UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY

Translation No. 11

CONSTRUCTION OF BOU-HANIFIA DAM, ALGERIA

by

J. C. Ott

Translated by Mrs. Séverine Britt

Washington
1947

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(A translation)

Ott, J. C., Engineer E.I.L. - La construction du barrage de Bou-Hanifia (Algerie); Special edition from the Societe du Bulletin Technique de la Suisse Romande, 1946, taken from Bulletin Technique de la Suisse Romande, Feb. 5, and 19, 1944._/

_/ The original text is illustrated by 42 figs. which have not been reproduced in the translation.

(Translated from the French by Séverine H. Britt in collaboration with D. J. Varnes, U. S. Geological Survey)

I. Introduction

In 1927 a vast program for irrigation development in Algeria was inaugurated. This called for the construction of several large dams and the raising of older structures.

The realization of this program, conducted by the Irrigation Service of the General Government of Algeria, does honor to the French engineers who were in charge of the construction (17)._/

_/ The numbers between parentheses refer to bibliography at end of report.

The broad outlook with which big problems were solved, the boldness in the conception, the caution in the execution, the careful testing of new methods and the constant control of construction techniques based on those methods, the overcoming of difficulties inherent to the natural foundations, all of this is praiseworthy.

It will be attempted here to give the reasons which influenced the choice of types of structures, the description of the means of execution, and finally the experiments and observations made in the course of construction with particular emphasis on the example of Bou-Hanifia dam.

II. Generalities on the Algerian dams

In contrast to the Swiss dams, which are intended exclusively for the production of electric power, the Algerian dams were originally planned for irrigation or for supplying towns with drinking water.

They are control structures generally located far upstream from the place where the water is utilized. Thus the stream itself serves as a feeder to the diversion works downstream.

The scarcity of power provided by plants located in the large ports, where the price of imported fuel is the cheapest, gave rise to the idea of also equipping the irrigation dams for production of electric power to cope with the peak consumption. It is, in fact, possible to release a high discharge from the dams during a few hours a day. Distribution of this flow of water occurs downstream along the bed of the river and the average discharge intended for irrigation is hardly modified.

The Algerian oued (stream) system is also different from the Swiss river system. It is characterized by a very weak flow with some floods of an exceptional intensity. Moreover, great irregularity of flow is noted from one year to the other. For instance, the rate of flow of the El Hamman Oued, at its lowest, may be less than 1 m^3/sec. In exceptional flood period, the flow reaches more than 5,000 m^3/sec., which corresponds approximately to the Rhine at Bale during flood period. The ratio is 1 to 5,000. On the Oued Cheliff, the range between extremes is still greater.

The destruction of the Oued Fergoug dam, which failed during an exceptional flood in 1927, as well as the failure of Oued-el-Kebir dam in Tunisia, showed the necessity of planning a diversion system for a very large flow, although the cost sometimes is nearly as high as the cost of the dam itself.

Finally, most of the Algerian dam sites rest on much more unfavorable foundation ground than those of Switzerland. It is commonly impossible to find a solid rock foundation.

1.- Classification of Algerian dam types.

The distribution of the different types of dams is closely related to the foundation ground as shown on Table I.

Gravity dams - Except for older structures of small importance, the solid concrete gravity dams which exert a very strong pressure on the ground are scarcely represented. The only large modern work of this type is the Oued Fodda dam.

The less important gravity dam of Zardezas was built under difficult conditions because the marly soil on which it was built presented a constant threat of landslides.

The Hamiz dam is an old structure which has been raised.

Multiple-arch dams - The multiple-arch dams, which do not load the ground so much but are susceptible to settling inequalities, are represented by the Beni-Bahdel dam. This dam will not be discussed here as it has been previously described in this Review (5).

Ksob dam, located in southern Algeria is less important.

The dikes (rock-fill dams) - Dams of the rock-fill type, which adjust themselves to strong deformations of the bearing ground, are the

TABLE I - CHARACTERISTICS OF ALGERIAN DAMS

NAME	RIVER	DEPARTMENT	RESERVOIR CAPACITY millions m^3	TYPE OF CONSTRUCTION AND VOLUME in m^3	HEIGHT ABOVE FOUNDATIONS m	CHARACTERISTICS OF FOUNDATION GROUND			Maximum stress upon the foundation ground		Remarks	Cost of dam and appurtenant works in French francs millions	Cost of Watertight & boring works in French francs millions	Directors of works for the Highway Department	Contractors in charge of the works: 1) Civil engineering 2) Water-stops
						Geologic Designation	Minimum Crushing strength on cubes kg/cm^2	Modulus of strain kg/cm^2	Empty lake kg/cm^2	Full lake kg/cm^2					
Oued Fodda	Oued Fodda	Alger	225	Gravity dam Concrete	100 (about)	Jurassic: limestone						150		M. Martin, Chief Engineer	1) Campenon-Bernard
Hamiz (raised)	Oued Hamiz	Alger	14 (theoretic) 12 (after silting)	Gravity dam	38 (initial) 45 (raised)	—			under upstream edge 11.8	9.1	Raised 7 m. in 1933			M. Martin, Chief Engineer	
Zardezas	Oued Saf-Saf	Constantine	15	Gravity dam Concrete		Marl and limestone								M. Piraud, Chief Engineer M. Levy, Engineer	1) Ballot 2) Bachy
Oued Fergoug (raised)	—	Oran	1½	Old gravity dam - rubble masonry	—						Accidental failure in 1927. Reinforcement of the gap by steel rods from 285 to 125 tons strength (Coyne system)	0.5 (reinforcement)	0.5 (reinforcement)	M. Vergnieaud, Chief Engineer M. Drouhin, Engineer	1) Saintrapt & Brice 2) S.E.C. (Rodio process) (raising)
Cheurfas (reinforced)	Oued Mekkera	Oran	18	Old gravity dam - rubble masonry	30	Miocene limestone					Reinforcement by 37 steel rods of 1000 t. for the purpose of raising		24 (reinforcement)	M. Vergnieaud, Chief Engineer M. Drouhin, Engineer	S.E.C. (Rodio process)
Beni-Bahdel	Oued Tafna	Oran	56 (raising planned)	Multiple-arches of 20 m span	46 (raising under way)	Sequanian: sandstone shale marly shale	450 to 500 27 to 90 17	25000 to 27000 5000 to 7000 480	under buttresses 4.7 under downstream toe: 2.1	 7.5 4.9	Calculated settling of buttresses: 10 to 17 mm.			M. Vergnieaud, Chief Engineer M. Martin, Dr. of Irrigation Services M. Drouhin, Assistant Director of Irrigation Services	1) Campenon-Bernard 2) Procedes de Cimentation Francois
Kaob	Oued Kaob	Constantine	50	Multiple arch	45	Eocene: limestone								M. Piraud, Chief Engineer	1) T.H.E.G.
Bou-Hanifia	Oued El Hammam	Oran	71	Rock-fill dike 765,000	56	Eocene marl Oligocene: sand sandstone marl	45 to 130 (without cohesion) (very variable resistance)	7000	7.8		Average permeability of Oligocene layers before injections: ⊥ 13 x 10^{-4} cm/sec ∥ 65 x 10^{-4} cm/sec Maximum ground settling, empty lake: 11 cm Total leakage: less than 3 lit/sec.; full lake	325 (not completed)	60	M. Vergnieaud, Chief Engineer M. Martin, Director of Irrigation Services M. Drouhin, Assistant Director of Irrigation Services	1) L. Chagneaud 2) S.E.C. "Rodio Process"
Bakhadda	Oued Mina	Oran	33	Rock-fill dike 320,000	45	Oxfordian: sandstone & clay Sequanian: limestone Cartenien (Lower Miocene): sandstone			6.4		Grout-curtain leakage: less than 0.2 lit/sec	120	21	M. Vergnieaud, Chief Engineer M. Drouhin, Engineer	1) Grand Travaux de Marseille 2) S.E.C. "Rodio Process"
Ghrib	Oued Cheliff	Alger	280	Rock-fill dike 675,000	71	Marl and sandstone			10		Total leakage: less than 2 lit/sec; lake ½ full	290	50	M. Martin, Director of Irrigation Services M. Drouhin, Assistant Director of Irrigation Services	1) L. Chagneaud 2) "Procedes de Cimentation Francois"
Foum-el-Gueiss	Oued Gueiss	Constantine	2.5	Rock-fill dike 130,000	23	Miocene: sandstone & shale Senonian: limestone	135 to 140	10,000	3.2 (cut-off wall 10.6)		Total leakage: less than 6 lit/sec; lake 3/4 full	45	5	M. Piraud, Chief Engineer M. Levy, Engineer	1) Etablissements Fourre et Rhodes 2) S.E.C. "Rodio Process"

most numerous. They require only a little concrete, which is an important advantage in distant areas where transportation costs are high. Without being entirely alike, they have common peculiarities which give them, so to speak, a family likeness. A trend toward standardization is noted. This series was begun by the Bakhadda dam, followed by the Ghrib, which is the largest. With its storage of 280 million m^3, the Ghrib dam compares to the future Splügen reservoir planned on the Hinterrhein, and it could contain the Dixence reservoir six times.

Finally the Bou-Hanifia dam completes the series. In 1941, it was nearing completion; only the outlet of the tailrace remained to be built.

2.- Characteristics of rock-fill dams and comparison with the American dams.

The Algerian dams are characterized by the following features:

Body of the dam: rock-fill (large stones carefully placed).

Very steep slopes (for a dam).

Accessible upstream deck.

Drainage to control underflow.

The large Algerian dams give the visitor an impression of magnitude. Their dimensions, however, are less than those of American dams. As a comparison, the original text shows a few American dams of the same type: for example, San Gabriel in California, and Quabbin dam (9a).

Generally the American dams, with their gentle slopes (1:3 downstream) are hydraulic-fill dams. The fine materials in the center form an impervious core, or else the core is of specially selected material placed in rolled layers. The other part of the dam is of loose-dumped or compressed material. The materials which render the dam impervious are embodied within the mass and are consequently inaccessible, but are more economical than a waterproof apron on the upstream face.

From these comparisons, it is apparent that the Algerian dams have a definite character which differentiates them from similar structures.

III. Bou-Hanifia dam

1. Local conditions

The local conditions, topographic and geologic, are frankly unfavorable. They were described in detail in this Review (10). It may be said that without the exceptional technical means which were employed it would have involved too much risk to set such an important structure on such bad ground. Unfortunately, no other site permitting the creation of an adequate reservoir was available for the construction of the dam.

The valley selected for the dam is relatively wide. The eroded and gullied aspect of its sides shows how much the upper slopes are susceptible to weathering. The strata dip upstream. Marly and sandy beds appear hollowed out, and the more resistant sandstone formations jut out.

The geologic section at the site is as follows:

At the base is an Eocene marl, which is impervious but settles under load. It is overlain by a complex of lenticular sand, marl, and sandstone layers of Oligocene age. The permeability of these beds, which are also crossed by more or less important faults, is extremely variable and, in places, very high. These factors made it difficult to insure a watertight structure.

The permeability of the Oligocene layers is especially great and generally varies between 0.1×10^{-4} cm/sec. and 150×10^{-4} cm/sec. Some sand and gravel beds are even more permeable. Because of the stratification, lateral permeability is greater than the vertical (see table I). The Oligocene formations are also dangerously liable to erosion through the action of running water. Any discharge over the crest of the dam could rapidly provoke the failure of the structure under conditions similar to the unfortunate experience of Oued Fergoug dam referred to previously.

For these reasons, the spillway and the tailrace were given very generous dimensions. They were designed for a maximum flow of 6,000 m^3/sec. 1500 m^3/sec. of which will be able to pass through the two diversion galleries which cut a meander of the stream.

2. Selection of the dam type depending on the nature of the site, the foundation ground and materials available.

It is obvious that a solid masonry dam was rejected immediately. There could have been hesitation between a flexible dam of the dike type with gentle earth slopes, and a rock-fill dam with steep slopes. For a structure of equal height, the second solution requires two and a half times less material.

An earth dike, in order to be economical, requires a quarry nearby where material of suitable composition can be found. The placing of this material according to a special technique should be followed very carefully. If complete security is desired, it requires a long and precise process for which time was lacking. Moreover, any appreciable variation in the composition of the material in the quarry may lead to great difficulties.

The risk of erosion of an earth dike at the time of an accidental overflow would be even more serious than with a rock-fill dam. Furthermore, due to the great width of its base, an earth dike is suitable only in the case of a valley with relatively parallel walls.

Finally, it would have been regrettable not to take advantage of the experience acquired at the Ghrib dam, where placing a rock fill was successfully accomplished and led to excellent results.

In short, the rock-fill dam offered a greater security of execution. Opinions may differ, however, and from the view of the authors of the project, an earth dike could have competed with a rock-fill dam in the case of Bou-Hanifia.

3. Characteristics of the project.

For planning the Bou-Hanifia dam, the Highway Department called upon a certain number of geologists, chemists, geo-technician engineers, and civil engineers, each of whom brought some original ideas. Among others mentioned is Prof. Terzaghi who contributed the ideas of the filter bed and the articulated cut-off wall. Further reference to these structures will be made later.

a) Defenses against deformation:

From the earliest planning stages of the project, the engineers took great care to avoid the detrimental effects of deformations as much as possible. As it is impossible to eliminate them completely, at least as far as the terrain is concerned, the structures had to be designed so as to be able to adapt themselves to these deformations. That was the reason for providing articulated cut-off walls with inserted joints. These joints were designed to permit a certain amount of relative displacement without endangering the imperviousness of the wall.

Settlement of the foundation ground was calculated on the basis of laboratory tests.

The rock-fill also will suffer some deformation through its own weight in spite of the care which might be taken in placing the blocks. Consequently a flexible upstream cover, which can adapt itself to the deformations of its support was provided.

In anticipation of ground settlement and movements of the rock-fill itself, a close network of reference marks and borings was made. The reference borings have a twofold purpose: they permit one to observe the settlement of the deep beds as well as of the beds at the surface.

b) Protection against erosion of the foundations (12).

The second great precaution taken by the engineers was to eliminate the danger of erosion of the foundations. This danger comes mostly from the total lack of cohesion of some sandy formations which could be carried away by ground water percolating under the dam.

Therefore a filter bed was constructed under the greatest part of the dam in order to retain the sand and still permit water to pass through.

By reducing the velocity of ground-water circulation, the filter bed also helps to limit the extent of the seepage zones which are concentrated at the downstream toe of the dam and at the points where the filter bed extends beyond the foundations.

c) Protection against leakage.

Finally, due to the very high permeability of the bedrock, it was necessary to develop to a maximum the watertight structures which resist water infiltration.

The following watertight structures are included in the Bou-Hanifia project:

1) a cut-off wall embedded in the impervious floor.

2) grout curtains on the banks, starting either from the surface or from the tunnels.

The three leading ideas of the project are consequently: defenses against dangerous deformations of the foundation ground and the dam by adaptation of the structures to these deformations, protection against erosion of the foundations, and control of leakage.

IV. Component features of the dam proper

Detailed description of the construction and functioning of the above-mentioned structures of the dam.

1. The filter bed

The principle of the filter bed is extremely simple. It lies in the concentration of the water outlet in certain points constructed so as to prevent the washing out of materials. The filter is composed of five layers which increase in grain size toward the top. The "granulometry" of each layer in the filter bed must be such that the grains cannot go through the interstices of the overlying layer.

The determination of the first layer which is in direct contact with the ground is the most difficult problem, as the composition of the ground must be considered. A rule given by Terzaghi makes it possible to rough out the problem.

The first layer, which proved to be the most effective, is composed of dune sand with particles ranging from 0.1 to 0.5 mm in size. In order to localize the zones of seepage where water may well up, the surface of the filter is divided into several panels separated by low walls; each panel is drained separately by accessible pipes 1.10 m in diameter.

The construction of the filter, which might have seemed rather difficult at first, proved to be relatively easy even where slopes were as steep as 70 percent. Supporting the filter is accomplished by cutting the beds in steps then temporarily protecting the cut faces with a coat of cement-gunite projected on a jute canvas.

The functioning of the filter proved to be very satis ctory. The specifications required that there should be no lasting increase of suspended material in the seepage water. This requirement was fulfilled under all conditions. An incident, which will be referred to later, caused the filter to work in the opposite way, but even in these completely abnormal conditions, no piping of material was observed.

It is interesting to note how the seeps from the filter were distributed after water was first allowed to rise in the dam.

The successive progression of the seeps could be observed as the storage level of the reservoir rose. A first observation was made when the level was at elevation 273 in December 1940, and a second one in the summer of 1942. In this interval the zone of seepage did not develop much, and it may be expected that even when the reservoir is full it will not affect the whole surface of the filter bed.

Consequently, for some future projects it seems possible to replace a continuous horizontal filter by a network of drains also horizontal but discontinuous. Such a filter system would require a very detailed study of the distances between the drainage pipes but would probably be more economical.

Several types of filters to prevent erosion may be considered. They are all based on the following principle: to concentrate the inflow of water at points which may be organized conveniently.

This result may be obtained in different ways.

1) By a horizontal and continuous filter such as that of Bou-Hanifia, which does not produce any increase in head as the water runs out at the level of the foundations;

2) By a network of vertical drains with filtering walls which will discharge freely into a deep gallery and which have been conveniently spaced. This system is used at the Ghrib. It has the advantage of requiring very small percolating surfaces. On the other hand, it presents the disadvantage of increasing the head of water to a certain extent as the underground water levels are lowered and, consequently, leakage is slightly increased.

3) A vertical continuous filter applied directly behind a cutoff wall could also be considered, but this system does not seem practical.

2. The grout curtains

a) Purpose of grouting:

The purpose of grout curtains is (1) to reduce the leakage while also reducing the extent of the seepage area, (2) to diminish the velocity of percolating water, and thus (3) to prevent the foundation materials from washing away.

b) Arrangement of the grout curtains:

In plan view a main curtain and a safety curtain branch out at both ends of the dam. On the left bank, the question is to close a gap well defined by the "North fault" and the cutoff wall, whereas on the right bank the problem is that the curtains cannot rest entirely on impervious formations. A possibility of bypassing remains. The purpose of the curtains here is to increase the path of percolation.

In order to avoid useless drilling through ground above the reservoir level, the laying of the curtains was accomplished from tunnels (galleries) located in each bank at about the maximum water level. The cost of these untimbered tunnels is relatively low and is largely compensated by economical drilling. The tunnel on the left bank, located at elevation 256, cuts the middle of the curtain in order to reduce drilling costs which increase very rapidly with depth.

For laying the curtains, one should take advantage as much as possible of the natural conditions which help excavation, especially when the beds are extremely variable in composition, passing from sandstone to marl and from marl to sand.

The main left-bank curtain is oriented so as to enclose, with a minimum of surface, a carefully designed canal. The safety curtain runs approximately along a level line, which lies at an angle to the main curtain.

In the course of construction, it was of prime importance to have the direction of the curtains coincide with particular directions along which trenching is more easily accomplished. It is hard to recognize these directions which appear in marl only after long dessication. In this respect, the best oriented curtains are the left-bank main curtain and the right-bank secondary curtain. On the other hand, the right-bank main curtain was much less satisfactory, and it was later thought useful to complete the grouting by digging a shaft from which the most dangerous layers were treated through radial borings taking advantage of the presence of the south fault.

The functioning of the curtains was also studied by means of tests on reduced scale models.

c) Foundation grounds:

The foundation grounds are of varied permeability. A full preliminary study was made through laboratory tests on samples. Briefly, permeability was determined by taking into account the thickness of each bed sampled, so that a well-balanced average was obtained, where the weights of samples are proportional to the thicknesses of the beds. The permeability parallel to the bedding is about five times greater than in the direction perpendicular to the bedding.

First method:

One method planned to treat only the large fissures and joints with cement-grouting. Grouting with chemical products would have been resorted to only to complement and facilitate the cement-grouting proper.

Second method:

The improvement of grouting processes (14) brought the engineers to consider also the watertightness of the sand beds, and led to a reduction of leakage. Another important reason which favored a more elaborate treatment was that the safety of the dam could be increased by diminishing the velocity of ground water in the foundation materials, particularly in the easily eroded sandy layers.

The question of the number of grout curtains was strongly debated. Was it preferable to have only one grout curtain and to concentrate on getting the desired result in a limited space by producing an important but very localized loss of pressure, or was it better to plan for two successive grout curtains? The last solution was finally adopted. In this way, the losses of pressure are distributed on a larger area, the slope of the water table is less steep, and the eventual defects are less dangerous and counterbalance each other.

d) Boring.

All the injection borings for grouting were made by the rotary method and are 45 to 85 mm in diameter. The boreholes were spaced from 1.50 m to 3.50 m apart. In the sandy zones, mainly under the tunnel at elevation 256, the boreholes were cased to avoid caving.

The injections of cement were made in downward stages 3 to 4 m long. The absorption was slightly increased when shorter cuts were made. This process was advantageous near the surface because it prevents grout leakage, but it required that the boreholes be redrilled many times.

An appreciable improvement in this grouting method was realized by a system consisting of a tube with valves (patented system). This tube is set in the borehole. The series of valves on the tube are at different levels and work from inside outward so that cement can pass out but injection products cannot be forced back into the tube. Inside the tube, a stop valve is moved upward. The injection can then be made continuously so that there is an appreciable gain of time.

e) Choice of grouting process_/

_/ A more complete analysis of the advantages and disadvantages of the different methods of chemical injection will be found in the publications at end of report.

Suspensions.

It is possible to inject suspensions into coarse-textured formations. This method is exemplified by the classical process of cement grouting. In finer material, sand for example, the grains of cement in suspension are retained very near the point of injection, the water from the grout is absorbed by the soil and a plug forms immediately. It is then necessary to resort to chemical injections. This patented process was used on a large scale at Bou-Hanifia dam.

Cement, marl and special clays, such as bentonite and others, can be injected as suspensions. The advantage of cement is its stability, but it is an expensive material as compared to a natural marl.

In some cases marl may be suitable, but generally it needs to be specially treated. As a matter of fact, if injected without special preparation, it may be washed out.

Bentonite is slightly superior to marl, but it presents the same defect, although this becomes serious only in cases of formations with large cavities. Very fine textured formations, sands for example, form a skeletal structure which holds the injected products.

True solutions.

In order to penetrate extremely fine mediums, it is necessary to resort to true solutions, which set only after a certain time. These are the gels.

They have the appearance of gelatine, and their own resistance is relatively weak. They are not intended to strengthen the ground but simply to clog the voids, however, they must be resistant enough not to be washed away. Most of the grouting at Bou-Hanifia was done by means of a silicate gel prepared following the "Rodio process".

The composition is as follows:

1) a solution of sodium silicate

2) a coagulative solution: whitewash or cement.

When whitewash is put in contact with the sodium silicate solution, a precipitate forms which is eliminated by decantation. The supernatant liquid is clear and has a relatively low viscosity (about twice that of water) which allows penetration through the finest media. The gel so formed is made up partly of a skeleton of gelatinous silica, and partly of an ungelled intramicellar liquid. The stability of the gel increases as the ungelled silicate flocculates. The factors which may cause the flocculation are the salts contained in solution in the ground water.

Gel of marl.

At the beginning of the war, it was necessary to contrive a system of grouting less expensive than the chemical grouting. It was also desirable to reduce the consumption of sodium silicate.

The contractor in charge of the watertight structures then perfected a gel which has the advantages of both suspensions and solutions. Suspensions of natural marl can be washed out. He, therefore, attempted to improve such suspensions by adding a certain quantity of gel which gives them a good physical resistance against washing away. Conversely, the marl contained in the gel is chemically more stable than the gel itself.

The preparation of this gel of marl is rather complex, and is divided in two separate systems. In the first, the dry marl is brought in, crushed, then mixed with water in mixers with horizontal axes. The marl suspension passes through screens which retain the coarse particles resulting from imperfect crushing, before it goes into closed circuit digesters, where a peptizing product is added to produce complete dispersion of the marl grains. A pump forces the material into the decantation vats, where any remaining coarse material settles out. This preparation is very important in order to obtain an extremely fine suspension able to penetrate into the smallest voids. In the second system, water is added to the concentrated silicate. Finally, the flocculating silicate is added to the marl suspension which was mixed in the first system. The final product is a grayish fluid which sets into a gel. The time required for setting can be regulated according to the quantity of flocculating product added. It is grouted in the same manner as cement, but it is an incomparably finer suspension.

f) Checking of the groutings.

The effectiveness of grouting can be checked in different ways.

First, prior to the filling of the reservoir, the boreholes were tested for water immediately after grouting. In order to check the distribution of the grouting material in the ground, a gallery was excavated low enough in the left bank to intersect the grouted zones. This made it possible to determine the distribution of the grout material in the formations. This inspection established the importance of the direction of the grout curtains in relation to the joint system.

Check wells were also drilled in the plane of the main grout curtains and at the end of the safety grout curtains.

Samples of the grouted formations were taken and tested in order to check the permeability and to find out if the grouting material had washed away.

Following the drilling of the check well on the right bank it was deemed necessary to improve the treatment of the sandy areas because of the unfavorable orientation of the joint system.

A third method of checking before filling the reservoir was carried out by pumping in small check wells located in the plane of the grout curtains below the level of saturation.

The filling of the reservoir and the underground water levels which developed subsequently were traced very closely by a series of piezometric measurements. To this end, a network of boreholes was equipped to take the pressure at different levels on both sides of the grout curtains. The first grout curtain permitted a relatively slight loss of pressure, whereas the safety curtain has a more pronounced effect. In other profiles, the phenomenon is reversed.

The watertightness of the cutoff wall was also determined by numerous check wells which cut the joints and concrete underpinning obliquely.

Finally, a last chemical analysis of the leakage water was made to see that there was no washing away of injected material. The silica contents of the river water originally was 3 mgr/lit but only increased to 7 mgr/lit.

The way the water level is allowed to rise plays an important part. It is advisable not to increase the pressure too rapidly in order to permit the clays brought by the river to silt up the reservoir.

The leakage was very small when the reservoir was first filled. The total leakage for all the structures, the impervious cover of the dam as well as the grout curtains, was of the order of 120 lit/min with the lake at elevation 273. Half of this figure may be attributed to leakage at the grout curtains and the cutoff wall. Naturally, it is necessary to wait until the level of water reaches its maximum in order to draw definitive conclusions on the effectiveness of the watertight structures.

3. Rockfill

The main rock fill dam is made of large blocks brought from a quarry 5 km away. The larger blocks are laid by crane, the smaller are hand placed. A very tight construction in which the voids do not exceed 26 percent is thus obtained. The volume of the rock fill attains 700,000 m^3, about one third that of the large pyramid of Egypt.

The dam was built slowly. As it rose the lower courses were compressed by the weight of the upper ones and underwent some deformation. The ground also was subject to the deformations which affected the lower courses of the masonry.

The measured deformations (max. 11 cm) are slightly larger than the calculated deformations (max. 4 cm) in which the settlement of the Oligocene beds had been neglected. The settlement of marl occurs rather slowly and continues for some time after the application of the load._/

_/ For Eocene marl, the calculated settlement (40 mm) coincides very well with the measured settlement (38 mm).

The settlement of the rock fill on itself is important. If a wonderful engine could be invented which could build the dam in a few hours, the total settlement at the crest, represented by the curve of cumulated settling, would reach 1.56 m. In reality, the settlement of each course is less, because the lower layers have had more time to settle. The profile of the dam changed under the effect of the deformations which caused the porous coating applied on the upstream face to separate from the stone masonry. This difficulty was remedied by installing horizontal joints filled with a bituminous compound.

When the rock fill reached the elevation 280, the upstream foot of the dam under the effect of the load started to move horizontally upstream, causing a strong pressure against the cutoff wall. The effect of this pressure could also be observed in the cross drains, which started to open in the joints. A drain located in the thalweg showed the most pronounced displacement. In short, the dam had a tendency to expand at the base.

Downstream, the movement caused the breakdown of a culvert-type drain. To obviate this difficulty, this culvert was replaced by a closed drain overlaid by a solid mass of rock fill to form a toewall.

The horizontal movements of the foot of the upstream face kept on, however, and caused an excessive pressure on the cutoff wall which cracked at the base and at the level of the tunnels. To alleviate this condition a trench was cut between the cutoff wall and the face of the dam. The trench crossed a masonry course which had formed a too rigid connection between the dam and the wall.

The exact cause of these movements is difficult to determine. The filter bed was held responsible. As a matter of fact such movements did not occur at Ghrib dam, which has no filter bed. However, the objection can be made that the rock fills at the Ghrib had a more regular shape and were laid on plane faces. Irregular blocks such as those found at Hanifia, have probably induced a kind of arching which produced oblique pressures concentrated near the faces of the dam.

4. Cutoff wall

The cutoff wall extends downward to the Eocene marl, so that its total height (above and below the ground level) is 70 m. The wall is only 4 m thick.

Two inspection tunnels give access to the check boreholes of the joints and collect part of the seepage water from the deck and the filter bed. Grouting was carried on from the tunnels for the purpose of integrating the cutoff wall and the ground. Evidently, it would have been possible to extend the cutoff wall on the left bank, but the increasing depth made it preferable to resort to a grout curtain.

The cutoff wall was designed so as to avoid sharp bends where deforming stresses would be concentrated.

The horizontal reinforcement at the bottom of the cutoff wall was not intended to completely prevent cracking but rather to distribute it. It has been recognized that a large number of fissures are considerably less dangerous than a small number of wide cracks with equal total opening.

Cracking caused by the rock fill pressure has already been mentioned. A system of strengthening was sought in order to prevent its extension. The engineers decided to install metallic tie rods to create a pre-stress on the concrete of the cutoff wall. The pre-stress on concrete is relatively slight, about 4 kg/cm^2, and would give the wall more resistance against bending stresses. Moreover, plans were made to integrate the different fissured elements and to grout the fissures without causing a general heaving.

The metallic tie rods are set in large diameter boreholes made by shot-drills. This method of using tie rods was first worked out on installations at Cheurfas dam but was materially improved at Bou-Hanifia. The tie rods used at Cheurfas were composed of a series of parallel wires and were made at the site. At Bou-Hanifia, another method was preferred. The cables were made in the factory, and provided at the upper end with cast steel heads which are very small compared to the concrete heads, several cubic meters in volume, on the tie rods used at Cheurfas. The cast steel heads can be completely embedded in the cutoff wall. The most exacting step is fixing the lower part of the cables. A satisfactory result was obtained only after a long period of trial and error. The end of the first test cables had not been provided with a ferrule, and the expansion of the cables caused the concrete to burst. This difficulty was overcome by properly spacing the loose ends of the different strands over a length of 4 m and by attaching to each a spiral ferrule made of three foils of hard steel. The lower end of each cable is sealed into place by means of a cement grouting tube which goes down to the base of the cable.

To allow free action in the upper part of the tie rod, each strand is first coated with a bituminous mixture, in the present case with "flintkote", then covered with a cloth strip. Seventy-eight cables were installed on the whole base of the cutoff wall. Their average spacing is one to every 4 to 5 m^2.

The cutoff wall is articulated by numerous joints which permit certain relative displacements. The original project called for a rather complex system of joints each of which consisted of a ring-shaped annular space containing bitumen. Every joint was completed by a sheet-metal groove fitted to each side of the joint. When the bitumen was first poured in these joints, an unexpected difficulty rose. Due to the presence of seepage water, the hot bitumen did not adhere well and a perfect seal was impossible. It was then decided to change the arrangement of the joints. Since the settlement of the structure was largely complete, cement grouting was used in the joints instead of the plastic bitumen. If the joints open as the result of further movements, additional groutings can always be made. During construction, therefore, the holes were filled with fine gravel, the injection tubes were placed at different levels and the grouting was carried on. Then the upper parts of the holes were closed with a reinforced concrete slab.

The construction of the cutoff wall was carried on by alternate timbered excavations about 10 to 15 m long.

At the beginning, quicksand raised great difficulties, because drainage of the drill holes by pumping resulted in the removal of the fine materials.

Instead of pumping, therefore, the whole water table was lowered. Drain shafts, about 8 m apart, were dug all around the excavations for the cutoff wall. The nature of the drains made it necessary to equip each borehole first with a sand filter, then with a metallic filter. The pumps were of the reciprocating type used for deep boring. The discharge of drainage was quite variable, ranging between 0 and 30 lit/min for each shaft.

Because the formations were finely bedded, drainage occurred through a steady drip of water from all the strata exposed in the shafts, rather than by a real lowering of the water table.

Thanks to this radical, but expensive, method, the excavations for the cutoff walls were carried on without any incident.

5. The impervious deck

The necessity of having a flexible cover was sufficiently established by the movements of the rock fill which have been described previously.

The cover is composed first of a smoothing layer of porous concrete applied directly on the rock fill. This layer is made in panels divided by horizontal joints which are filled with a bituminous mixture; thus, any differential movement of the structure will not cause separation of the flexible coat from the rock fill.

The porous concrete is coated with bitumen which was dissolved in gasoline, to aid the adhesion of the bituminous material to the concrete. The bituminous mix is placed in 2 layers 6 cm thick. The first layer includes a "Zimmermann" lattice which prevents large tearing. The bituminous mix was laid by means of a telpher line for the low part of the deck, and by a traveling crane for the upper part.

The bituminous mix is similar to that used for road surfacing. A complete study was required to obtain the optimum composition. Laboratory tests were carried on at Bou-Hanifia.

The impervious deck is exposed to harsh conditions. The temperature of a black body exposed to the sun may reach 70°C. By use of a protective cover over the bituminous mix the maximum temperature was reduced to 45°C. The slope of the dam face reaches 125 percent so that the bituminous mix has to be rigid enough not to flow down the slope in warm weather. On the other hand, it must be plastic enough to conform to deformations of the dam even at low temperature. Consequently the optimum composition had to be found. By choosing an aggregate with a minimum of voids and a dense bitumen (penetration 10/20) a rigid material was obtained, but it was extremely difficult to work and to place. By increasing the percent of fines in the aggregate and the bitumen content, placing was made easier, but the risk of flowing increased. A satisfactory solution between the two extremes was found. The sizing of the aggregate for the bitumen cover is as follows:

Diameter of the grains	Percent of each size fraction	Cumulative total percent
Fine gravels		
larger than 25 mm	2.9	100
25/18	17.64	97.10
18/12	14.65	79.64
12/5	19.96	64.81
5/2.5	6.75	44.85

Diameter of the grains	Percent of each size fraction	Cumulative total percent
Sands and filler		
2.5/0.63 mm	10.17	38.10
0.63/0.28	13.90	27.93
0.28/0.1	4.28	14.03
smaller than 0.1	9.75	9.75

For bitumen (20/30 penetration) content a proportion percent of the weight of dry materials was used.

The cover protecting the impervious deck is reinforced with continuous parallel wires, extending from top to bottom of the face. These were put up and welded at the site. The protective cover is suspended by its reinforcement from the crest of the dam. A layer of paper was placed under the protective cover in order to allow it to move independently from the impervious bituminous deck.

The bituminous mix was put in place from a movable working platform. It was compacted by means of an oblique roller which was suspended from the crest of the dam so that it could move horizontally across the face. Tests have shown that rolling made from top to bottom, although technically more simple, produces waves at the surface of the bituminous concrete.

The junction between the deck and the cutoff wall constituted a particularly difficult problem. The open trench between the base of the upstream face and the cutoff wall had to be able to contract as movements of the structures were not yet completely stabilized. It was decided to fill it with a bituminous mixture, which is strongly compressible and impervious. The whole thing was then covered with a blanket composed of 7 cotton cloths which were plastered with an emulsion of "flintkote" at the time of laying.

When the reservoir was first filled this device caused great disappointment. Under the increasing pressure of the water, which then reached a depth of about 15 m, the material poured into the joint compressed to such extent that the impervious cloths flattened out and finally tore on some projecting points. A leak, which was later located by means of fluorescein dye tests, developed, and a discharge of about 800 lit/min escaped through the hole.

Although this incident fortunately did not have serious consequences, it necessitated removing the water which had already accumulated behind the dam. This was done by pumping the lower trench, as no drainage had been provided for the water between the base of the upstream cofferdam and the dam itself.

Following this incident, the joint between the cutoff wall and the impervious deck was changed. A V-shaped trench provided with concrete keys was built and overlaid with a bituminous blanket covering the head of the cutoff wall.

V. Construction of the spillway and tailrace

1. Various designs

The spillway is one of the structures which underwent the most modification in the course of its construction. The project called for the construction of a trapezoidal canal.

The first plan was to go through the Chabet el-Guendoul ravine to discharge directly in the Oued el-Hamman. This alignment was the shortest.

As soon as the digging started, landslides occurred and the alignment had to be carried north into the Quaternary plateau in order to place the tailrace in formations which were judged to be more sound (10). Numerous drill holes and exploratory shafts were dug and a series of depressions in the marl bed were found along the route; this made it necessary to lower the elevation of the canal floor. The spillway sill was also moved downstream to take advantage of a rise in the marl.

2. Landslides of the banks

In 1937, when the excavations were not yet completed, landslides started on the canal banks.

The most serious ground movements, however, appeared at the intersection of the canal with the valley side, where the canal was to join the stream bed. These landslides increased and affected a large volume of ground.

They are attributed to the disintegration of the marl through ground-water action. In this area the ground water is strongly alkaline and charged with sulphates and alkaline carbonates.

3. The trapezoidal shape replaced by a semicircular profile

Following Prof. Caquot's advice, it was decided to give up the trapezoidal profile and to put in a lined semicircular canal which would resist pressure much better.

The idea for this modification was developed as follows:

The landslides were localized in the lower Quaternary marly formations, whereas the upper layers of the Quaternary, composed of conglomerates and gravels, were less subject to landslides in spite of being much faulted. The semicircular lining acted somehow as an inverted arch with its abutments resting on the stronger upper formations.

Whereas the lining for the trapezoidal canal was to be made of concrete grouted in place behind a large rolling form, an entirely different method was adopted for the semicircular lining. The latter is made of pre-cast concrete blocks designed after certain standardized types. These large blocks were laid by crane.

The downstream landslide zone also required changing and extending the alignment. Deepening the bottom of the canal was avoided. On the other hand, the end of the tailrace was placed on loose formations, carefully rolled.

4. Corrosion of concrete by gypsiferous water

The water, running through marl rich in salts and pyrite, was highly charged and gypsiferous,_/ and hence corrosion of the concrete

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_/ Sulphates expressed in SO_3: 0.5 to 2 gr/lit.

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was dreaded. To meet this eventuality, the sill of the spillway was provided with a concrete lining of aluminous cement; only the core was made of Portland cement. Further downstream, the lining was protected by another layer designed to prevent the contact of the ground water with the concrete. It was soon found out that even the aluminous cement did not resist the corrosive action of the water. This is probably due to the strong alkalinity,_/ as it is known that aluminous

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_/ Alkalinity expressed in CaO: 23 mg/lit.

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cement resists sulphated water perfectly if the solution is neutral (11). To prevent further corrosion of the concrete, a drainage trench was dug around the structures of the spillway crest.

5. Functioning of the stilling basin

The force of water which comes into the stream bed at a very high velocity (more than 10 m/sec) had to be broken.

The stilling basin, under construction in 1941, is a large circular basin, 100 m in diameter, with a beveled upper lip. Its purpose is not to stop erosion of the streambed but to prevent it from working upstream.

Functioning: The jet of water coming from the tailrace forms two symmetrical eddies, which strike against the outside wall of the stilling basin and flow over it. The impact against the wall sufficiently reduces the velocity so that material in suspension is deposited and cannot act as a scouring agent to undercut the structure.

VI. Conclusions

The description of the structures gives an idea of the complex problems which have to be solved, often in a very short time, to successfully carry out a large scale project such as Bou-Hanifia under such difficult conditions.

All the obstacles which nature accumulated day after day were finally overcome thanks to the tenacity of the engineers and to the perfect collaboration between the directors and the different contractors in charge of construction.

Among large irrigation projects, the Bou-Hanifia dam is a remarkable example showing what modern techniques can accomplish in spite of very unfavorable circumstances.

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