevation. For arch dams we prefer to construct the spillway independently of the arch dam structure.

Dissipation works are required only where there is need to avoid damage to the foundation of the important structure or impariment of draft tube operations. Discharge from tunnel spillways will generally cause erosion and create barriers in the channel resulting in reduction of power head. Channel maintenance costs from spillway action are part of the economic calculations.

Whether the spillway is conducted over a dam, through a tunnel, or released to fall freely gives great concern, especially for high dams, regarding serious detrimental action at the lower terminus of the flow. Misalignment of boundaries and flow lines may cause serious damage by cavitation. Joints or cracks in either concrete or rock are conducive to deflection of high velocity flow with creation of disruptive pressures or cavitation. The risk of subjecting foundation rock or protective slabs of concrete adjacent to the toe of the dam to disruptive force is an important consideration. Calculated risks should apply to secondary and minor structures but never to the integrity of the dam or its foundation in any attempt to obtain economy.

Due to progress brought about by experience, research, including studies of prototype behavior and thorough analytical processes, including photoelastic studies, our confidences have been supported to the extent that we have been agreeable to increasing permissible stresses in dams from 500 to 1,000 pounds per square inch in favorable instances within the last 25 years. The 1,000 p.s.i. permissible stress providing a safety factor of 4 to the concrete strength may be the limit we will hold to until our gains in knowledge of materials and behavior have been consolidated to give additional support to our confidences.

The American engineer and contractor constantly are on the alert endeavoring to find ways and means for reducing construction costs. Stimulated by the spiraling cost of living, demands for wage increases are ever before us. Because of increased efficiencies in plant construction and operation, the cost of materials and equipment has not advanced in the same degree as labor. With this changing ratio of labor over materials, labor saving methods in construction are given greater importance in this endeavor to reduce overall costs. One of the important elements of cost in a concrete dam is the cost of forms. Inasmuch as form construction consists mainly of hand labor, multiple use of forms is of prime importance in reducing cost of mass concrete in dams.

Unlike conditions that exist in many foreign countries, our labor costs are comparatively high and materials supply generally adequate. In many foreign countries the reverse is true; materials are scarce while labor is cheap and abundant.

For illustration let us consider a gravity dam, an arch dam, and a dome arch type of dam for the Flaming Gorge site. Comparative estimates have already been made for a gravity and arch dam at this site. The gravity dam will require about 1,900,000 cu. yds. of concrete and the arch dam will require about 940,000 cu. yds. of concrete. Even though the cost per cu. yd. for the larger volume will be less than for the smaller volume dam, the total cost for the gravity dam will be greater than for the arch dam. Where the dam abutments are competent and the crest length suitable, the arch type of dam would be adopted because of the substantial saving in overall cost. Suppose we now consider what effect a European dome-arch type of dam would have upon the cost at this site. It may be found that the dome type of dam will reduce the volume about 15 percent from the arch type. With this reduction the dome-arch dam will require about 800,000 cu. yds. of concrete as compared to 940,000 cu. yds. for the current American arch dam. On a volume basis, disregarding for a moment the more complicated form work required for the dome type dam and the limited reuse of forms, the cost per cu. yd. for the smaller volume will be greater than for the larger volume. When consideration is given to the additional cost for the more complicated form construction and limited reuse thereof the costs in America of the dome-arch construction will be equal to or be greater than the arch type of construction. Essentially the difference of the two practices may be 15 percent in volume of dams, but when converted to respective costs the margin may be small, so that economic considerations may dictate different choice within the several countries.

Mr. Robert E. Glover has addressed inquiries to our friends in Italy, Portugal, and France on the subject of Costs of Concrete Dams. It was thought the interchange of specific information would assist in explaining the difference of practices. Gracious responses were received and furnish interesting data for our study and analysis. These replies are reproduced here in their original form.

a) Societa Adriatica Di Electricita, Italy, N. A. Biadene

"Further to our letter of August 18 and with reference to your letter of August 8 concerning the cost of concrete employed in European arch-dams.

"We were instructed by Mr. Semenzo, who is now taking a journey in the Far East, to supply you with all the information available on this subject—which unfortunately is rather limited.

"With regard to European costs in general, we regret we are not in a position to give you any details—with the exception of Switzerland where the price we believe is about 70 Swiss Francs, cement included.

"As for Italian prices, the cost of the concrete used in medium size arch-dams (about 75 000 cu.m. volume) built by our Company, is approximately 6 000 to 7 000 Lire per cu. m. cement excluded. The cost is, of course, higher for dams with smaller volume, and lower for those with greater volume. The incidences of labor in the making of concrete is about half the said figure. The incidence of materials amounts—approximately and with percentages varying according to whether it is river material or crushed rocks, distance from quarry, available transportation, etc.—to 10% - 15%. The remainder is constituted by working expenses and plant amortization.

"We cannot give you exact data concerning the other Italian dams not built by our Company, but assume they do not differ much from the above mentioned figures.

"We have good reasons to suppose that the cost of our concrete—in spite of pouring difficulties resulting from the particular shape (thinness, pronounced curvature, etc.) and the location of our constructions (often situated at great altitudes a.s.l., in cold areas, etc.)—may be regarded as rather low. This is due both to the relatively low cost of labor in Italy and to the keen competition among Italian builders that we consider very efficient and well equipped.

"We are sorry we cannot supply you with further details in this connection. Additional information on a collateral subject may be obtained from the Paper presented by Mr. N. A. Biadene to the Int. Comm. of Large Dams—5th Congress—'Considerations sur la comparaison entre les elements du prix de revient de quelques barrages italiens en beton,' that we send you under separate cover."

b) Laboratories Nacional De Engenharia, Civil, Portugal Manuel Coelho Mendes da Rocha.

"In replying to your letter of the 8th August 1956 I am sending you herewith the requested information about the cost of the concrete in some of the Portuguese dams.

"The prices are given in escudos. To convert them into dollars the equivalence is 29 escudos per dollar:

	Year of the beginning of the works	Amount of cement (kg/m3)	Cost of concrete per m3 (escudos)*	Cost of cement per kg (escudos)	Cost of forms per m2 (escudos)
Castelo do Bode	1948	250	313	0,56	87
Venda Nova	1949	250	346	0,60	---
Cabril	1952	250	351	0,56	85
Salamonde	1952	250	337	0,60	---
Canicada	1953	250	345	0,60	---
Bouca	1954	250	372	0,69	50

* Cost of forms not included.

c) Andre Coyne & Jean Bellier, Paris, France
Symposium on Arch Dams—translation:

"We are answering your letter of 8 August 1956 in M. Martin's absence.

"The question of the data required on European concrete costs is a somewhat difficult one for the price per cubic meter depends on the costs of the auxiliary work such as excavation and formwork Contractors do not always distribute their remuneration in the same way under their various items. Their overheads and amortization of plant are not easily broken down with accuracy. In France, moreover, the question is complicated by the instability of the Franc.

"Here, however, are some prices for concrete in place in Arch Dams (including cement).

"(1) The cost given in F/m3 (Francs per cubic meter) is that at the date given in the relevant column.

"(2) The figure in brackets is the unit price.

"We also refer you to Report by Mr. Xerxes on Question 17 of the 5th Congress on Large Dams where you will find Portuguese prices:

Name of Dam	Volume (cubic meters)	Date (1)	Cost F/m3(2)	Remarks
La Chaudanne	27,000	1950	13,600	Comprising: formwork, plant, overheads Excluding: excavations appurtenant works
Bort	705,000	1950	12,800 (4,750)	Comprising: Formwork, plant, overheads Excluding: excavations appurtenant works.

Name of Dam	Volume (cubic meters)	Date (1)	Cost F/m3(2)	Remarks
Enchanet	66,000	1952	6,900 (4 300)	Comprising: Formwork, excavations Excluding: appurtenant works, plant, overheads
Chastang	250,000	1948	8,000	Comprising: formwork, plant, overheads, excavations Excluding: appurtenant works
Tignes	630,000	1952	9,500	Comprising: Formwork, plant, overheads, excavations Excluding: excavations appurtenant works
Castillon	125,000	1950	15,000	Comprising: Formwork, plant, overheads, excavations Excluding: appurtenant works
Rassisse	12,500	1954	14,000 (7,300)	Comprising: Formwork, plant, overheads, excavations Excluding: appurtenant works.

Contract bid prices indexed to January 1952 for mass concrete of several dams in the United States are shown in Table I.

The unit costs of mass concrete in several countries of volumes less than 1,000,000 C.Y. based upon information described above are as follows:

Country		Unit Cost including cement except as noted $/C.Y.	Exchange rate per dollar used
Switzerland		12.50	4.28 Swiss Francs
Italy	(1)	8.00	620 Lire
Portugal	(2)	9.00	29 escudos
France	(3)	10.40	350 Francs
Australia	(4)	17.81	0.447 pounds
United States		15.60	

(1) Cost of cement not included

(2) Cost of forms not included

(3) May be a misunderstanding of information due to translation. Assumed that cost of Bort Dam concrete of 4,750 Francs per cubic meter was correct.

(4) Cost of cement not included (Tumut Pond Dam, 170,000 C.Y.)

CONCLUSIONS

The above limited data indicate tentative conclusions as follows:

a) Unit cost of mass concrete in the U. S. is higher, including both labor and materials, than in Europe.

b) Amount of cement per unit volume used in European dams is greater than in the U. S.

c) Possible efforts to reduce unit costs in the U. S. offer slight hope of improvement:

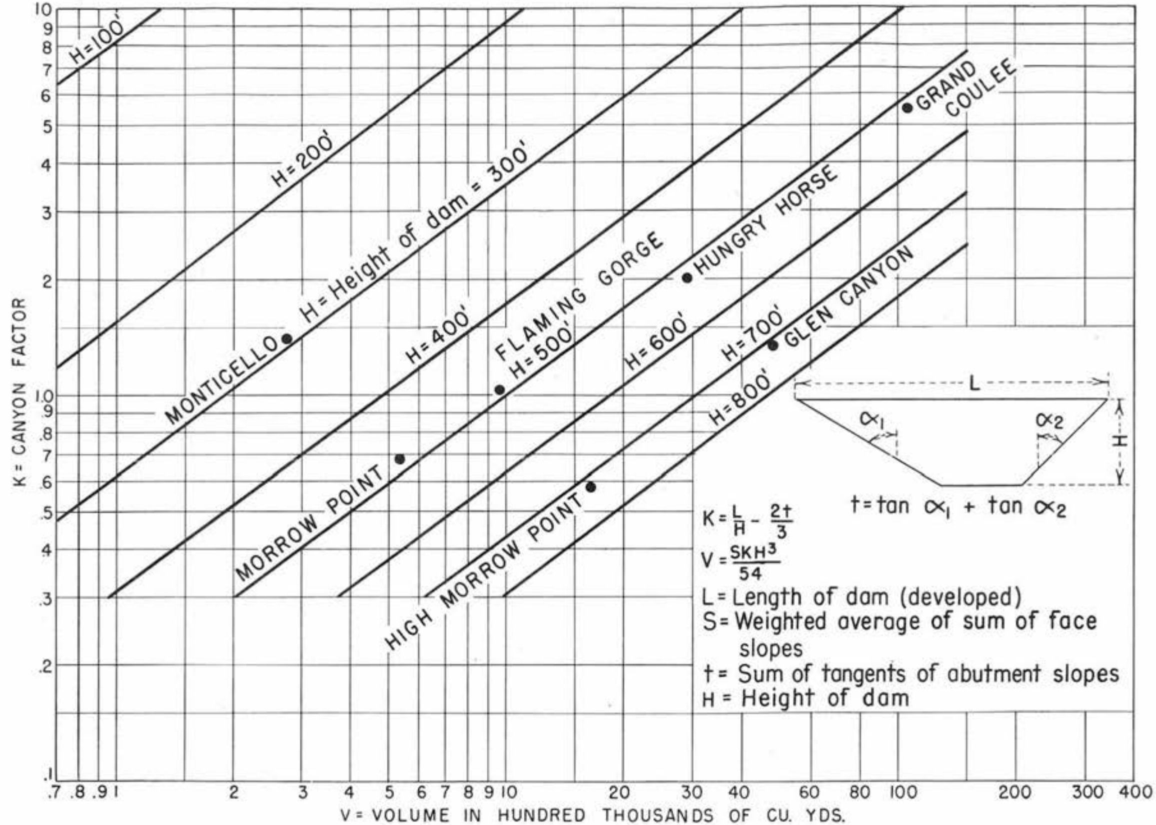
1. The present use of about 280 pounds or less of cementing materials per c.y. is minimum to obtain satisfactory quality.

2. In view of present policy to design for maximum stress of 1000/lbs.-sq.in., where applicable, any further reductions in volume would induce higher unit prices and tend to compromise factors of safety.

3. The dome type of dam, generally used in Europe, would probably demand higher unit prices than the vertical arch type used in the U. S. where labor prices are high and are not receptive to additional complicated formwork.

TABLE I COST FOR MASS CONCRETE IN DAMS (U. S. BUREAU OF RECLAMATION)

Year	Dam	Volume C. Y.	Unit Price $/C.Y. Low bids			Unit Price $/C.Y. Indexed to Jan. 1952		
			Excluding Cement	Cement	Total	Excluding Cement	Cement	Total
1931	Hoover	3,400,000	$2.70	$1.60	$4.30	$8.20	$3.95	$12.15
1934	Grand Coulee	3,100,000	3.00	1.90	4.90	8.35	4.25	12.60
1937	Grand Coulee	5,500,000	3.53	2.07	5.60	8.26	4.24	12.50
1938	Shasta	5,400,000	4.00	1.95	5.95	9.36	3.99	13.35
1939	Friant	1,850,000	2.48	1.97	4.45	5.80	4.00	9.80
1946	Kortes	130,000	10.00	3.75	13.75	15.90	4.50	20.40
1948	Hungry Horse	2,900,000	7.00	3.50	10.50	8.40	3.85	12.25
1949	Canyon Ferry	390,000	11.00	3.00	14.00	12.65	3.25	15.90
1953	Monticello	260,000	12.90	3.20	16.10	12.25	3.15	15.40



ESTIMATED VOLUME OF CONCRETE DAMS

Figure 1.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: DEVELOPMENT OF MODEL RESEARCHES IN ITALY

Guido Oberti,* M. ASCE
(Proc. Paper 1351)

FOREWORD

This paper is one of a group presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams (April, 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all. The papers in this Symposium are "Arch Dams: Their Philosophy," by Andre Coyne (Proc. Paper 959); "Arch Dams: Trial Load Studies for Hungry Horse Dam," by R. E. Glover and Merlin D. Copen (Proc. Paper 960); "Arch Dams: Portuguese Experience with Overflow Arch Dams," by A. C. Xerez (Proc. Paper 990); "Arch Dams: Theory, Methods, and Details of Joint Grouting," by A. Warren Simonds (Proc. Paper 991); "Arch Dams: Santa Giustina Single-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 992); "Arch Dams: Measurements and Studies on Santa Giustina Dam," by Claudio Marcello (Proc. Paper 993); "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 994); "Arch Dams: Isolato Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 995); "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-Offs," by Claudio Marcello (Proc. Paper 996); "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira (Proc. Paper 997); "Arch Dams: Development in Italy," by Carlo Semenza (Proc. Paper 1017); "Arch Dams: Design of the Kamishiiba Arch Dam," by C. C. Bonin and H. W. Stuber (Proc. Paper 1018); "Arch Dams: Observed Behavior of Several Italian Arch Dams," by Dino Tonini (Proc. Paper 1134); "Arch Dams: Measurements and Studies of Behavior of Kamishiiba Dam," by H. Kimishima and C. C. Bonin (Proc. Paper 1182); "Arch Dams: Construction of the Kamishiiba Arch Dam," by K. M. Mathisen and C. C. Bonin (Proc. Paper 1183); "Arch Dams: Review of Experience," by Robert E. Glover (Proc. Paper 1217); "Arch Dams: Stress Studies for Ross and Diablo Dams," by Joe T. Richardson and Owen J. Olsen (Proc. Paper 1267); "Arch Dams: Economy

Note: Discussion open until January 1, 1958. Paper 1351 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 4, August, 1957.

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of Concrete Dams," by Louis G. Puls (Proc. Paper 1286); Arch Dams: Development of Model Researches in Italy," by Guido Oberti (Proc. Paper 1351).

ABSTRACT

Experimental researches on models in Italy have greatly contributed to the national design of arch dams. Actual possibilities of models based on the theory of similitude are discussed. Important cases of Italian arch dams studied by models are described and principal results obtained are reported.

PRELIMINARY CONSIDERATIONS ON STRUCTURAL MODELS

Introduction

Research on models in the quiet atmosphere of a laboratory permits an accurate analysis of variables, and renders the experimental investigation simpler and less costly than on the prototype. Furthermore, it permits investigations of the static and dynamic behavior of the structure, predicting its real degree of security and realizing the best economy.(9)

The use of models is especially valuable when the mathematical solution of a problem is either unknown, extremely laborious, or difficult to define as to the boundary conditions.

The theory of models is based on a well-known principle of similitude which states that two systems are physically similar when there exists a geometrical correspondence between the points of the two systems, and the quantities of the same physical nature have a constant ratio at corresponding points. Complete physical similitude between prototype and model is reached when all the relations between the "scales" with which the model reproduces the physical quantities, on which the problem depends, are taken into consideration.

If there are n physical quantities upon which the problem depends and we choose the q functions which are fundamentally and dimensionally independent, corresponding to the degree of dimensional freedom of the problem (3 for mechanical problems), it is always possible to get $n - q = m$ dimensionless π ratios, which correspond to the m quantities relating each of these and the q functions assumed as fundamental. Having then chosen the dimensionless π_1 ratio relative to the quantity that one particularly wishes to know, this ratio becomes then the function of the remaining m - 1 dimensionless ratios $\pi_2 \ldots\ldots\ldots \pi_m$. The model will therefore correspond to the prototype in all respects if the values of these ratios remain unchanged in passing from the prototype to the model, and the π_1' ratio for the model will then also be $\pi_1'/\pi_1 = 1$; a relation which permits, having measured the quantities on the model, to obtain the corresponding value on the prototype.

In addition, the inherently dimensionless physical constants upon which the problem may depend must be conserved in passing from the prototype to the model, for example: Poisson's ratio, the coefficients of friction between the various materials, and so on.

It is useful to distinguish between the cases that one may wish to study on a model, separating those for which one possesses a thorough mathematical theory, from those in which this is not so. A classical example of the first case is presented by a study of the behavior of any structure made in a homogeneous isotropic elastic material and bounded in a statically determinate way, as mathematics then offers the theoretically complete solution to the problem: a system of differential equations having as unknown functions the stresses.

Even though the numerical solution of such a system is often extremely laborious, as for example in the most complex problems of concrete dams, the knowledge of the theory simplifies the investigation with the model as the equations of the theory furnish a complete and precise list of the quantities which influence the phenomena studied. In particular, it should be remembered that such equations postulate the independence of the stresses from the physical-mechanical characteristics of the material (elastic modulus) yield point (with the exception of Poisson's ratios for three dimensional problems) therefore permits the use of model materials which differ from those of the prototype, as applied in photo-elasticity.

Without a theory and the relative equations which set out the physical problem to be studied, it is more difficult to realize a complete similitude, as naturally all the non-dimensional ratios on which the phenomenon we study depends, may not be identified. It is in these cases that dimensional analysis, as a precious resource, becomes of use, in fact, once the quantities which are present in the phenomenon are listed, it provides us with the independent dimensionless ratios which can be built with these quantities and thereby provides a guide to the proper use of the model to obtain satisfactory results.

It seems advisable to mention these preliminary fundamentals of the model theory in order to outline the difficulties that beset work on models. Such difficulties are often present in hydraulic, electric and aerodynamic researches.

The Structural Models

a) Structural investigations generally present favourable conditions since the independent fundamental quantities upon which the behaviour of a structure depends are generally three; the classical "length," "mass" and "time," or three equivalent dimensionally independent quantities and that these are reduced to two when only the static behaviour of the structure is studied, as in this case the variable "time" is missing.

If λ and x represent respectively the ration of similitude of the lengths and of the forces, the ratio of similitude ζ between the stress values, during the passage from the prototype to the model must be:

$$\frac{\lambda^2}{x} = \zeta \tag{1}$$

All the other physical quantities, occurring in the problem, which have the dimensions of a stress (modulus of elasticity, yielding value, failure unit loads) must then have this same ratio. The materials which the models and their foundations are built, must generally conform to this same ratio which we will call "effectiveness ratio." (In the elasticity problems only and within the limits of the mentioned theory, this dependency may be avoided. For example, photo-elasticity utilizes materials for models which are quite different from those of the prototype.)

In the particular case of only superficial loads, with π indicating the ratio between the intensity of these loads the required relationship will be: $x = \pi\lambda^2$ and here π coincides with ζ, the latter is then independent of the scale ratio λ.

But if the stresses due to dead weight are not negligible, and ρ represents the ratio of the densities, then it will also be necessary that

$$x = \rho\lambda^3 \tag{2}$$

and therefore the condition that is obtained by placing (2) into (1) must be considered. This is

$$\zeta = \rho\lambda \tag{3}$$

The difficulties are increased by this requirement which may justify the expedient of using large scale models and also to increase—with artificial devices—the density of the model material.

In the particular case where the most important stresses are due to body forces, as for example, in dam problems (hydrostatic load and dead weight) only the relationship (3) is required.

When it is possible to find materials to build a model and its foundations for which the conditions of invariability of ζ are met, in the sense that the "intrinsic curve" of the model-material is similar to that of the prototype in the constant ratio ζ and the scale of the density ratios satisfy the relation (3), similitude may be considered attained and it may be then considered effective not only within the limits of elasticity but as far as to the breaking point.

This result, which the author reached some twenty years ago, permitted, with the use of convenient materials, to study on models with tests up to the breaking point, the degree of safety of important structures and in particular that of practically all the big Italian dams designed and built in the last decade.

For several years plaster of Paris had been employed, but it was rather difficult to use, specially when the models were large and thick, because the plaster took a long time to dry and the interior was not uniform. That was the principal reason which led to the use of a special mixture for the most recent models. It was a special concrete in which the aggregates were volcanic pumice stone from the island of Lipari. Such stone is cheap in Italy and is therefore very useful for laboratory tests. Using volcanic stone, tests had been made with different percentages of cement, and it is now possible to construct a model of this special mixture having a large variation in the ratio of its modulus of elasticity to the modulus of elasticity of the concrete of the dam.(11) In some cases the ratio is as low as one to twenty. Furthermore, in the large models it has been possible to include the natural rocks in the model, incorporating the same ratio as existed in nature. This was done by means of research made in the field by means of special devices, particularly for limestone. The necessary information having been gained by measuring "in situ" the deflections (in all directions) caused by pressure in a large tunnel which had been excavated in the interior of the mountain.

When one has to undertake experimentation on models of this kind it is convenient to begin the research work directly on the strains ϵ, assuming them as fundamental unknown quantities (instead of stress components); these being already dimensionless will insure that a complete similitude will be obtained when they are equal at the corresponding points of the prototype and of the models. All the precedent considerations of the case are still valid and, in particular, the relation (3).

Then the strains on the model, which it is possible to measure with a high degree of accuracy, by the use of extensometers having a very high magnification (fig. 1) will be equal to those of the prototype.

It is thus learned that the displacements and in particular the deflections (that dimensionally are $\epsilon \cdot \iota$), will be proportional to the scale ratio λ between the prototype and the model (fig. 2).

b) Previously when static problems alone were considered, deformations and stresses were not influenced by "time."

In reality the collapse of a structure may also depend, even if in a different degree, on the time of the application of loads because of the viscosity of the materials of which they are built.

It is known from theory that, in such cases, the stresses are linearly related to the corresponding velocity of deformation through a coefficient of viscosity, which is a constant providing the material is homogeneous and isotropic. It is then necessary to add a new fundamental quantity, such as "time," for which the non-dimensional ratio μ between the viscosity coefficients must be equal to $x \lambda^{-2} \tau = x \zeta$ with τ representing the ratio of times between the prototype and the model.

The variable "time" comes into play again when the dynamic behaviour of a structure must be studied. This is of particular interest for seismic effects. In such cases it is necessary to bear in mind that among the forces to be considered in the first place are those of gravity, and being unable, obviously, to alter the value of the gravity acceleration in passing from the model to the prototype, one is obliged to presume that this quantity is a fixed dimensional constant. It is useful therefore to consider acceleration as a fundamental quantity to add (instead of time) to the two preceding ones and the ratio between the accelerations acting on the prototype and in the model must be equal to one. The ratio of the times τ must then satisfy the condition:

$$\tau = \sqrt{\lambda} \tag{4}$$

It follows that the vibrations will reproduce themselves on the model with a higher frequency. For example, on a model having a scale of 1:100 the frequencies will be ten times as high as in the prototype (Froude-similitude).

It is useful to point out that in order to satisfy relation (4) it is not possible to work a model made of the same material as the prototype (or which has the same density) since, said ρ the ratio between the density of the prototype and model materials, must be in any case (the forces being dimensionally equal to the product of mass times acceleration):

$$\tau = \lambda \rho^{\frac{1}{2}} \zeta^{-\frac{1}{2}} \tag{5}$$

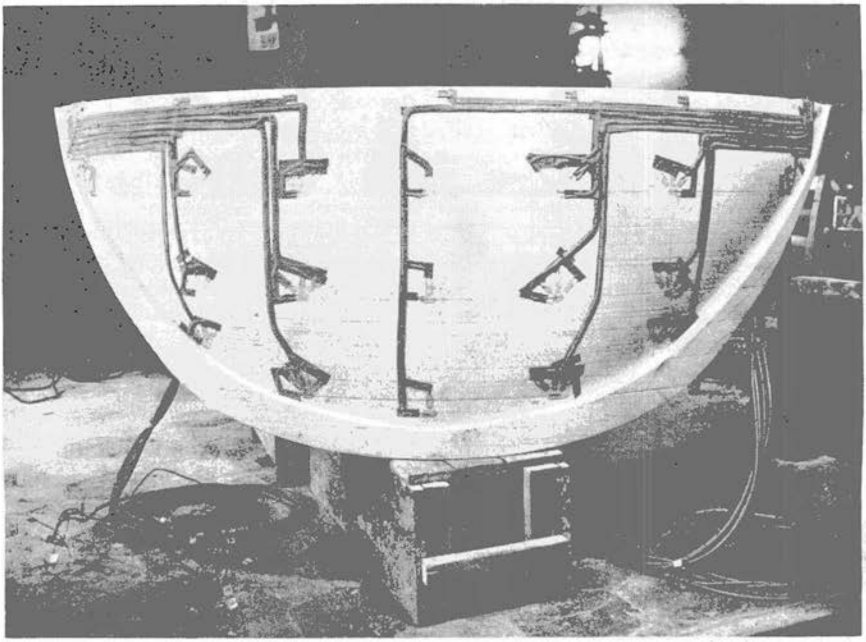

Fig. 1. Electrical Strain Gage Equipment in Place on the Upstream Face of an Arch Dam Model.

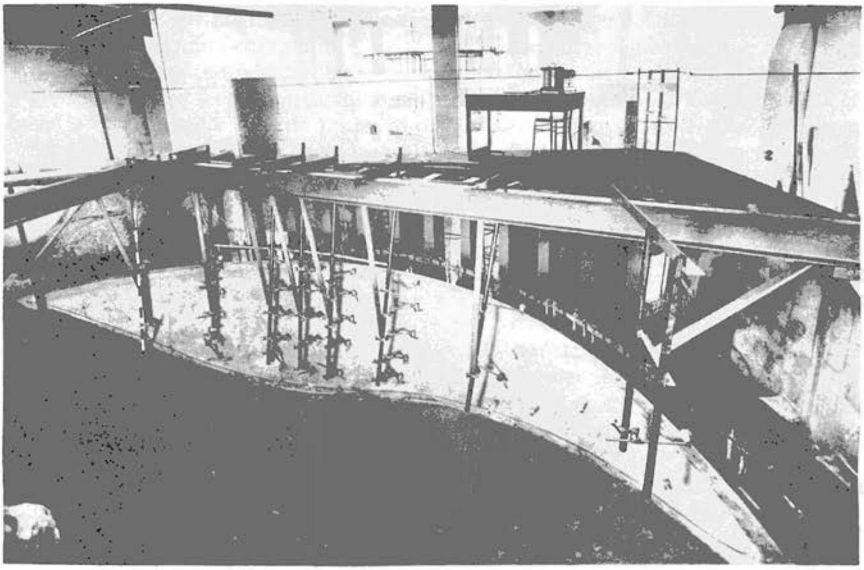

Fig. 2. Application of Dial Gages on the Downstream Surface of a Large Arch Dam Model.

The equations (4) and (5) are satisfied by imposing the condition (3).

As an example, in the case of the model studies of the earthquake-effects on the Ambiesta dome dam, by conforming to the requirements of these fundamental ratios, we have assumed

$$\lambda = 75 \qquad \zeta = 50 \qquad \rho = 2/3$$

using for the model a special mortar of litharge and plaster.

c. With regards to what is assumed in order to obtain by calculations and by direct research conducted on the real dam, models have been objected to on the grounds that internal stresses are not examined and that thermal effects are not studied.

For the first question it is pertinent to point out that—as a rule—the stresses of more interest are found on the surface. Presently, internal measurements at single points in models are possible with a good accuracy.(12) Furthermore, if one limits himself to the elastic behaviour of plane structures, it is also possible to obtain by photo-elasticity, the stresses in the interior.

With regards to the second point, it may be interesting to remember that, should the thermal variations (or its equivalent as shrinkage) not be directly modelled but are linear functions of the coordinates, the thermo-elastic problem may be replaced for elastic conditions, by the artifice of applying on the model suitable volume or surface forces instead of the thermal actions (and with the precaution to measure also the deformations, which the model, being free from redundant ties would develop under the action of such ideal forces.)

Generally speaking, if the deforming actions are deduced to simple Volterra dislocations, it is possible to employ models. Only if one passes to more general types of distortions, does the use of models become really arduous as it is then necessary to reproduce and to model the local physical causes which introduce such distortions.

The Evolution of Experimental Methods on Models

In Italy the experimental procedures on models have been progressively extended in these last years and I would divide them, following their chronological evolution, into three groups of substantially distinct methods.

a) With the methods used in the first group the plain elastic problems are studied: prevailingly with the aid of photo-elasticity and deformeters. Photo-elasticity is a particularly elegant system which permits us to obtain photographic reproductions of stress patterns, lines of uniform shear stress, and to deduce the stress-trajectories (isostatic lines) in the prototype by the experimental observation of the isoclinics obtained in the model. This method of research is now successfully used in many laboratories which deal with structures.

The deformeters, developed from the first Beggs types, are also tools of great utility.

b) The methods of the second group investigate three-dimensional elastic problems with extensometer measurements; that is by applying directly to the model mechanical, optical or electrical extensometers.

Many structures were thus studied in Italy (especially by the author in the laboratory of testing models in the Polytechnical School of Milan) by adopting model materials (celluloid, plexiglas, etc. . . .) which were very different from those of the prototype provided that they are elastic. In these tests the continuity of the structure, the rigidity of the abutments or the elastic deformability of the ties were accounted for. In arch dam models the load was applied usually with quick-silver (fig. 3).

Under this condition the model functions as a "mechanical calculator" of stresses and gives results that may be usefully compared with those of the various calculations made for the study of the elastic behaviour of the structure.

c) Lastly, with the third group, having ascertained that some structures, particularly those of concrete, do not conform to the postulates of elastic theory, and that the noncompliance gives better results, it appears preferable to search for a more exact similitude with the prototype rather than for confirmation of elastic calculations. This is a decisive step towards conformity with nature which characterizes the considerable amount of work done by the ISMES in Bergamo in building models, which tend to substantially reproduce not only the behaviour of materials but also the particulars of the form and the performance of the prototype, of the bearing seats and the real deformability of the ties and of the foundations.

The tests made on the model may then be divided into two distinct and successive steps. In the first series of tests which we call "normal load tests" deformations are measured in order to obtain the values nearest to the condition of similitude, which will impose equality of the strains on the prototype and on the model, under normal load conditions corresponding to those of the structure in service. It is important to point out that during the application of the load various types of inelastic adjustments or permanent sets may take place which is good to stimulate, by repeating load cycles, reaching a regime working of the model which will be elastic, regular and suitable for the measurements of deformations and for useful controls. It is then possible to evaluate the stresses (knowing the stress-strain diagram of the material) and the static behaviour foreseeable in the prototype during normal service.

It must be observed that the stresses thus observed may not agree with those deduced, by calculations, since the adjustments (which generally have a beneficial effect) are not taken into consideration.

Having finished these tests and the relative measurements one passes gradually to the destruction tests or ultimate load tests. It is convenient then to assume as the overall safety coefficient of the structure, the ratio K_s between the value of the maximum load actually supported and that considered normal during service.

In the particular case of arch dams, having made the ultimate load tests on the model (conducted until collapse or until the first signs of cracks appeared on the upstream face), the safety factor will simply be the ratio between the maximum final value γ'_m of the specific weight of the liquid, fictitious or real, acting on the model, and the value γ' relative to the liquid conforming to the foreseen "normal load" (which is realized in the dam for the designed maximum water level). If ζ is the "Ratio of Effectiveness" between the material of the dam and that of the model, λ is the scale ratio, $\gamma_o = I\ T/m^3$ is the specific weight of water acting on the prototype, the following relations will hold:

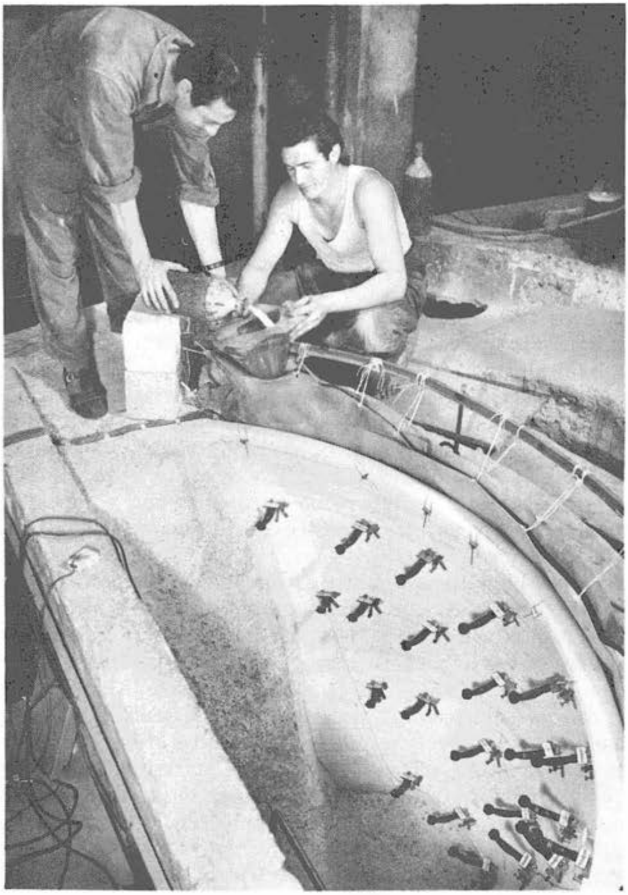

Fig. 3. Model of Arch Dam Tested with Mercury in a Rubber Tank Acting on Upstream Surface.

$$\gamma' = \frac{\lambda}{\zeta} \gamma_o \quad \text{and:} \quad K_o = \frac{\gamma'_m}{\gamma_o} \frac{\zeta}{\lambda} \tag{6}$$

In the case of gravity or arch-gravity type dams in which the effect of weight (also a volume load) has the essential function of maintaining stability, it is necessary to arrange the model for its increase and to maintain it at the correct ratio in respect to the hydrostatic load (Fig. 6). It is with this precaution that we may proceed with the ultimate load tests. If in these tests a safety ratio, which is considered sufficient under normal conditions, is reached without collapse of the model, it may be interesting to continue increasing only the hydrostatic thrust and not the weight in order to learn the strength at exceptional thrusts (such as earthquakes, bomb effects, and so on).

These latter methods in our opinion represent a notable progress in comparison with the former which, on the other hand, are still useful especially as a means of comparison with the theoretical results. In fact the methods of the first two groups are based, on a set of hypothesis, that lead to results which may not conform to reality since they would only hold for an ideal structure which faithfully obeys the initial hypotheses.

Instead, our methods, rather than obeying preconceived idealizations come closer to the reality of a specific case. Thus we do not hesitate to introduce into the model materials, foundations, ties and joints and the general building particulars, which may prohibit the possibility of an analytical check (and producing sometimes a certain dispersion of results) but, compensate by a better agreement with the real case and therefore adhere closer on the true and final aim of the designer.

This point of view permits and justifies the use, in these third methods, of useful devices which may produce local troubles which are negligible in respect to the final aim of the tests. Thus, for instance, the hydrostatic load on the up-stream face of the dam models may be applied with hydraulic jacks provided with suitable diffusion plates instead of liquids; also the corrective addition to the dead weight may be concentrated in a number of points instead of being distributed continuously on the whole volume of the structure. With similar practical means (the influence of which is possible to check experimentally) the conditions of similitude are retained in the model and problems, which otherwise would be difficult or unsurmountable, are resolved.

This conclusive phase of experimentation which is more comprehensive and delicate than the preceding ones, requires a critical attitude of mind, a good experimental ability, a patient research on, and preparation of materials suitable for the construction of models, and coatings capable of preserving them, specially from shrinkage, during the tests. This is what has been developed and studied in these last years in the ISMES laboratories.

MODEL AND STRUCTURAL TESTING AT THE I.S.M.E.S.

1. In order to fully appreciate the function performed by the Instituto Sperimentale Modelli e Strutture, "I.S.M.E.S.," (Model and Structural Testing Institute) of Bergamo (Lombardy Region), it should be noted that in Italy most of the study and research work in the field of civil engineering is done by University Institutes. Their work is largely theoretical, and in the field of

experimental research it is confined chiefly to material testing, due also to the limited facilities and staff available for this work. The I.S.M.E.S., instead, is a private corporation established for the purpose of solving, with adequate financial means and the freedom of action which, by their own structure, University Institutes do not possess, specific structural problems arising for designers and builders. Therefore, I.S.M.E.S. carries out a technical-scientific activity which is, so to say, complementary to the activity which is carried out, or should be developed, in the Italian University Research Laboratories.

I.S.M.E.S. was established by a group of companies and contractors including: EDISON Co. of Milan, ITALCEMENTI Co. of Bergamo, Acc. FALCK Co., SADE Co. of Venice, SIP Co. of Turin, "Societa Italiana Partecipazioni Industriali," SME Co. of Naples, MONTECATINI Co., ROMANA ELETTRICITA' Co., SELT-VALDARNO Co., TERNI Co., ACEA of Rome, AEM of Milan, AEM of Turin and the contractors: GIROLA, ITALSTRADE, LODIGIANI and TORNO. The scope of the Institute's work extends to experimental research on the behavior of structures, by means of tests conducted on large three-dimensional models, or on the structures themselves at the construction site.

The experimental study of the structures by means of models has been gaining momentum in the last few years, as a result of the improvement of measuring instruments, and has gained increasingly wide acceptance as an effective aid by open-minded designers and builders.

The practical usefulness comes from the fact that it is possible to work out a solution for complicated structural problems even in the cases where calculations cannot provide sufficient assistance.

This process yields, in the design stage, valuable information making it possible to anticipate the static or dynamic behaviour of the structure and, if necessary, to select among several designs the solution which is likely to produce the highest efficiency and the lowest construction cost. In addition to tests under normal load conditions, the Institute usually carries out ultimate load tests intended to yield an indication as to the order of magnitude of the overall safety-coefficient of the structure modelled.

So much for what we can define as the "technical" work which the Institute is called upon to do for its customers in Italy and abroad.

In the field of pure scientific research the Institute studies improvements of the techniques for research on structural models. In this field, some results of fundamental value have already been achieved, such as the extension of model testing beyond the elastic limit, thanks also to the development of suitable materials and measuring instruments.

The Institute officially was established in 1951, and is in a phase of continuous development. Among its equipment are special structures of heavily reinforced concrete, built to contain large models or structural elements to be put through static tests, and to support without appreciable deformations the loads involved in the testing.

One of these structures is a rectangular-base tank measuring approx. 32 x 16 feet, particularly suitable for testing model dams; and the other is a circular-section tower, 32 feet interior diameter and 60 feet high, designed to hold tall models (dams built in narrow gorges, skyscrapers, cement silos, etc.). The tests on models or structural elements exerting no thrusts above ground level, i.e., resting or anchored upon level ground (penstocks, floors, etc.) are conducted in a large shed-type building.

A new department for research on the physical-mechanical characteristics of concretes, including those with large-size gravel, has been activated last year. It is equipped with a 2,000-ton compression and bending materials-testing machine, which can also be used for studies on various structural elements (pillars, girders, etc.)

The Institute possesses an extensive set of loading devices (hydraulic jacks, springs, etc.) to apply on model or structures any kind of stress, such as dead loads corresponding to their own weight; accidental loads, hydrostatic loads, wind pressure, etc. Its laboratories are also equipped with a complete set of measuring devices (bending gauges, stress gauges, recording units, etc.) and with auxiliary equipment (including a photo-elastic section for research in two-dimensional elastic fields) which are used, in specific problems, to supplement the research on three-dimensional models.

A set of loading and measurement devices was developed for field tests. Particularly interesting is the equipment for the determination of the deformability of foundation rock by underground tests (special jacks, pumps, waterproof gauges), which is one of the fields in which the Institute specializes.

The Institute has ready for use from this year a set of equipment, the only one of its kind in Europe, which is used for studying on models the effects of seismic (or, generally, vibrating) actions on building structures and dams. The seismic testing devices include:

a) A metal platform, measuring approx. 10 x 15 feet, of highly rigid construction, upon which will rest the structure to be tested, complete with foundations (Fig. 6);

b) Three vibration-generating units, including:
   1. A variable-constant pendulum and spring system for the reproduction of shock waves;
   2. A centrifugal vibrodyne for the reproduction of unidirectional harmonic movements, capable of applying forces up to 10 metric tons;
   3. Four electronic-controlled electro-magnetic vibrators, for the reproduction of both unidirectional and vortex-type movements;

c) An equipment for recording the strains and distortions of the model during the testing.

Better than any list of equipment, however, the capabilities of the Institute can be illustrated by the record of research work done.

2. Systematical tests on models, including some of very large size, have been conducted by the Institute, particularly for large dams.

The models of dams are built and tested in accordance with specifications which are the product of long experience, and therefore usually applied to different models. The following is a brief description of the processes and specifications usually adopted for arch dam tests.

The first step is the construction of a preliminary model, of plaster or wood, which is used to study the design details required and for the construction of molds for casting the actual model. When necessary, this preliminary model is also used, on completion of tests, to reproduce the principal-stress trajectories.

The actual working model is laid on a foundation bed of the required "modulus," and its construction follows quite closely actual prototype construction processes. Among other details reproduced are the joints to be grouted by injection when the setting process is completed (Fig. 4). Concrete

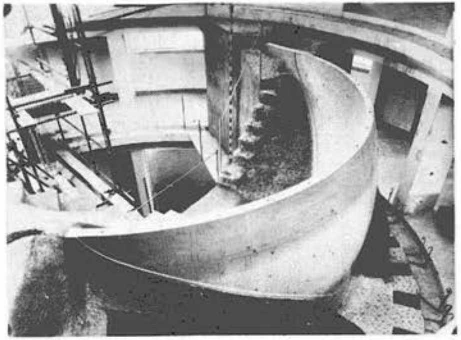

Fig. 4. The Model (1:50) of the Beauregard Arch-Gravity Dam Immediately after Casting. The Holes for the Injection of the Joints May be Seen.

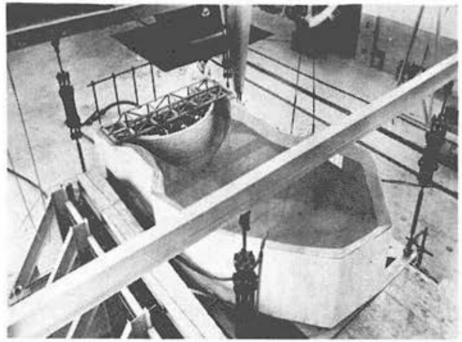

Fig. 6. Vibrating Table for Earthquake Tests. Model (1:75) of the Ambiesta Dome-Type Dam Ready for Seismic Tests.

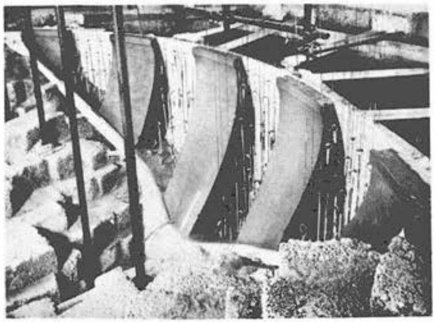

Fig. 5. Casting a Model Arch-Gravity Dam; Tie Rods for the Application of the Dead Load are Seen.

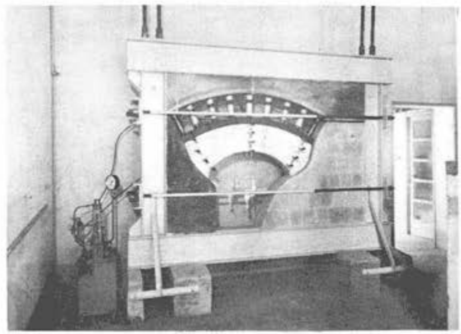

Fig. 7. A Hydrostatic Load Test, by Means of Jacks, on a Single Arch Model with Abutments on Two Different Kinds of Rock (Modulus Ratio 1:10): Preliminary Tests for Beauregard Dam.

is reproduced by suitably dosed cement and stone pumice mix, approximating the mechanical characteristics of the actual rock and construction materials.

Before pouring, fastenings for additional own-weight loads are inserted at appropriate points in the body of the models (Fig. 5). On completion of the hardening process, vertical loads are applied by means of spring-loaded dinamometers (Fig. 6). Water pressures are usually reproduced by means of hydraulic jacks, fitted with special attachments which distribute the load over a sufficiently wide surface. In elastic tests, local measurements are taken by means of extension gauges of various types when spot readings are possible or by centralised electrical indicating systems. Mechanical dials (deflectometers) are also used to measure bending deflections and overall shifts in the structure.

On completion of the normal tests, which lead to the determination of the stresses and, as a rule, to the tracing of the stress trajectories on the model structures, destruction (ultimate load) tests are conducted. The loads are gradually increased until the model collapses, yielding the value of the overall safety coefficient.

In the individual practical cases, the technical and financial importance of this research has always proved substantial. As for the financial side of the problem, we would like to mention here as an example the fact that the results of tests conducted with the first model built (Pieve di Cadore Dam) have led to reducing the overall volume of the structure by about 15%, with a saving in the order of $1,600,000.

In some cases the scope of the experimental research was extended to include a comparative study of different alternate solutions. This was done, for instance, in the case of Osiglietta arch dam (acc. Falck Co.) and more recently for the SADE Company's Fedaia Dam, when model tests were conducted on two different designs of arch-gravity dams, and on a third (then actually adopted) for a buttress dam.

In other cases, model tests were conducted for the purpose of observing the influence of particular conditions on the static behavior of the structure. Thus, for instance, in the case of SIP's Beauregard Dam, the Institute's research staff has reproduced, after overcoming considerable difficulties, the pronounced difference in elasticity of the two valley sides (with elasticity coefficients in the ratio of 1:10) (Fig. 7). In the case of the Cancano Dam (AEM), adjustments were made to allow for the heterogeneous structure of part of the rock on the right abutment, reproducing the precise lay of the rock layers.

For the Val Gallina dome dam and for the arch-gravity Piave River Dam, a comparison has been conducted between the deflections observed on the model and that of the actual structure, and the results were found to substantially agree.

For the Giovaretto buttress dam, a study has been conducted, by means of model, of the stresses in the highest spur, extending it to the inner part around a lightening and drainage hole, around which the existence of tension stresses was feared. Studies of the same type were conducted on the arch-gravity Cancano Dam mentioned above as regards the concentration of stresses around some large tunnels and the elevator shaft.

For the Falck Steel Company, studies have been conducted on three models representing the arch-gravity Frera di Belviso Dam, which is designed to be built in two separate stages (Fig. 8). Because of the particular shape of the joint between the successive construction stages, the static behavior of the

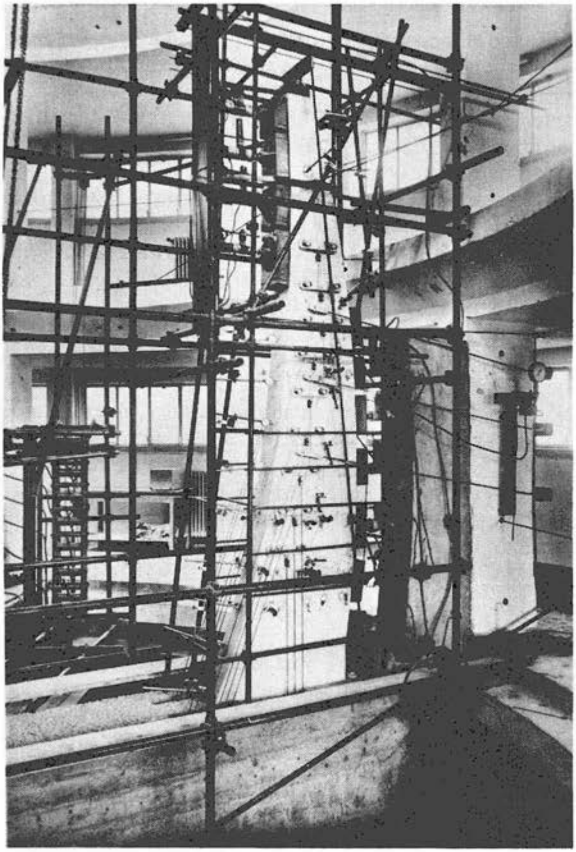

Fig. 8. Testing of a Single Cantilever Model of a High Arch Dam Designed for Construction in Two Steps with a Longitudinal Joint.

completed dam involved problems which could be studied only on an experimental basis.

Outside the sector of dams, different structures and structural components have been investigated. For instance, studies were conducted into the actual pushing action exerted by pulverulent materials (cement) in tall storage silos. Researches were carried out, by laboratory and site tests, on the static behavior of pre-stressed concrete penstock and acqueduct pipe, in different practical cases involving pressures from 50 to 350 tons per square meter. The Instituto has already completed the experimental tests on a 1:15 scale model of the skyscraper which will be built in Milan for Pirelli Co. to a height of 450 feet above foundation level. The reinforced concrete structure was modeled in its essential lines, basically respecting cross-sections, moments of inertia of the individual parts and the rigidity of the joints. The model was built of a pumice-cement mixture reinforced with iron wire and mesh. For the foundation slab, the pre-stressing framing was also reproduced. Particularly interesting were the wind-pressure tests, conducted by measurements under static load by recording of the dynamic effects. In addition to the stress pattern, these tests made it possible to ascertain periods and accelerations involved in connection with bending and torsion oscillations.

The basic of this brief and incomplete review of the activities of I.S.M.E.S. is to show, in the light of facts, that testing structural models is not only useful towards the distant goals of progress, but also serves the cause of immediate economy. It is worth recalling—as written by Prof. Danusso, the Institute's President—that we are still forced to use but a small fraction of the actual resistance of the construction, since we must provide a wide margin of safety against our own ignorance. This margin can be largely reduced by using models, which come much closer to reality than any calculation, and therefore can give much better answers for many construction problems, not only in arch dams.

TESTS ON ARCH DAM MODELS AND THEIR RESULTS

Two-Dimensional Models—Photo-Elastic Tests

Early tests (1935-36) were carried out employing photo-elasticity as a means for investigating the problem of the elastic behaviour of the arches of dams having a heavy thickness, which was particularly interesting for the designing of a large pure arch dam, that of Santa Giustina (Edison group Co.).

The arch was assumed to be elastic, continuous, and set on a rock yielding elastically with the same modulus as that of the dam material. Many arches were investigated with a circular upstream face outline, both at constant and variable thickness and with various openings, from about 60° to about 120°.(1)

From the analysis of the results and their comparison with those of the ordinary elastic calculation, it was possible to draw the following general conclusions:

a) The elastic calculation gives results which are close to the experimental ones for arches having average slenderness ratios (radius/thickness) higher than 3, and having notable central angles. Decreasing such central angles (arches narrower than 90°), it was observed that the calculation leads as a rule to higher tensile stresses at the abutment extrados, slightly lower, instead, at the intrados, both at the crown and at the abutment, with a closer agreement the higher is the slenderness ratio.

b) For thick arches of limited angular opening, only tests carried out on models can give positive results. Neither the elastic calculation, nor the quite simple one based on the (fairly common) assumption of assimilating such structures to ring elements are even approximately approaching reality.

The investigation emphasized also the desirability of designing arches with variable thicknesses, particularly when dealing with arches having small slenderness ratios and reduced openings.

The above considerations, although concerning facts today accepted everywhere, represented at the time of testing a substantial contribution to the designing of such high dams. These results, as well as other test results which will be briefly discussed further on, are here referred to essentially for their historical interest.

Three-Dimensional Models of Thin Arch Dams

a) Early investigations concerned a 70 m. high single curvature arch dam: the Rocchetta Dam, erected (by Falck Steel Co.) in 1937-38. The dam, set up directly against the rock banks, has angular openings varying from 100^{o} at the crown to 120^{o} at the bottom; thicknesses at the middle section varying linearly from 3.15 m. at the top, to 11 m. at the bottom.

The model (scale 1:40), besides being tested under normal hydrostatic load, in elastic conditions, was also subjected to endurance tests, up to collapse. The tests led to favourable conclusions about the static behaviour of the Rocchetta Dam and threw light on the manner in which this type of structure actually reacts to the water loads applied to it, and more particularly to check experimentally for the first time the great safety resources of arch dams. The following general conclusions were thus drawn for arch dams of a type similar to the one tested:(3)

1. If the structure has a high safety factor, as is the case with the Rocchetta Dam, a substantial contribution to strength exists, offered by the cantilevers, actually acting more like beams fixed at the bottom and elastically supported by the crown arch; such contribution slightly relieves the stresses in the horizontal arches.

2. The "ultimate load" tests showed that if the structure has not too high a safety factor, which would be the case if the ultimate strength of the concrete were low, or if the structure, remaining geometrically similar to the one under test, were much higher, it tends to work as by independent arches, disengaging from the cantilever effect. The presence of joints, the unavoidable slight settlement at the abutments, the plasticizing of the material, all favour the deflection of the arches, which, if heavily loaded, tend to work almost as individual arch-rings.

3. The usual calculation, which considered only independent and clamped arches, associated with the high safety factors imposed in Italy, did not agree with the static behaviour of the normal structure; it could show a better correlation, and limitedly to the superior part of the dam, if the safety factors of concrete were reduced.

4. The assumption made to consider the inferior horizontal section of the dam as comparable to ring elements uniformly loaded at the extrados does not agree with reality.

The analysis of the results obtained in this first investigation supports what had already appeared in the early tests on the Stevenson Creek Dam models about the considerable flexural stresses, acting on the cantilevers.

Keeping in mind also the balancing effect of their own weight, the idea arose quite spontaneously to no longer consider single curvature dams, but double curvature ones. The chance presented itself shortly after when, on behalf of the same Company, we were to study on a model the Osiglietta Dam to be built across a mountain gorge of about 215 m. span and 75 m. height, therefore very open and, in those days at least, exceptional for an arch dam. In order to compare the two types, the conventional, single curvature one and the double curvature one (dome shaped), it was decided to make parallel tests on models of the two types. In consideration of the character of the investigation, it was deemed convenient to use celluloid models and to apply the hydraulic loading with mercury. The tests were therefore limited to the elastic range.

The results obtained were extremely interesting and confirmed that the adoption of the second curvature did considerably reduce the tensile stresses along the cantilevers, improving the strength contribution of such vertical elements, from the fact that they acquire an effective curved shape. It was also observed that, in the specific case, given the high span-height ratio and the slenderness of the structure, it was convenient to increase the thickness of the crown arch. But another interesting fact emerged from the experimental study: the possibility of reducing the flexural stresses along the perimetral abutment of the structure caused by the clamping into the foundation rock, assumed here—as it was customary in those days—practically indeformable. Such a clamping effect is particularly harmful since it has a strong tendency to induce tensile stresses along the upstream face, as was clearly shown by the mentioned early tests on the models of the Rocchetta and the Stevenson Creek Dam.

The idea then occurred of radically modifying the tie condition, no longer by clamping the dam into the rock, but terminating it with a perimetral continuous joint against a concrete support saddle ("pulvino") solidly fixed to the rock. Tests carried out on a model with a perimetral joint fully confirmed the anticipated views, and quite safe stress values were obtained also on the downstream face.

A problem similar to the one above, rose with the Val Gallina Dam of S.A.D.E. which closed a large gorge having a somewhat similar shape as the Osiglietta Dam, but of larger size: 90 m. high and 240 m. span. An early model of this dam had been investigated at the Polytechnic Laboratory Structures of Milan (1947), but the results then obtained were later found to be partially changed by the variations introduced in the design of the dam at the time of construction as well as by the findings of further geological research which had emphasized the peculiar non-uniform condition of the dolomite foundation rock. For this reason, a second model was tested later, at the I.S.M.E.S. laboratory, on a 1:100 scale.[8]

The foundation rock for the whole of the volume which might have affected the stresses of the dam, i.e. for about 30 m. upstream, 60 m. downstream, and 40 m. deep, was then reproduced with the model. This zone was subdivided into single sections, by means of watertight thin layers, so as to permit the variation of the deformability of each section in agreement with the researches made at the site.

The model dam—cast in a pumice-cement mix according to the most advanced technique—reproduced all the alternations brought to the design during the construction of the dam; particularly, the increase of the thickness of the right side of the dam-abutment zone and the realization of the perimetral joint, which was complete along the sides and limited to about one-third of the thickness upstream at the central part of the dam.

From the elastic tests it was also possible to draw the isostatic lines (stress-trajectories) which showed the characteristic trend already noticed in testing other similar types of dams; this trend should confirm the validity of the method of calculation based on inclined arches, along planes approximately perpendicular to the average fibre of the main cross section, rather than on horizontal arches.

The existence of isostatic lines that do not follow the traditional "horizontal arches-vertical cantilevers" lattice, is confirmed for every single or double curvature dam having a high span-height ratio, by the mentioned tests on the Rocchetta, Osiglia and Val Gallina dams, which cast doubt on the "trial-load" or "Ritter" type calculations, based only on the "horizontal arches-vertical cantilevers" lattice.

In this connection, a certain interest attaches to the results of an approximate theoretical investigation which were based on the measurements taken on the last model of the Val Gallina Dam.

For an evaluation of the water load distribution among the various analyzable strength capacities in the dam—evaluation in which, in ultimate analysis, the comparison with the calculation methods consists—assuming as coordinate axes the horizontal one, x, according with the parallels (arches), and the other, y, according with the meridians (cantilevers)—the behaviour of the dam was assimilated to that of a shell defined by the inter-actions (per unit length of principal normal sections of the shell): N_x, M_x, M_y, Q, M_{tx}, M_{ty} which—admitting the linearity of the law of variation of the stresses inside the structure, a characteristic proper of thin vaults—are obtained from the stress test values.

N_x normal force in the arch element; N_y normal force in the cantilever elements; M_x bending moment in the arch element; M_y bending moment in the cantilever element; Q shearing force in the cantilever; M_{tx} and M_{ty} twisting moments.

$$N_x = h \cdot \frac{\sigma_m + \sigma_v}{2} \qquad M_x = \frac{h^2}{6} \cdot \frac{\sigma_v - \sigma_m}{2} \qquad N_y = h \cdot \frac{\sigma_m' + \sigma_v'}{2}$$

$$M_y = \frac{h^2}{2} \cdot \frac{\sigma_v' - \sigma_m'}{2} \qquad T = h \cdot \frac{\tau_m + \tau_v}{2} \qquad M_t = \frac{h^2}{6} \cdot \frac{\tau_m - \tau_v}{2}$$

where h is the vault thickness, σ_m' and σ_v' are, respectively the normal up-stream and down-stream stresses in the cantilever direction, σ_m and σ_v are respectively the normal up-stream and down-stream stresses in the arch direction, τ_m and τ_v are respectively the tangential up-stream and down-stream stresses.

By indicating with p_m the water load referred to the mean fibre, r_1, and

$r_2 = \dfrac{r_x}{\operatorname{sen}\phi}$ the radiis of curvature of the shell, the equations of equilibrium takes the form (5):

$$
\begin{aligned}
p_m = & \left(\frac{N_x}{r_2}\right) + \left(-\frac{1}{r_x}\frac{\delta}{\delta y}\left(M_x \operatorname{sen}\varphi\right) + \frac{\delta^2 M_x}{\delta x^2}\right) + \\
& + \left(\frac{N_y}{r_1}\right) + \left(-\frac{1}{r_x}\frac{\delta^2}{\delta y^2}\left(M_y r_x\right) + \frac{1}{r_1}\frac{\delta M_y}{\delta x}\right) + \\
& + \left[\frac{1}{r_x}\frac{\delta M_t}{\delta x}\frac{\delta r_x}{\delta y} + \frac{\delta^2 M_t}{\delta_x \delta_y} + \frac{1}{r_x}\frac{\delta^2}{\delta_x \delta_y}\left(M_t r_x\right)\right]
\end{aligned}
\tag{7}
$$

where the sums of the terms enclosed in brackets, functions of the internal actions previously obtained, indicate in the order: the axial strength of the arches, the bending strength of the arches, the axial strength of the cantilevers, the bending strength of the cantilevers, the torsional strength.

Deflections of the crown arch of the Val Gallina dam deduced from the tests on the final model, varying the deformability of the rock foundation as we have said, agree closely with that obtained directly with measurements made on the prototype.

At the end also the model of the Val Gallina dam was subjected to the ultimate load tests by increasing gradually the intensity of the hydrostatic load. The collapse came for a load about ten times the normal one, with interesting flow strains which were registered by means of oscillograph recorder equipment.

b) Together with the above described tests on comparatively slender and thin arch-dams, built across very open U-shaped gorges, the occasion presented itself for approaching the problem—having certain reciprocal aspects—of the arch dams for very high and narrow gorges with almost parallel walls. There were two principal instances to be considered in Italy: that of the Santa Giustina Dam (Edison Group Co.), having a height of nearly 150 m. and a maximum span of about 80 m., and that of the Vajont Dam (former solution), over 200 m. high with a mean span of about one-half the height, but tending to expand towards the top section.

The tests on the model of the Santa Giustina Dam were all carried out before the year 1941 in the Polytechnical School of Milan; their purpose was to compare the results with those of the traditional calculation for elastic arches, as well as—for the lower section—to compare them with those of the previously mentioned photo-elastic tests.

The 1:66 scale model was stressed under the action of pure water load. The weight effect was not brought into play: actually, for well-known reasons, it contributes directly to the stresses in the cantilevers and indirectly also to those in the arches.

The cast was made in plaster of Paris mix; this caused slightly different moduli, higher in the top section and lower in the bottom (thicker) section of the model. This was taken into account in passing from the model results to those inherent to the prototype.

Among the other findings, the values of the main tensile stresses, based on extensometer measurements, and the trend of the isostatic lines were deduced here too. It was noticed that, as a rule, such values differred only a little from those pertaining to the arch-cantilever lattice and this because the directions of the main tensile stresses nearly coincide with those of the arch-cantilever lattice, with the exception made for the section close to the bottom.

The tests on the Vajont Dam models have been so far developed in three subsequent stages or versions, corresponding to three different models that we shall list them in the order "A," "B" and "C" (Fig. 9). A fourth model, the "D" type corresponding to the new solution studies for the dam, which will attain to 260 m. of height, shall be tested next in the I.S.M.E.S.' tower.

The first two models, tested around 1940, had been made in order to investigate the static behaviour of the lower section of the dam, up to about 100 m. in height, inasmuch as the design was considering comparatively thick arches, which, given the limited span, stood out as extremely massive. In the "A" version it was attempted to keep the abutments as much as possible adhering to the gorge walls; the structure was therefore highly dissymmetrical, with comparatively very thick arches and reduced central angles. In the "B" version it was instead attempted to reduce as much as possible the dissymetry of the structure—at the expense of greater excavation—as well as to reduce the thickness of the arches increasing considerably the central angles and the slenderness compared to the "A" version, with an overall volume reduction of about 30 per cent. As far as the profile of the arches was concerned, the greatest advantage was taken of the photo-elastic tests previously mentioned. The results obtained from the tests carried out on the models, cast in a plaster of Paris-celite mix, emphasized the fact that the static behaviour of the "B" type was on the whole somewhat more favourable than the "A" type (Fig. 10). The dissymetry of the dam considerably affects the trend of the stresses; for these reasons, the "A" version shows much more pronounced inequalities between the stresses at the abutments, and as a consequence, in ultimate analysis, the maximum stresses were found to be lowest in the "B" version, which was more symmetrical than the "A" version. The water load is fundamentally carried by the arches, whereas the stresses along the cantilevers are here comparatively moderate.

The first two models were also tested for the weight effect, due of the upper part—not modelled—of the dam, and these tests showed that such effect was discharged sideways, as though in the framework of the structure some vertical arches had been created to support it; as a consequence in ultimate analysis, in its lower part, at the bottom, the vertical compressive stresses due to the weight was found to be only a small fraction of the very high stresses expected.

The static behaviour of the models was undoubtedly affected by the fact that the upper part was missing; it was therefore considered advisable to design and subsequently test another model of the entire dam. In the designing of such a structure, those variations were taken into consideration which the analysis of the preliminary tests suggested. This model, the "C" version, could only be constructed after the end of the war (1945-47) and was tested merely for the action of the water load (Fig. 11) with the principal purpose of checking whether it was desirable to further limit the thickness of the lower section; to give to the top section a second, limited curvature, and to collect all elements necessary for the final study of the structure, which it is foreseen will reach a height of 250 m. or 840 feet ("D" version).

In order to determine the effects of the local settlements of the material, particularly at the large abutment surfaces, it appeared appropriate to make two complete series of tests and of measurements, the first—known as the "normal load series"—with measurements taken over a load range limited to

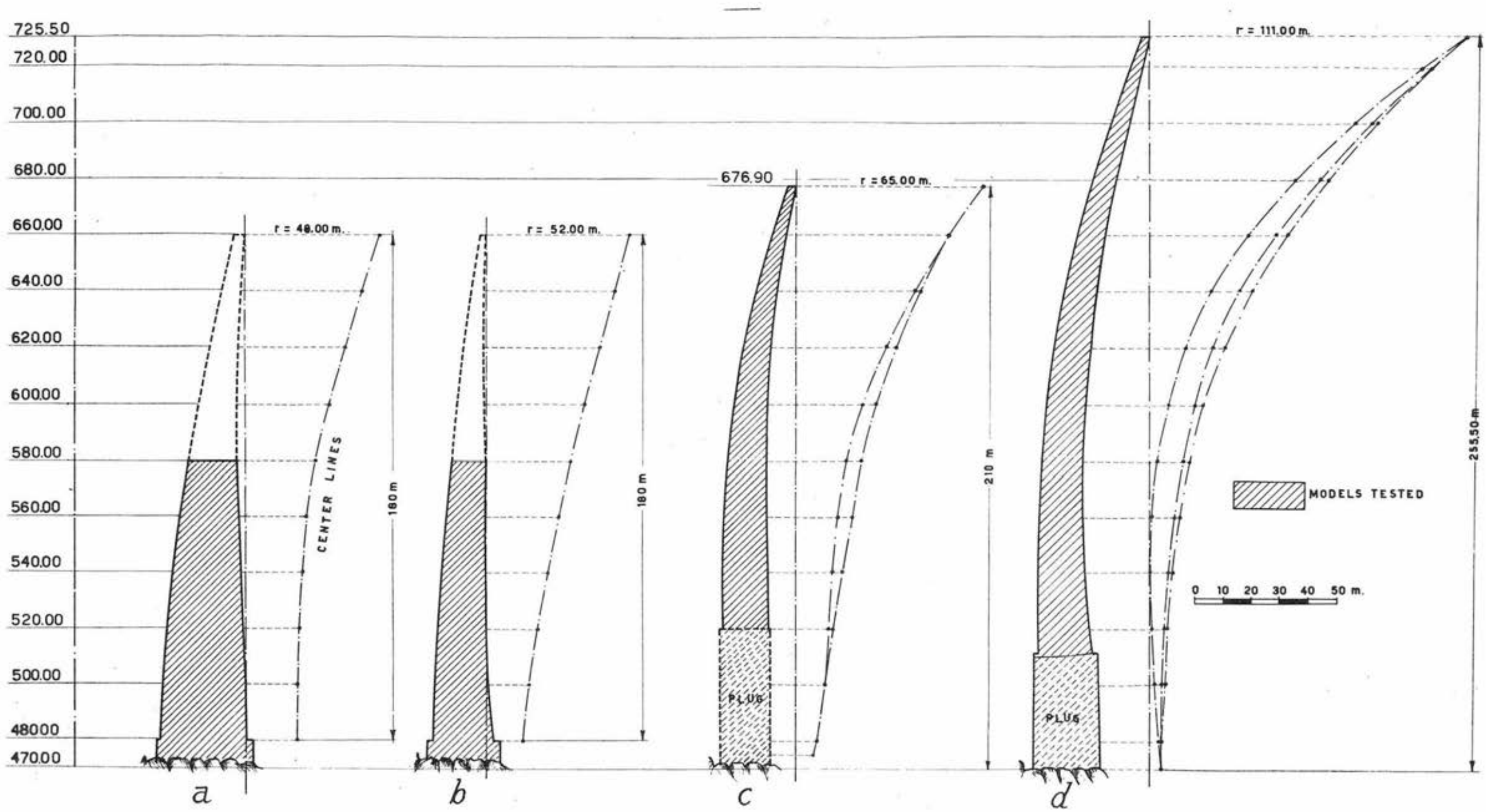

Fig. 9. Vajont Arch Dam: the Cross Section of the Three Models Tested During Preliminary Studies, and that (D) of the New Type to be Tested.

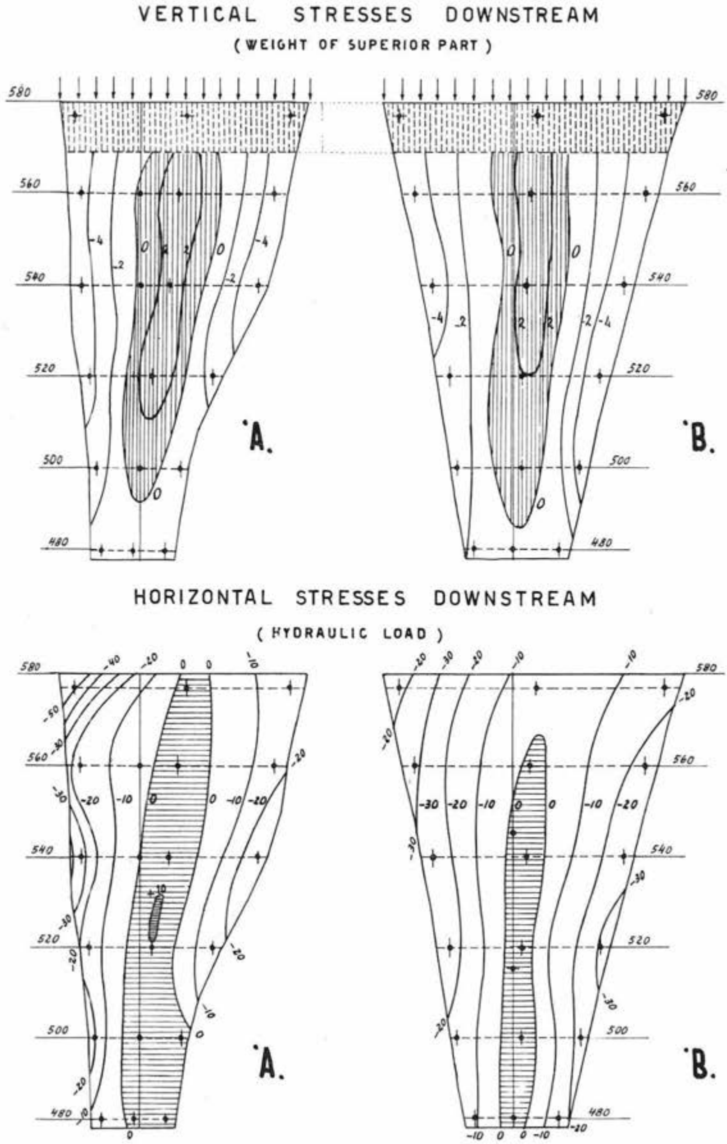

Fig. 10. Results on Comparison Tests Between Types "A" and "B" for the Preliminary Study of the Lower Part of the Vajont Arch Dam.

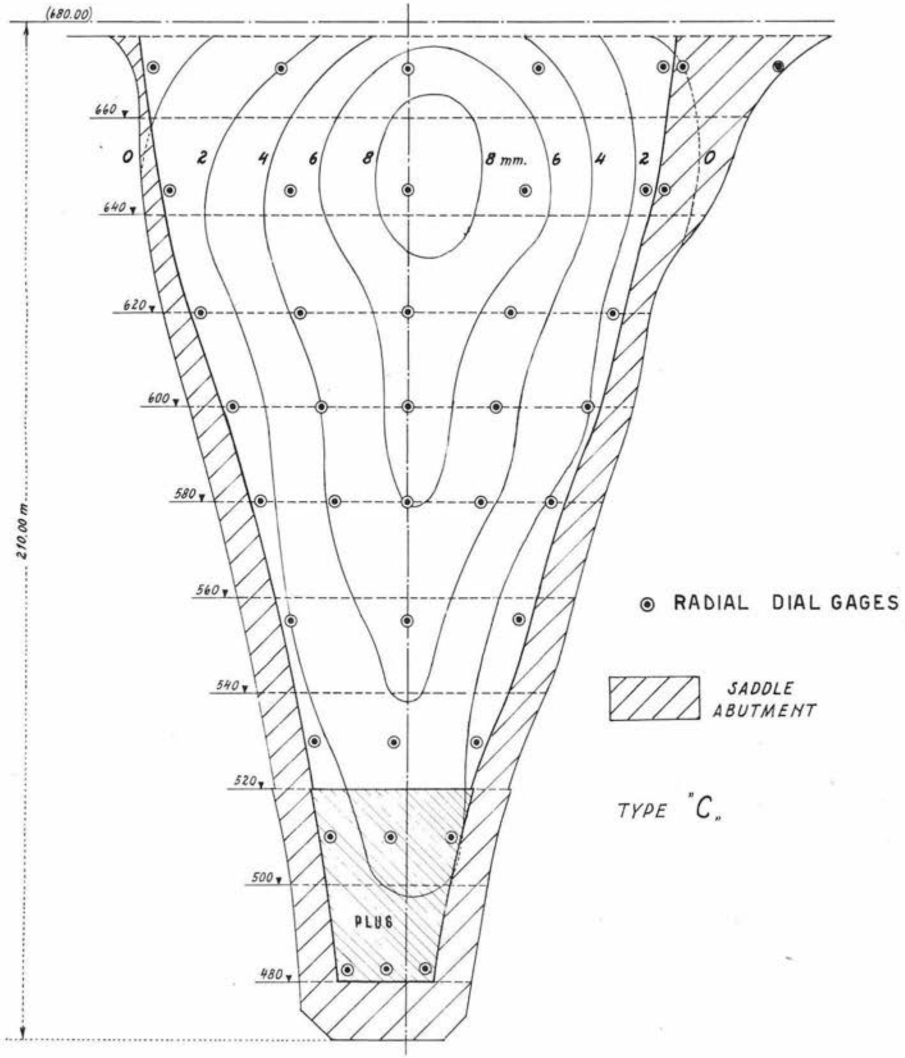

Fig. 11. Curves of Constant Deflection on the Downstream Face of Model "C" of the Vajont Arch Dam.

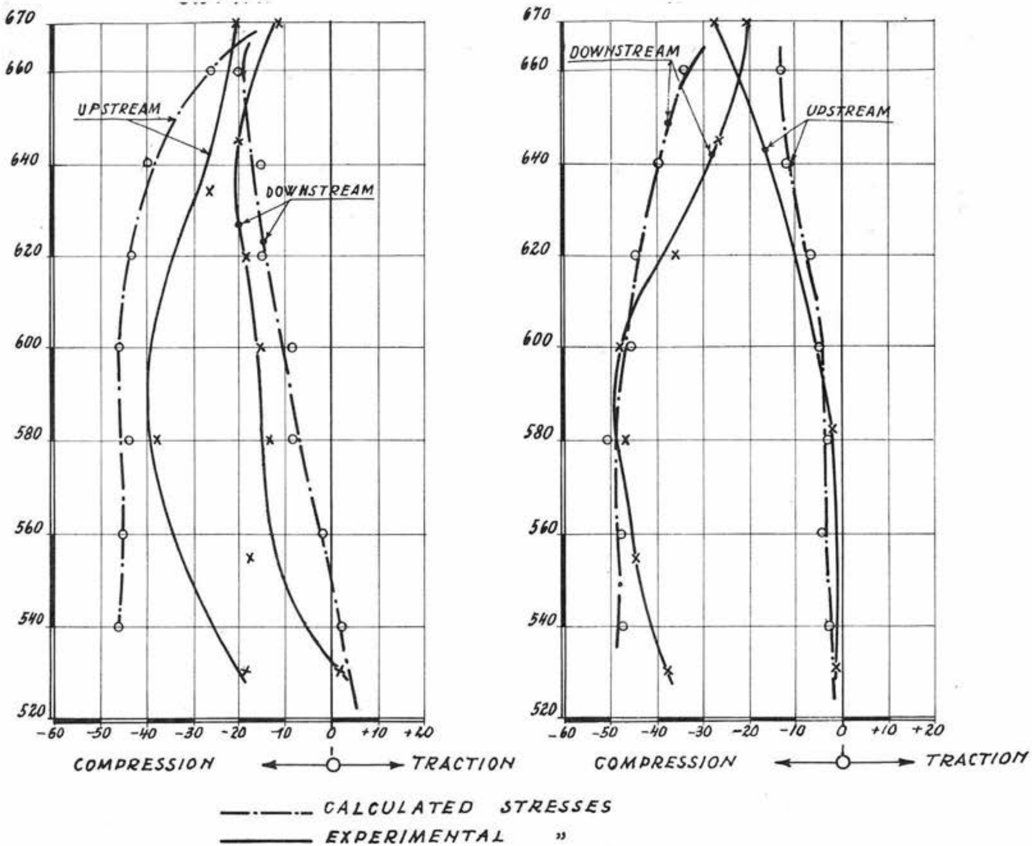

Fig. 12. Comparison Between Calculated and Experimental Stresses in the Arches of the Vajont Dam, Type "C."

a normal water load intensity, p_o; the second, with measurements taken over a load range included between the previous maximum intensity and one twice as big, $2p_o$.

From the results obtained on the model, those corresponding to the prototype (at full reservoir load) were derived in the two cases (Fig. 11); the non-coincidence of the values at corresponding points gave the measure of the settlement effects. It is interesting to note how the stress peaks are reduced in passing from the first to the second series of tests.

These various tests carried out on models of high and narrow dams have brought to light the fact that their static behaviour is distinctly different from that of the structures previously considered: the arch effect is here distinctly prevailing over the cantilever effect, already at normal loads, and, if symmetry is sufficiently conserved, the trend of the isostatic lines follows the arch-cantilever lattice, as it was possible to ascertain experimentally. The agreement with the elastic calculation (Fig. 12) is good enough. Only at the bottom the cantilever effect prevails over the arch effect.

Special mention should be here made to the abutment ties. Clamped setting was obviously done away with, replacing it with a mere leaning of the dam on the rock: about the profile of this bearing, particularly for the deeper arches, an abutment following a trend on a radial surface would have required a considerable amount of excavation and hence large amounts of concrete. Stepped abutments were then designed, which were adopted in the first models without giving rise to difficulties during the tests. In the last model of the Vajont scheme, however, as in the model of the Lumiej Dam, which will be dealt with further on, it was preferred to do away with the steps, adopting a continuous support saddle ("pulvino") and leaning the dam itself up against it along an accessible perimetral joint (protected upstream by a watertight strip, and an asphaltic sheet with filler block); this in order to eliminate the concentration of tension stresses, the steps should have produced and to facilitate the shrinkage and the thermal strains by reducing the consequent stresses.

c) The Lumiej Dam of S.A.D.E., finished in 1947, is an intermediate type in respect to the previous ones, the Lumiej dam having a height of 136 m. and a span at the crown of about the same length, with a V shape rather closed at the bottom.(6) The dam was built with a perimetral abutment joint and with the interposition of a support saddle cast between the dam proper and the rock shoulders containing it. The purpose of the support saddle, beside the practical one already mentioned was that of reducing the values of the normal pressures exerted against the rock, spreading them on a wider surface.

The tests carried out on models (4)—cast on a 1:60 scale, in a plaster celite mortar—had the fundamental purpose to compare the stresses occuring under the normal elastic conditions, with those obtained from calculation; and here the problem was particularly interesting since the designed structure had also a limited second curvature approaching the "dome-arch" type. Moreover, testing was pushed on, through the execution of ultimate load tests, up to the collapse of the model, checking its margins of safety, as had already been done after the Rocchetta Dam, for several other Italian dams.

As far as the dam abutment zone was concerned, the tests did actually indicate that the stresses on the up and downstream faces due to the water load were contained within tolerable limits, but they said nothing about either their local concentrations, close to the abutments, the trend of such enhancements within this zone and in the support saddle, or their increment when thermal action should add to the water load.

On the other hand the calculation, made according to the elastic theory assuming the arches to be clamped into the abutments, was leading—particularly when thermal action were considered in the most unfavourable period (winter)—to pressure curves which showed, near the abutments, eccentricity towards the intrados, such as to cause remarkable tensile stresses along the up stream face. These stresses although tolerable in themselves, were anything but logical, their presupposing a continuity which in effect did not exist. It was therefore to be presumed that in reality, thanks to the perimetral joint, a certain fraction of the abutment zone towards up-stream would remain practically inert—i.e., unstressed—whilst towards down-stream the compressive stresses should increase. Special model testing was organized, limited to those arches of the dam which (from calculation), appeared to be more stressed, such as to furnish quantitative elements useful in clarifying the static behaviour of the abutments zone and of the support saddle. Two successive series of tests were thus carried out on two-dimensional models: the first, in 1944, on celluloid models; the second, the following year, on large concrete 1:10 scale models.

As it was foreseen, the tests showed that the stress values, here really localised, were considerably affected by the condition of the bearing surfaces. It appeared therefore advisable to extend the tests and obtain results both in the case of surface cast directly one against the other, and in the case of a surface brought into contact with the interposition of a thin coat of cement mortar.

It was ascertained that the value of the local compressive stresses at the corners (edges) of the down stream face increased, reaching at the arch abutment as much as 80 Kgs. per sq. cm. (arch at 900 m. elevation) against the 60 predicted. It is not surprising to have found here higher values than in the previous tests and in the calculation, since only here strongly localized stress values had really been obtained.

Among the arch dam models recently tested at the I.S.M.E.S. Institute, the author wishes to further mention those of the Rio Freddo Dam (C.I.E.L.I.) and the Pontesei Dam (S.A.D.E.) the former quite interesting because of the abutting of the vault, which is not made directly on to the rock, but onto gravity buttresses; the latter, because it was designed as a rather thin dome type, with remarkable compressive stresses and gave very good results. This model moreover was utilized for an experimental study which, even though not concerning the prototype itself, supplied the solution of not too easy a problem (raised in connection with other Italian dams) concerning the possibility of damming a large U gorge formed by two high strength rocky side walls and a bottom foundation that did not give sufficient guarantees of resistance under vertical loads. A small cleft had been cut horizontally in the bottom section of the model, through from upstream to downstream face. It was then investigated what strength resources the dome structure possessed to discharge sideways also the load of its own weight, should the bearing effect fail at the bottom. This was achieved by measuring the stresses induced on the model by alternate cycles (loading and unloading) of the self-weight equipment.

In concluding this chapter, the author wishes to point out the high values of the "safety coefficient" (whose significance has already been discussed) from 8 to 12 shown by testing the models of this type of thin arched dam. The reason of this is to be sought in those peculiar strength resources which are a property of the three-dimensional vaults and dome dams, resources which, difficult to frame in a schematical calculation, are instead perfectly reproduced by the model.

Three Dimensional Models of Arch-Gravity Dams

Designs and construction of arch-gravity dams have been developed in Italy since some years ago, which are particularly suitable for the closure of a gorge having high span-height ratio, when the condition of the foundation rock allows their employ.

The first example of a complete design study for a large structure of such a type was the dam built by S.A.D.E.[6] across the Piave River, which, finished in 1949, stands out to this date as an outstanding example of its kind. The tests made on models of this dam turned out to be of the greatest importance, both for the evaluation of its safety margin and as an aid to the theoretical studies made concurrently with the tests, with a real team-work collaboration. (Various procedures were followed, including the "Trial Load Method" and the extension of the Tolke calculation.)

The Pieve di Cadore Dam has a maximum height, at the narrow gorge where the river flows, of 112 m., a crown span of about 330 m. and very large central angles. The section of the structure actually resisting as an arch-gravity type, of a mean height of 55 m., has an almost symmetrical development in respect to a middle vertical plane and is for the greatest part placed on the rock (limestone) and for the remaining part on a plug closing the deeper and narrower part of the gorge. The plug was designed almost exclusively with the aid of models testing which permitted the evaluation of its behaviour under water load.

Such tests were carried out following two procedures which supplemented one another. According to the first, tests were made using models of considerable dimensions (1:25 scale) of suitably compounded concrete, capable of reproducing both the elastic and the elasto-plastic static behaviour; according to the second procedure, the study of the elastic behaviour was improved using small photoelastic models. The first investigation, the closest to reality, reaches and evaluates the unelastic behaviour of the material and deduces the actual safety coefficient of the plug; but, unlike the second, only permits a limited number of measurements and is experimentally less exact. The second, instead, repeatable at will, lends itself to the comparative study of different profiles, successively corrected, with particular reference to the abutment zone.[10]

The most important tests were however those connected with the study of the whole dam in its curved plate continuity, bringing into play the non-computable and yet important factors, such as the asymmetry of the dam caused by the existence of the plug, the presence of the perimetral joint and the radial joints, and by the irregular outline of the foundation and abutment sections.

These tests were carried out at Bergamo in a big reinforced concrete double deck basin, on two large models, both in a 1:40 scale.[10] The first of these models was set up in 1947 and was useful, among other purpose, for indicating those variations of design which were later reproduced in the second model, so that this latter (having also been modified in the elastic behaviour of the foundation model according to the latest results of the geological investigation) did faithfully correspond to the final prototype; the results hereinafter briefly mentioned, refer only to this second model.

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Two principal tests cycles were made; one at normal regimen of the water load (corresponding to 140 atm. at the jacks), the other at a pressure increased by 50%. A comparison of the results clearly showed the good proportionality of the two series of tests. It may be noticed that due to the high deformability of the rock foundation the displacements at the foot are considerable, particularly in the central zone and that they particularly affect the trend of the total deformations.

The experimental results so far mentioned, obtained directly from the model measurements, offered a first fundamental criterion for judging the static efficiency of the dam. It seemed however convenient to work them out again so to obtain a deeper insight of the structure's strength to compare them with those established by the various methods of calculations, and draw some general conclusions, as far as possible, for structures such as dams of similar type; and since this structure—like other arch-gravity dams later designed and built in these last years in Italy, i.e., Pian Telessio dam—exhibits an ample central zone of nearly uniform characteristics, it appeared convenient to limit the theoretical investigation to this central zone and above to the perimetral joint, the remaining part of the dam being so complicated by local conditions as to frustrate any attempt to make a generalized investigation. For this latter part (in each single case) the model will be the only necessary and sufficient means of solving the problem; it clearly reveals the forming of inclined arches (plongeants) whose static function prevails over that of the vertical cantilevers and horizontal arches. As far as these lateral zones are concerned, therefore, even the usual calculation schemes are but illusory.

In the Piave dam the above-mentioned central zone extends approximately over half of the dam; here the stress trajectories appear as a network with a rectangular mesh lattice; at each elevation, the values of both vertical and horizontal stresses are distributed with fair uniformity on both faces.

In our case, for the evaluation of the water load distribution among the various strength capacities of the dam, equation (7) may be further simplified; in fact:

a) for the whole of the width of the vault here considered errors, we have practically $\tau = 0$; hence $M_{tx} = 0$, which means that for the central zone of the dam the torsional contribution is negligible;

b) it is also noted that $M_y(x) = \text{constant}$; from the expression of the bending strength of the cantilevers, the term $\frac{I}{rI} \; \frac{\delta My}{\delta x}$

c) the term $\frac{N_y}{r_I}$ deserves a special mention.

In the case of a thin shell, it represents the axial strength of the cantilevers, i.e., the contribution of the second curvature. The model results show that in our case N_y is very small; it is however uncertain whether its existence might be attributed either to a real second curvature effect or to an enhancement of the up-stream stress owing to a non-perfect linearity, or still to the existence of a system of subvertical arches within the dam thickness, or finally and more probably, to the contemporaneous presence of some of these factors. In any case, the action is small, probably of the same order of magnitude as inherent errors of the computation procedure. It is therefore practically admissible to neglect the second curvature effects.

From the computations done, the load appears supported almost exclusively by the axial strength of the arches and the bending strength of the cantilevers. The bending strength of the arches is very small everywhere, with a slight tendency to increase towards the foot of the dam.

Likewise very small, just perceptible at the foot of the dam, as was to be expected from the considerable width of the zone under consideration, was the residual contribution (necessary for the equilibrium equation 1 to be verified), due to the torsional stresses not directly deduced from model measurements.

A direct comparison between the results obtained from the tests and those from the calculations is also interesting. Among the latter, we found those of the Tölke method to have a particular significance for all the successive stages of the structure design, up to the final one, as tested on tri-dimensional models.

From the brief discussion which follows it will appear why it is justified to adopt such a method of calculation as fundamental for these types of very large dams—having large central angles and vertical radial section of nearly uniform characteristics for a considerable width.

A first comparison, always limited to the central zone, may be made between the load distributions between arches and cantilevers, and the consequent mean axial forces in the arches as well as the stresses at the up and down-stream face of the cantilever. The agreement is, on the whole, good and would further improve if a modulus ratio of 1/7 between rock of the foundation and concrete of the dam, had been assumed for the calculation, as the one obtained in the tests, which would only lead to a slight decrease of the contribution of the arches.

Now, the most notable difference between the Tölke method and the experimental one lies in the fact that the former does not take into consideration the effect of the side abutments and considers the dam as a sector of a shell. The good agreement among the stresses shows that, in the central zone at least, the arches having central angles around 130°, therefore much smaller than 360°, little affects the values of stresses.

The agreement is also good in the practical centering of the pressure curve in the arches. This may be explained considering that, whilst the abutment tie tends to create bending stresses, the considerable rigidity of the cantilevers tends to annul them quickly.

A second comparison may be made between radial (horizontal) deflections; in fact, they constitute an integral phenomenon, responding to the strains acting both in the central and the abutment zones, and are therefore apt to supply indications, even though of a general character, on the behaviour of the structure as a whole. Now, it was observed(10) that the test deflections are intermediate between those obtained from the Tölke calculation and those that would be obtained considering the arches as independent, clamped into the abutment and loaded with the Tölke portion of hydrostatic load p_a. The explanations of the phenomenon may be sought in the fact that the load is to be considered as increasing from the key towards the abutments (and it certainly is such at least close to the abutments), and in the limited extension, ascertained above, of the bending stresses due to the abutment tie. Both these causes, in fact, are concurrent in reducing the central line deflections.

In conclusion, the Tölke method seems to be fairly well suited to outline the behaviour of a dam having a high span-height ratio, uniform vertical sections and curvatures (for a sufficiently wide angular range).

If the Piave arch gravity dam shows a special interest besides the one it raises on account of its priority in order of time, various other models, among the many of the arch-gravity dams tested at the I.S.M.E.S., would also deserve a special mention on account of the peculiarity of their problems and in order to emphasize the substantial contribution brought by their testing on models to the solution of such problems. In the following pages we shall briefly discuss the testing of the three other models of Italian important arch-gravity dams.

b) The Beauregard Dam (S.I.P. Turin) has a maximum height of 135 m., a development at the crown of about 400 m. and the thickness increasing along the main section from 5 m. up to a maximum of 45 m. at the foot. The curvature radius is of 163 m. and the central angle 132° at the crown.

The main peculiarity of the problem of this large dam is the considerable dissymmetry of the elastic characteristics of the rock foundation, the ratio between the moduli of the two opposite abutment banks being 1:10; a further complication rises then from the presence of a huge pocket of "mylonitic" material at the foot of the yielding abutment (left side), of such proportions as to require the construction of a substantial concrete masonry subfoundation.

For the designing of this dam, the contribution of model testing was remarkable ever since the preliminary studying stage. The task of checking the validity of the theoretical calculation which were to be applied for the solution of the plain problem of an isolated thick arch with dissymmetrically yielding abutments, was entrusted to the ISMES. We have solved this problem by creating special testing equipment (fig. 7) with which it was possible to test on plane models several arches measuring the magnitude of the pressure curve displacements in respect to the central line; such displacements were considerably close to the stiffer abutment, tending to be very little towards the more yielding abutment and to be maximum, towards the extrados, in a section that no longer was the key one (as the case would be with a symmetry of abutment conditions), but was shifted in respect of it towards the more yielding abutment. Based on these preliminary results we also made the suggestion to draw the arches (horizontal sections) of the dam, not according to a symmetrical profile, but adopting instead a greater curvature of the axis on the side of the more rigid foundation.

Starting from these presuppositions, a first complete design was worked out and a 1:50 scale model was constructed according to it, faithfully reproducing the elastic characteristics of the foundation, including the reconstructed pocket. This model, cast in a special pumice mortar concrete with an "effectiveness ratio" ζ = 4 was tested in the ISMES big testing tower. From the results obtained, the possibility arose to proceed to a rational reduction of the structure volume, thus achieving the double purpose of increasing the structure efficiency and reducing at the same time considerably its cost.

The same model was consequently chiselled out in order to make the variation agreed upon, and was subsequently subjected to a new series of tests.

Finally, the "ultimate load tests" were executed with very good results. The first cracks (downstream) appeared after having increased the dead load twice (maximum of the experimental equipment) for a hydrostatic load 5 times the normal one. Successively, this load was yet augmented and for an intensity of 6.75 times the normal value the model failed through yielding of the more deformable abutment.

The Pian Telessio Dam (A.E.T. Turin) introduces into the field of the "span height" ratio values, usually pertaining to the arch-gravity dams, a peculiar value due to the exceptional development of the crown arch, which is equal to about 6.5 times the maximum height. This latter is in fact 80 m., while the crown has a mean development of 510 m. (1,700 feet), the maximum value till now reached for the arch dams.

A preliminary designing study having been executed, ISMES was requested to proceed with the investigation of a model also cast in pumice concrete on a 1:70 scale; the mountain was faithfully reproduced using deformable materials having about the same moduli as those of the dam, ($E_c/E_r = 1$), resulting from tests made "in situ." The model was subsequently tested for the full reservoir water load condition, after having been subjected to the equivalent load of its real dead weight applied at open joints.

The Cancano Dam (A.E.M. of Milan) has required for its designing the overcoming of substantial difficulties. The first of such difficulties is represented by the necessity of building the dam in two subsequent stages. The first stage construction reaches a height of 136 m. with a span of 330 m. and shall have to be placed under load for a period today not yet determined; the second stage construction shall have to be later cast on the first and will bring the height of the dam to a maximum of 176 m. (575 feet), with a crown span of 460 m. (1,600 feet).

The foundation rock (limestone) which shows a pronounced anisotropy ($E_{max}/E_{min} = 4$) due to the inclination of the strata outcrops on the sides of the valley, so that the flow of the stresses transmitted by the saddle of the dam to the rock is perpendicular to the bank in one side and parallel to it in the opposite one. The model, constructed on a 1:50 scale, was so far limited to the first stage construction and faithfully reproduces the anisotropy characteristics of the rock, this effect being obtained by the interposition of rubber sheets conveniently perforated so as to permit their expansion in a lateral direction.

This experimental study on model disclosed how a rational dimensioning of the structure may lead to a substantial reduction of the dissymmetry effects mentioned above.

The "ultimate load tests" also gave very good results giving a factor of safety for the structure of about 6.5.

c) It may be interesting to note how the elastic results obtained in testing these and other arch-gravity dams have emphasized some systematic differences with the theoretical calculation. The author believes that it may be partly due to the fact that the theory follows these principal postulates:

1. Linearity between stresses and strains (Hookes' law);
2. Structure continuity (absence of the radial and perimetral joints);
3. Symmetry of the dam and mountain foundations in respect to a main vertical section (generally);
4. Settlements of the foundation obtained by extending the (Vogt, Boussinesq) theory on the deflections of a semi-space due to a uniform load on a rectangular abutment area;
5. Law of conservation of the linear diagram in the stress distribution through the thickness (both for arches and cantilevers).

Based on the above postulates, we have compared the values of the stresses obtained from calculation (with $E_c/E_r = 2$) for the central zone of the Cancano Dam with the corresponding stresses deduced from the model, which are affected by the ascertained space dissymmetry of the structure. The same comparison was made for the other two dams already mentioned.

The following findings were clearly arrived at with the tests:

1. Tensile stresses on the downstream face in a vertical direction (cantilevers) somewhat greater than those indicated by the calculation;
2. Compressive stresses reduced on the downstream face and increased on the upstream one (therefore beneficial to stability) in the middle and bottom part of the cantilevers;
3. Higher stresses in a horizontal direction; i.e., higher arch contribution to the overall strength.

To better support these findings, which we deem to be of a general character (for arch gravity dams of the same type), we have thought it interesting to further elaborate the results obtained.

By assimilating the dam to a shell, as we have already said, from the strain measurements read directly on the model, by calculation we have traced back to the evaluation of the (water) load distribution among the main strength elements of the space structure, applying the relation (7).

The arguments the writer has already had occasion to assert, in connection with the study of the static behaviour of arch-gravity dams, appear evident. It also appears quite evident that to the fundamental strength elements, i.e., arches and cantilevers, a not negligible additional effect is to be added in the lower part of the structure due to complementary strength participation (torsional strength) which, in the ultimate analysis, do mitigate the load share supported by the traditional arch-cantilever lattice.

SUMMARY AND CONCLUSIONS

In conclusion it will be seen that the experimental researches on models have given, in Italy, a large contribution for the rational design of structures and, particularly, of the arch-dams.

After having summarized the actual possibilities of structural models, obtained from the theory of similitude and from laboratory researches, with particular reference to the researches of the ISMES works Institute the paper gives a description of the most important cases of Italian arch dam studied, till now, by means of models and reports some of the principal results obtained.

The experience and knowledge of the authors of the project for a dam permit, during the preliminary studies (in according with the topographic and geologic conditions) to establish a first economical balance among the different kinds of dams which may be considered as convenient to construct, and analytical calculations may be carried out for providing a first idea of the elastic stresses to be present in the dam.

But the possibility of rational economies in the design of the dam, of saving concrete, the confirmation of the principal stresses foreseen and determinations of the true factor of safety are most reliably accomplished by the aid of model-tests which, in several cases of Italian dams, have finally decided between the different types of dam-projects initially proposed.

Moreover when the arch dam exhibits dissymmetry, heterogeneous foundations, irregular abutment-lines and other irregularities; such as joints, spillway opening, interior tunnels and so on, only the model-tests can give good results. Such results have been recently confirmed by their good agreement with the measurements made on constructed dams.

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