

REPAIR WORKS AT FLUMENDOSA ARCH DAM (*)**R. SILVANO***Project Management**Ente Autonomo del Flumendosa***F. FRONGIA***Project Management**Ente Autonomo del Flumendosa***A. MONDADA***Head of structural analyses department**Lombardi Engineering Ltd.***A. PIAZZA***Design Engineer**Lombardi Engineering Ltd.***ITALY****1. INTRODUCTION**

The 115 m high double curvature arch dam of Nuraghe Arrubiu on the Flumendosa river, owned by the "Ente Autonomo del Flumendosa" of Cagliari in Sardinia, was completed in 1957 after a 5 years construction period. The dam is part of an extensive drinking water and irrigation system of the the city of Cagliari and shows an active storage capacity of nearly $300 \cdot 10^6 \text{ m}^3$. Already at final construction stage and immediately after its completion, a great number of cracks appeared on the upper part of the upstream face. As consequence, for nearly 40 years the reservoir was impounded only partially with a maximum water level 31 m below the design level and a $\frac{2}{3}$ loss of its storage capacity.

(*) *Travaux de confortement du barrage voûte de Flumendosa*

In order to meet the constantly growing water demand, the owner started a few years ago an important rehabilitation project with the aim to lift these operational restrictions.

This paper deals with the analyses carried out to find the reasons which had led to said extensive cracking. Furthermore, the rehabilitation concept of the dam is presented as well as some details of the works giving emphasis to the grouting procedure.

The remedial works started in summer 1995 with test groutings and are due to completion one year later.

2. MAIN CHARACTERISTICS OF THE DAM

The double curvature Flumendosa arch dam is founded on sound porphyritic gneissic rock of quite satisfactory geomechanical properties. The general layout of the dam with its ancillary works is shown in **Figure 1**. The main dam characteristics may be summarized as follows:

- Maximum dam height : 115 m
- Total crest length : 300 m
- Thickness at dam crest : from 3.77 m to 6.90 m
- Thickness at dam base : 29 m
- Crest elevation : 270.00 m a.s.l.
- Operational water level (O.W.L.) : 267.00 m a.s.l.
(original design)
- Live storage at O.W.L. : $299 \cdot 10^6 \text{ m}^3$

Referred to the characteristics of the dam geometry, the following general aspects may be pointed out:

- The ratio between the crest length and the maximum dam height is 2.6 which may be considered as relatively favourable for an arch dam.
- With a developed main surface of the arch body of approximately $S = 18'950 \text{ m}^2$ and a concrete volume of $V = 310'000 \text{ m}^3$, the slenderness factor C defined by $C = S^2/(V \cdot H)$, where "H" is the maximum dam height, turns out to be 10.1. Compared to other arch

dams under similar conditions, this slenderness factor may be considered as rather conservative.

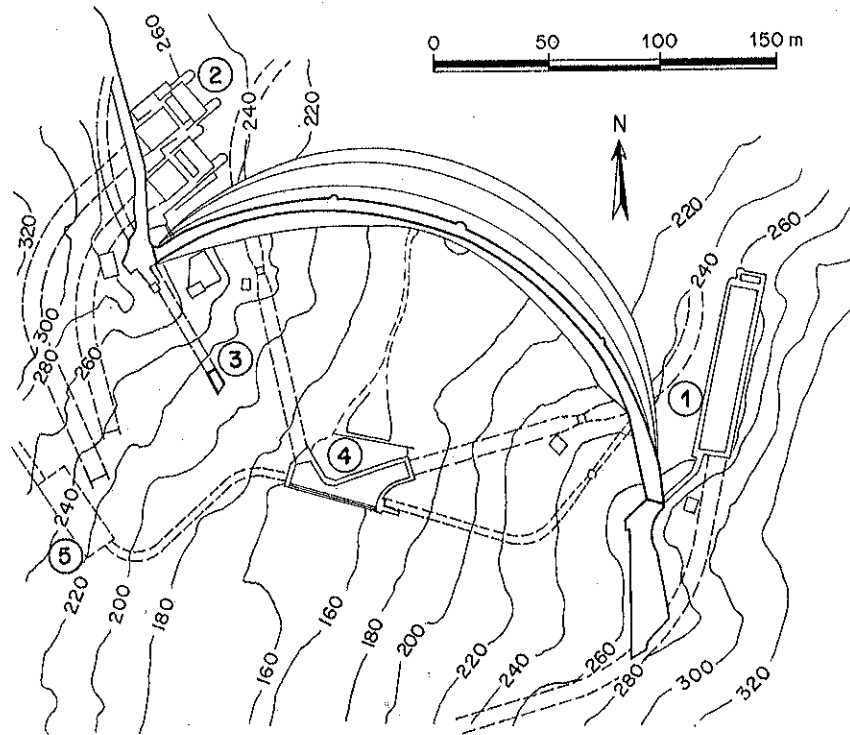


Fig. 1: General layout of the Flumendosa arch dam and its ancillary works.
Plan général du barrage voûte de Flumendosa et de ses ouvrages annexes.

Legend:

- ① Uncontrolled spillway
- ② Gated spillways
- ③ Mid-level outlet
- ④ Bottom outlets
- ⑤ Pumping station

Légende:

- ① *Evacuateur de crue à seuil fixe*
- ② *Evacuateurs de crue vannés*
- ③ *Vidange de demi-fond*
- ④ *Vidanges de fond*
- ⑤ *Station de pompage*

As regards the ancillary works, the Flumendosa dam is provided with the following outlet works (Fig.1):

- one uncontrolled spillway with a straight crest,
- two gated spillways equipped with double leaf vertical gates,
- one mid-level outlet, and
- two bottom outlets.

The dam geometry is defined by three-centred circular arches with a thickness increasing from the crown to the abutments. The nearly symmetrical dam was cast in 24 blocks each approximately 12.0 m wide. Furthermore, according to a design practice developed in Italy since 1940, the dam is provided with a perimetrical joint and a "pulvino" as shown in Figure 2. In addition to the three horizontal inspection adits in the dam body which are connected by inclined shafts, a drainage gallery follows the perimetrical joint.

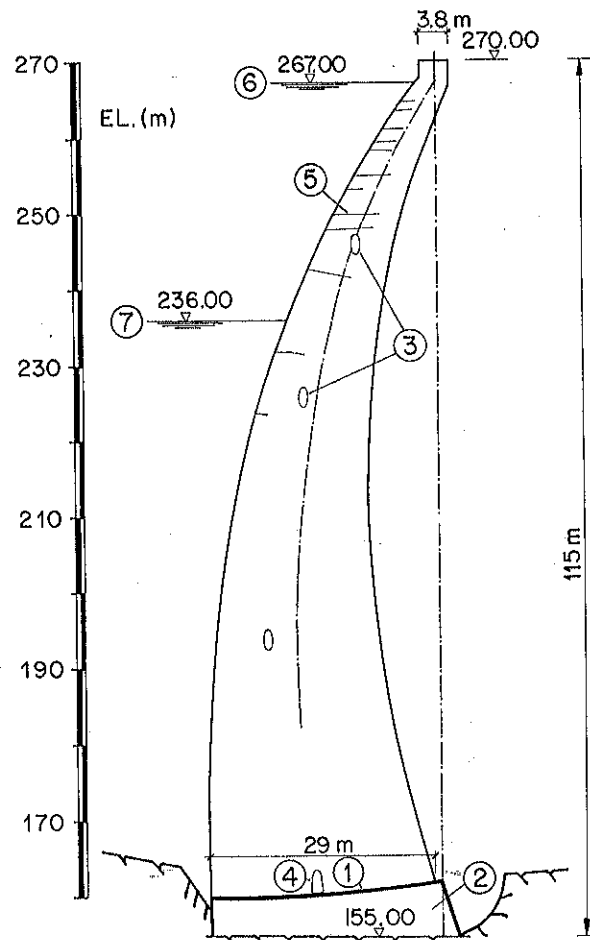


Fig. 2: Cross section at the crown.
Section en clef de voûte.

Legend:

- ① Perimetrical joint
- ② Pulvino
- ③ Inspection adits
- ④ Drainage gallery
- ⑤ Main cracks
- ⑥ O.W.L. according to design
- ⑦ Effective O.W.L. up to repair

Légende:

- ① Joint périmétral
- ② Pulvino
- ③ Galleries d'inspection
- ④ Galerie de drainage
- ⑤ Fissures principales
- ⑥ R.N. selon projet
- ⑦ R.N. effective jusqu'à réparation

This figure clearly shows the significant downstream overhang of the dam blocks. It should be also noticed that the multicentred arches are progressively changing to a circular shape at the crest zone.

Based on the classical structural analysis, the maximum principal compression stresses are below 6 N/mm^2 while the principal tensile stresses do not exceed 0.4 N/mm^2 . Compared to the average compression strength of 34 N/mm^2 measured on concrete samples at 90 days and a corresponding tensile strength of 2 N/mm^2 , the stress levels in the concrete may be considered as rather low for a dam of this type and dimensions. In addition, the perimetrical joint practically excludes tensile stresses in the dam body all along the foundation line.

Despite of the relatively low stresses resulting from the analysis, numerous cracks were recorded mainly in the lift joints of the upper zone of the upstream face as schematically shown in Fig. 2. During the first years of operation, the cracking further extended with a propagation of the existing and the developing of new cracks. The phenomenon slowed down progressively and since 1971 no new cracks were detected.

Core drillings indicate that the cracks depth is relatively limited near the dam crest as well as below 220 m a.s.l., whereas it may attain 60% of the dam thickness in between.

Due to the extensive cracking in the upper dam zone, the owner decided, in agreement with the State Supervising Authority, to maintain the reservoir level below 236.00 m a.s.l., instead of 267.00 m a.s.l. according to the design. The main reason invoked for this limiting operating condition was that uplift pressures in the cracks could negatively affect the structural behaviour of the upper dam zone.

In the past an attempt was therefore undertaken to seal the cracks at the upstream dam face using a two-components joint sealer. However, the difficulty to achieve this goal with sufficient reliability resulted, for a long period, in the abandon of any trial to repair the dam.

3. CAUSES OF THE CRACKING

Prior defining the remedial concept, the causes of the cracking had to be clearly identified. Only once the causes of the damage were known, an adequate rehabilitation project could be suggested.

The general aspect of the upstream dam face, as shown in **Figure 3**, attests the extensive opening of the lift joints. A total of nearly 176 one-block-long cracks were detected.

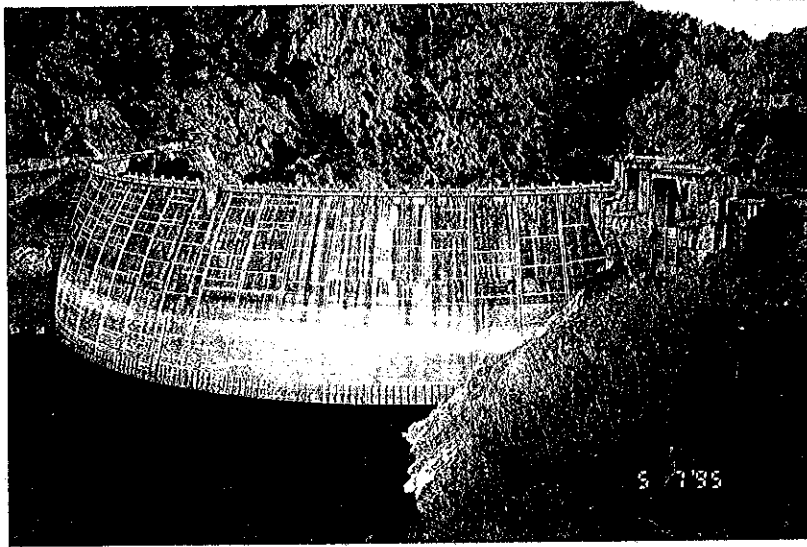


Fig. 3: View of the upstream face attesting the extend of open lift joints.
Vue du parement amont mettant en évidence l'étendue des joints de reprise ouverts.

Due to the very important downstream overhang, the upper zone of an independent cantilever (before grouting the contraction joints) is not stable under its own weight. Stability could therefore only be achieved by grouting the contraction joints in order to allow the arches to act as soon as possible. But, due to the fast progress of the concrete placing, the grouting had to take place before the hydration heat could dissipate completely. It has to be mentioned that no artificial cooling by embedded pipes was used, which resulted in a relatively slow cooling of the concrete. At the time the concrete pouring was completed, the temperature of the upper zone exceeded thus by far the yearly average temperature. Following, the cooling continued well after the construction was finished. Furthermore, the presence of a geological fault on one abutment delayed the concreting works of some lateral blocks. The effective concrete placement was thus significantly different from the planned schedule. Consequently, at the end of the construction, the temperature of the upper dam zone was generally higher at the abutments than at the crown.

Due to the reduced thickness of the crest, the cooling was there more rapid than at lower elevations where the dam is quite thicker. The decrease of the temperature resulted in a shortening of the arches causing a downstream deformation of the blocks. Consequently, the tensile stresses on the upstream face further increased leading to the described crack pattern.

Figure 4 shows the influence of the reaction forces between cantilevers and arches on the extend of the zone of vertical tensile stresses. The progressive reduction of the arch action due to the decrease of the concrete temperature is schematically shown by the forces "R1" to "R3".

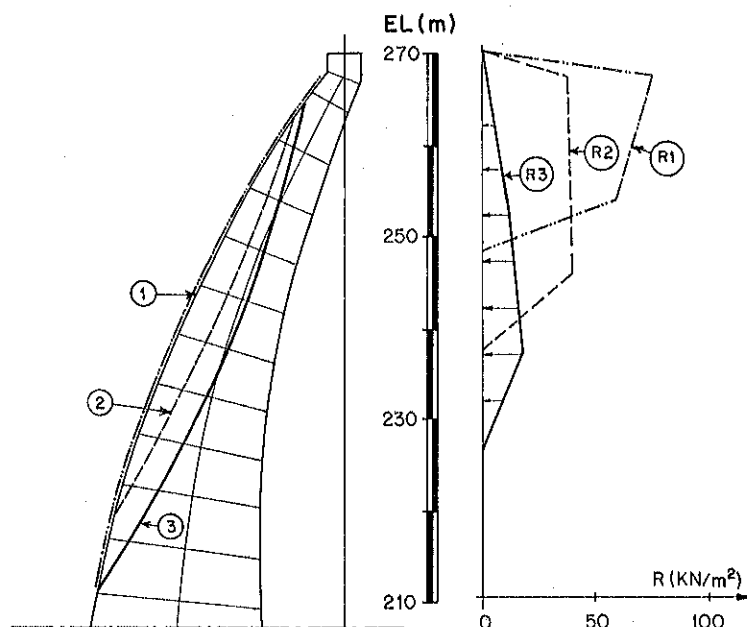


Fig. 4: Influence of arch reaction forces on the extension of the zone of tensile stresses in the crown cantilever.

Influence des forces de réaction des arcs sur l'étendue des zones de traction dans la console centrale.

Legend:

- ① Assumed limit of the tensile zone immediately after joints grouting
- ② Limit of the tensile zone at partial cooling stage
- ③ Limit of the tensile zone at complete cooling

- R1 Initial arch forces
- R2 Arch forces at an intermediate situation
- R3 Final arch reaction forces

Légende:

- ① Limite admise de la zone tendue immédiatement après le clavage des arcs
- ② Limite de la zone tendue pour un refroidissement partiel du béton
- ③ Limite de la zone tendue pour le refroidissement total du béton

- R1 Allure initiale de la réaction des arcs
- R2 Allure intermédiaire de la réaction des arcs
- R3 Allure finale de la réaction des arcs

Although the previous considerations may qualitatively explain the observed situation, a quantitative analysis was required as base for the rehabilitation project. A detailed thermal

analysis of the construction phases was thus carried out using a program system written by Lombardi Engineering Ltd.

Based on this simulation, it was possible to know the thermal status of the dam during and after the construction period. Some results are summarized in **Figure 5**.

As shown in this figure at the end of July 1957, the joints at elevation 254 m a.s.l. were grouted while the average temperature was approximately 26°C.

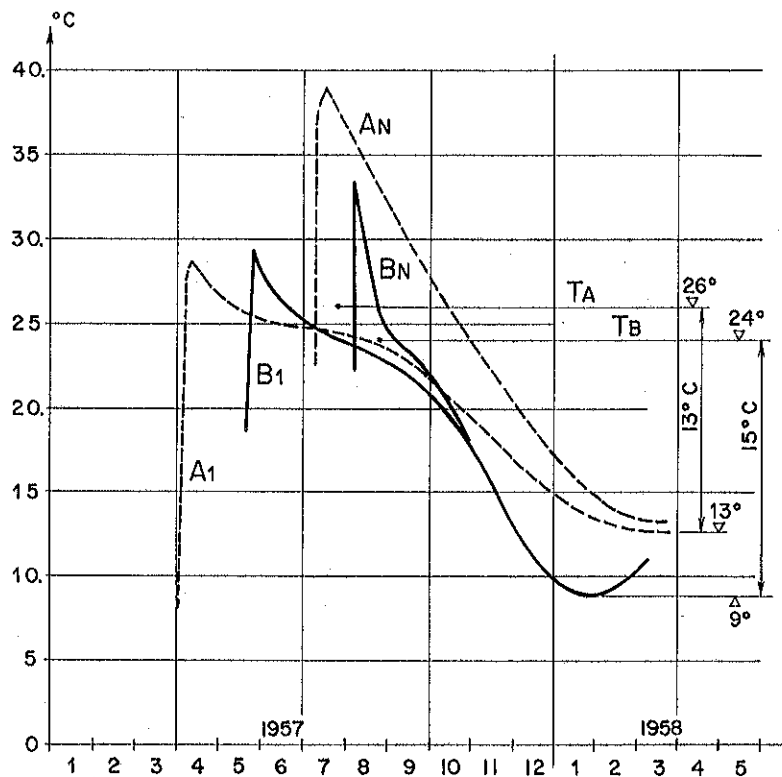


Fig. 5: Results of the thermal analysis for the arches at el. 254 and 270 m a.s.l.
Résultats de l'analyse thermique pour les arcs aux niveaux 254 et 270 m s.m.

Legend:

- A₁ Temperature of the 1st concreted block at el. 254 m a.s.l.
- A_N Temperature of the last concreted block at el. 254 m a.s.l.
- B₁ Temperature of the 1st concreted block at el. 270 m a.s.l. (crest)
- B_N Temperature of the last concreted block at el. 270 m a.s.l.
- T_A Average temperature of the dam blocks at el. 254 m a.s.l. at contraction joints grouting
- T_B Average temperature of the dam blocks at el. 270 m a.s.l. at joints grouting

Légende:

- A₁ Température du premier bloc coulé à la côte 254 m s.m.
- A_N Température du dernier bloc coulé à la côte 254 m s.m.
- B₁ Température du premier bloc coulé à la côte 270 m s.m. (couronnement)
- B_N Température du dernier bloc coulé à la côte 270 m s.m.
- T_A Température moyenne des blocs à la côte 254 m s.m. au moment du clavage
- T_B Température moyenne des blocs à la côte 270 m s.m. au moment du clavage

Similarly, the arch at 270 m a.s.l. was completed at mid August 1957 and its average temperature was then approximately 24°C. Approximately 6 months later, the average temperature had decreased significantly to a value of 13°C for the arch at 254 m a.s.l. and to 9°C at the dam crest. At lower elevations the temperature decreased in the same period by only 3-5°C.

The cooling that took place after the grouting of the joints resulted in a downstream deformation of the crest of 34 mm producing a bending of the cantilevers. This relevant deformation entirely explains the wide zone of tensile stresses in the dam body and the extensive cracking at the upstream face.

4. CONCEPT OF THE REMEDIAL WORKS

The rehabilitation project basically consists in grouting the existing cracks with epoxy resins. The aim of the remedial works was to restore a monolithic structure and to avoid the formation of interstitial pressures in the dam, in sealing the lift joints.

The stress distribution after the completion of the remedial works depends obviously on the actual conditions of the dam during the grouting, namely the impounding level and the concrete temperature.

After the cracks were grouted, the dam may be considered as a monolithic body, although the stresses acting in the structure before and after grouting cannot be linearly superposed, but have to be considered separately.

As an example, **Figure 6** shows the stress distribution in the crown cantilever after the completion of the grouting at summer time by empty as well as by full reservoir.

The structural analyses of the rehabilitated dam were therefore carried out by "freezing" the stress distribution existing before the grouting and superposing it to the stresses generated by the forces acting later on the monolithic structure. This results in a non linear stress distribution across the dam.

The grouting of the cracks must be carried out during the winter at reduced water level (January - March). In fact the low concrete temperature at this time leads to a downstream deformation of the dam resulting in a maximum opening of the cracks on the upstream face.

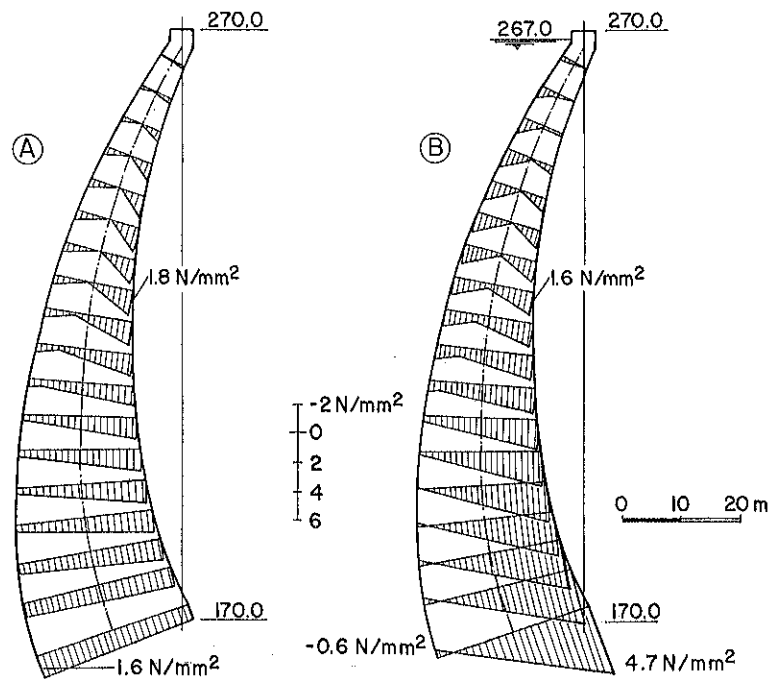


Fig. 6: Stress distribution in the crown cantilever after grouting of the cracks.
 (A) empty reservoir, during summer (B) reservoir at O.W.L. during summer
Distribution des contraintes dans la console centrale après injection des fissures
 (A) retenue vide, pendant l'été (B) retenue normale, pendant l'été

The main results achieved for the dam restored by the remedial measures may be summarized as follows.

- After completion of the remedial measures, carried out under the previous conditions, the maximum principal compression stress still remains below 6 N/mm^2 .
- The area of the former cracks will be free of tensile stresses no matter under which conditions.
- Furthermore, the vertical compression stresses on the upstream face will everywhere exceed the hydrostatic pressure that may exist at that point at any time.

5. REMEDIAL WORKS

In order to define in more detail the remedial measures, test groutings were carried out during the summer 1995. The main purpose of the tests was the selection of the type of epoxy resin to be used as well as the identification of the most suitable grouting procedure. Considering that

the opening of the cracks attains usually 0.5–1.0 mm at the face and diminishes towards downstream, the accurate selection of the resin is of primary importance in order to achieve a satisfactory grouting while avoiding excessively high pressures which could be harmful.

Although the test groutings had to be carried out during the summer (under different conditions as the remedial itself) the results were quite useful.

Both the test groutings as well as the main remedial works were carried out in an upward direction starting with the cracks at lower elevation.

In a first step a 10'000 m² scaffolding was erected on the upstream face from elevation 220 m a.s.l. to the dam crest level.

Then the extension of each crack was investigated by core drillings almost up to the downstream face. A videocamera introduced into the drillholes was used to detect the cracks as well as the frequent anomalies like honeycombs. All the groutholes required for the grouting were drilled in the following stage. The general lay-out of the groutholes is shown in **Figure 7**.

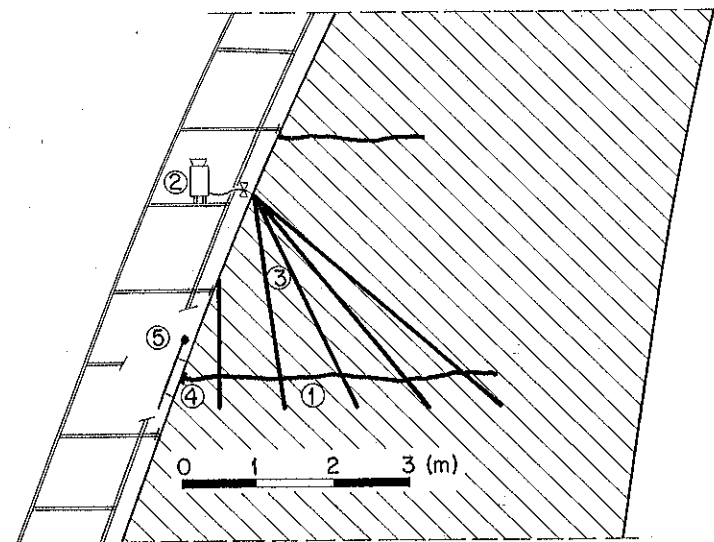


Fig. 7: Schematic lay-out of the boreholes and equipment.
Disposition schématique des forages et de l'équipement d'injection.

Legend:

- ① Crack to be grouted
- ② Grouting equipment
- ③ Groutholes
- ④ Sealing on the upstream face
- ⑤ Micrometer

Légende:

- ① Fissure à injecter
- ② Equipement d'injection
- ③ Forages pour l'injection
- ④ Scellement de la fissure sur le parement amont
- ⑤ Micromètre

After completion of the drilling works the cracks were accurately washed. The remaining water and the fines were sucked up in order to dry as much as possible the concrete. Indeed the adhesion between the resin and the concrete is the better, the cleaner and dryer are the surfaces. Prior to grouting, the fissure on the upstream dam face was sealed using a synthetic mortar in which plastic pipes for checking and aeration were inserted. The grouting was then carried out in a single stage starting with the most inner holes and continuing toward the upstream face as schematically shown in Fig. 7.

The propagation of the grout was controlled in observing both the adjacent grout holes as well as the mentioned check and aeration pipes. The injection was stopped as soon as clear resin was flowing out of the pipes or became visible on the downstream dam face.

The pressures varied between a maximum of 60 bars at lower elevations to a minimum of 10 bars near the crest. The grouting pressure was kept as high as possible in order to insure an adequate penetration of the resin but still low enough to avoid any damage to the structure. An excessive grout pressure would have caused an additional extension of the cracks. As regards the grout volume the presence of voids and honeycombs in the concrete made its evaluation quite difficult as most of the injected resin was necessary to fill the voids rather than to seal the cracks.

The remedial works carried out may be quantified as follows:

- Total surface interested by the grouting works : 13'000 m²
- Total number of cracks grouted : 176
- Total quantity of epoxy resin : 75'000 kg
- Total length drilled : 11'000 m

The entire rehabilitation required only one year. The grouting took three months.

In addition to the extensive laboratory tests on the resins, a complete site investigation program is planned to quantify the results of the rehabilitation. So concrete samples will be used for laboratory tests. Also permeability in the investigation boreholes will be measured.

Vertical test lines for geophysical investigations will be implemented on each block.

Figure 8 shows the improvement of the wave velocity and thus of the apparent over-all elasticity modulus achieved in grouting the cracks in the test block Nr. 17.

The results of the complete tests on the other blocks will be available in a next future.

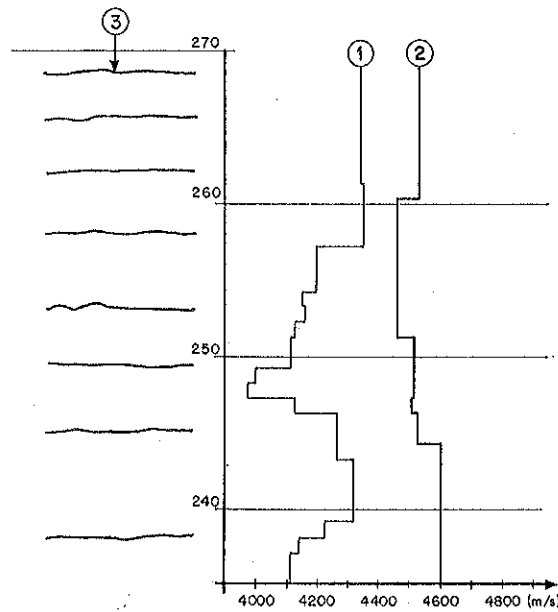


Fig. 8: Geophysical investigations in the test block Nr. 17.
Géophysique dans le bloc d'essai N° 17.

Legend:

- ① Seismic velocity before grouting
- ② Seismic velocity after grouting
- ③ Cracks and their elevations

Légende:

- ① Vitesse sismique avant injection
- ② Vitesse sismique après injection
- ③ Fissures et leurs niveaux

Obviously the dam was adequately reinstrumented and a monitoring program implemented to exactly follow the behaviour of the dam during its future life.

6. CONCLUSIONS

The double curvature arch dam of Flumendosa suffered extensive cracking on the upstream face due to excessively high temperature of the mass concrete at the time the contraction joint were grouted. The following cooling of the concrete resulted in relevant downstream deformations of the dam leading to extensive cracks on the upstream face. Since most of the cracks occurred at the lift joints, it is felt that the tensile strength of these contacts was particularly low.

The ambient conditions, in addition to an adequate selection of the epoxy resin and to the best grouting procedure, play an important role. The load conditions of the arch dam have to be accurately selected in order to carry out the grouting at the optimal crack opening. Finally, the present experience confirms that even extensively fissured dams can be rehabilitated in restoring a monolithic behaviour of the structure by grouting with epoxy resin.

SUMMARY

The extensive fissured arch dam of Flumendosa in Sardinia was rehabilitated.

An important grouting program with epoxy resins was carried out.

The origin of the fissures was found in its big overhanging to downstream. This shape required a grouting of the contraction joints before the complete cooling of the concrete. The cooling which took place in the afterwards caused the fissures.

The problem was entirely cleared in simulating the thermal and the structural state with an adequate computing program. Also the best grouting procedure could be selected. The loads and the thermal conditions prevailing at grouting time are determinant for the results of the operation.

The works could be carried out in just one year. The grouting itself took three months.

This rehabilitation will allow the full impounding of reservoir after 40 years of reduced operability.

RESUME

Le barrage voûte de Flumendosa en Sardaigne fortement fissuré a été remis en état à l'aide d'importantes injections de résines époxydes.

L'origine des fissures est due à un grand surplomb qui a requis l'injection des joints de contraction avant le refroidissement complet du béton. Le refroidissement qui a eu lieu par la suite a été la cause des fissures.

La simulation de l'état thermique et statique du barrage par ordinateur a permis d'éclaircir participant la question et de choisir la procédure d'injection la plus adéquate. Celle-ci a été vérifiée sur un plot d'essai. Les conditions thermiques et de charge au moment de l'injection sont déterminantes pour le succès de l'opération.

La durée des travaux a pu être limitée à une année, dont trois mois pour les injections.

Les travaux de réhabilitation vont permettre d'atteindre le niveau de service prévu au projet après 40 ans d'opération à régime réduit.