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DESIGN OF A SYSTEM OF WATER MANAGEMENT PLANNING
IN THE UPPER SENEGAL RIVER CATCHMENT

VOLUME 7

GOURBASSI, MANANTALI AND BOUREYA PROJECTS

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Chapter I

INTRODUCTION

7.1. INTRODUCTION

The present volume of the Final Report prepared by SENEGAL-CONSULT is devoted to a description of the schemes, which are to be constructed on the tributaries of the river Senegal. The barrage sites discussed here were selected after a comparative study of all the storage possibilities which SENEGAL-CONSULT's reconnaissances were able to locate.

Previously, in December 1968 and August 1969, SENEGAL-CONSULT had submitted interim reports to the United Nations, based on preliminary specifications for the storage works. Equally, the unit prices used to obtain a rough estimate of the construction costs had only been given a summary study. Both on the technical and on the economic level, all the envisaged sites had been studied on the same basis, in order to make valid comparisons possible.

The conclusions of these interim reports by SENEGAL-CONSULT were analyzed in detail by the United Nations. Representatives from the various United Nations organisations concerned with this scheme, together with SENEGAL-CONSULT, took part in working sessions in New York in March 1969 and in Dakar in February 1970. These discussions led to the selection of the most important sites and to the specification of the objectives to be attained.

SENEGAL-CONSULT was given the task of studying and defining the characteristics of 6 schemes on the river Senegal and its tributaries, on the basis of various hypotheses. These 6 schemes are the following :

On the river Senegal :	Galougo Petit Gouina Felou
On the river's tributaries :	Manantali (Bafing) Gourbassi (Falémé) Boureya (Bafing)

Of the schemes which have been studied, those on the river itself are dealt with in Volume 6 of the Final Report, and the three others are described in the present volume. An optimisation study - its results, with comments, are given in Volume 5 of the present Report - made it possible to fix the leading data of the storage reservoirs to be built on the tributaries of the river Senegal. This made it possible to decide the dimensions of the storage structures, to set up pre-projects and to specify construction costs.

It should, however, be pointed out that due to circumstances outside its control, SENEGAL-CONSULT was not able to collect as many geological and geotechnical details at the Boureya site as at Gourbassi and Manantali. This is why the Boureya storage dam has not been studied as fully as the others, principally as regards the specification of the characteristics of the dam which is to be built of laterite material.

Moreover, the optimisation study produced by SENEGAL-CONSULT has shown that the effect of the storage reservoirs on Guinean territory is very small as far as the river regulation at Bakel is concerned. It also showed that as regards the production of energy, the two storages at Boureya and at Koukoutamba are practically equivalent, both quantitatively and qualitatively. However, generating costs are particularly high, and it appears doubtful whether the construction of either scheme will prove economic in the near future. There are in fact sites along the Bafing in Guinea and on its tributaries where there are natural waterfalls which could be utilized economically. For this reason it is most unlikely that the Boureya and Koukoutamba storage reservoirs will be built, except perhaps in the very distant future, after the other available hydroelectric resources of the region have been utilized.

Although none of the criteria selected by SENEGAL-CONSULT has allowed one of these two sites to be definitely eliminated in favour of the other, it seemed appropriate to describe in the present volume the Boureaya storage dam, because its role in regulating the flow of the river Senegal at Bakel is more important than that of the Koukoutamba barrage.

Chapter II

MANANTALI PROJECT

7.2. MANANTALI PROJECT

7.2.1. General

The Manantali site lies on the Bafing, about 80 km in a direct line from the village of Bafoulabé where the river Senegal originates from the confluence of the rivers Bafing and Bakoye. Approximately 3.6 km upstream from Manantali, the Bafing runs in a westerly direction. Here the rocky northern flank of the valley forms a well-marked promontory in the direction of the river bed. At this site, therefore, the cross-section of the valley is a minimum. The choice of this position for the siting of a dam follows as it were automatically, the more so because the regular profile of the terrain renders the site particularly suitable for such a purpose.

The topographic conditions of the Bafing valley upstream from the Manantali site make it possible to envisage the creation of a very large storage reservoir. In fact, a dam rising to a height of 90 m above the level of the terrain would make it possible to create a storage volume of 3000 million m3. Topographically, the maximum height of the dam is limited to that of the lower flank, i.e. that on the left bank; this rises to a height of 240 m.a.s.l., 92 m above the river bed.

The optimisation study of which the results are given in Volume 5 of the present Report, has shown that the specific investment per m3/s of regulated flow at Bakel passes through a minimum for a regulated discharge of about 500 m3/s. This would correspond to the creation of a useful storage of 25000 million m3 at Galougo. While such a storage reservoir could indeed be set up equally well at Manantali, it would represent a slightly greater investment. In addition, such a reservoir would be less effective than one at Galougo because the Bafing carried less water.

However, it is not very economic to build the Galougo barrage as part of the first stage, for the full capacity of this scheme would only be utilised after several decades. Indeed, both the need for irrigation and energy requirements will grow progressively, and a regulation of 500 m3/s at Bakel will only be justified economically in the final stage of the regional development with which the project is linked.

For this reason, a regulating barrage which does not need such a large investment should be provided for in the first stage. This structure ought to take into account the medium-term needs of the various beneficiaries of the regulation. From among several possible solutions, the Chiefs of State of the OERS member countries in March 1970 decided on a regulated flow of 300 m3/s at Bakel for the first stage of the scheme. The most economic storage structure for achieving this objective is a barrage at Manantali, as demonstrated in the interim report by SENEGAL-CONSULT.

In the discussion which followed the submission of the interim report, the United Nations called for the Manantali dam to be studied taking into account various complementary roles which the Manantali storage reservoir could play while still keeping to the main objective of a regulation of 300 m3/s at Bakel. In the first instance, it was a question of studying the possibility of a guaranteeing that the annual Senegal floods would start on a set date. Secondly, it was desired to analyze the economic aspects of reducing the flood peaks to a greater or lesser degree.

From among the several possible solutions set out in Volume 5 of the present Report, SENEGAL-CONSULT chose the one which appeared to lend itself best to the progressive development of the Senegal valley, and best to satisfy the often contradictory requirements of the different beneficiaries of the regulation. The project presented in the sections which follow, makes it possible to construct at Manantali a storage reservoir with a live volume of 10000 million m3. Such a storage makes it possible to regulate the flow of the river Senegal at Bakel in successive steps of 100 m3/s up to a discharge of 300 m3/s. With such a stage-by-stage regulation it is possible to guarantee a flood of at least 3500 m3/s during the month of August when the regulated flow at Bakel is about 100 m3/s. and a flood of at least 3000 m3/s during the same month when the regulated flow is about 200 m3/s. Moreover, this storage capacity makes it possible to increase progressively the guaranteed output furnished by the Manantali power station which increases from 30 MW to 40 and then to 100 MW, while the regulation at Bakel is 300 m3/s. The characteristics of the storage structure described here moreover make it possible to reduce the peak of a centennial flood at Bakel to that of a decennial flood. A paragraph must be devoted to analyzing the incidence of the different options taken by SENEGAL-CONSULT.

The mean annual flow of the Bafing at the Manantali site calculated on the basis of the period 1902 - 1968, is 377 m3/s. Minimum and maximum annual discharges are respectively 221 and 528 m3/s, recorded respectively in 1945 and 1924. The total catchment area is 27800 km2. A statistical analysis of the flood discharges has been carried out from observations along the Senegal and its tributaries. This has made it possible to establish for the Manantali site the magnitude of floods of various frequencies of occurrence, as required for dimensioning the structures. Thus, the decennial flood discharge amounts to 3820 m3/s, the centennial flood to 5600 m3/s, and the millenial flood to 7400 m3/s.

7.2.2. Topography and morphology

At the site of the dam the river flows in a bed which is very little confined by the banks. The width of the river bed is about 250 m and its bottom is at an elevation of approximately 148 m.a.s.l. The banks are rather flat at first, near the water, but then rise sharply in abrupt cliffs which unambiguously limit the cross-section which can be shut off by a dam. Although the cliffs on the right bank rise above an elevation of 270 m.a.s.l., those on the left bank are cut off at a height of 240 m.a.s.l., which determines the maximum height of the storage dam. Above that level the crown length increases in a prohibitive manner. As a result, and allowing a certain margin of safety, the maximum level of the storage reservoir should not exceed an elevation of 230 m.a.s.l.

The valley bottom is covered with scattered river deposits, and coarse blocks from the foot of the cliffs. A few small monadnocks rise from the plain.

7.2.3. Geology and geophysics

7.2.3.1. Stratigraphy

The rock series consists of greywackes and siliceous pelites at the base and quartz sandstones on top.

The lower portion does not show up on the surface but has been traversed by three out of the six boreholes which have been carried out at Manantali.

The greywackes and quartz sandstones are solid detrital rocks formed from cemented sands. These sands consist of quartz in the case of the quartz sandstones, and also have a strong proportion of feldspar in the case of the greywackes. In the observed samples mica is very rare. The siliceous cement is coloured by iron oxide. The grains are generally rounded and have a mean diameter of 0.3 mm.

The pelites are formed by a very fine and abundant cement containing silica and iron; some detrital elements are present in the cement in the form of rounded grains of quartz. Coarser beds alternate with finer strata.

Jaspers are rocks of chemical origin with a banded texture. They constitute strongly oxidised siliceous beds.

The quartz sandstones form high cliffs, while the jasper-bearing pelites are located in the valley, upstream from the dam.

7.2.3.2. Tectonics

The sandstones forming the cliffs show a very slight dip towards the southwest. They represent an intersecting structure charactristic of shallow marine deposits.

Of the four fracture systems observed, the most important is orientated from WNW to ESE. It was created by vertical movements of large amplitude which have lowered by at least 35 m a central block situated on the left bank. The geological map shows the most important fractures and faults. The geological profile gives an idea of the appearance of the subsoil. The location and direction of the faults are only estimates, however, and supplementary prospecting work will make it possible to specify their size and number more accurately.

7.2.3.3. State of the bedrock

The massive homogeneous and impermeable quartz sandstones change lower down either to greywackes through an increase in feldspar content, or to pelites through an increase in the proportion of cement (see the result of drillings Ma 1 and Ma 3). These two kinds of rocks are both impervious, but the greywackes are slightly less strong than the sandstones. The pelites by contrast have a schistose structure as well as a high percentage of oxidised cement, which renders the rock more fragile.

These are rocks of inferior geotechnical quality, a drawback which in the present case does not matter too much.

According to the available information, the faults explored by boreholes are sound and should not present insuperable difficulties for the project, although fairly extensive grouting will be needed. The situation and the characteristics of these faults will have to be carefully studied when the details of the project are being worked out. It must be stressed, however, that in no case are there any active faults.

The geological profile which has been determined at right angles to the line of the valley, shows noticeable differences in absorption between the sandstones and the underlying pelitic complex. There is also the effect of the proximity of the river bed upon those same sandstones. It should be noted that the results of the permeability tests hardly ever correspond to the appearance of the drill cores where the sandstones appear very frequently in the schists which break up once they are decompressed. One also finds numerous ruptures which can be deduced more or less from the shape of the curves, without however it being possible to specify the exact pressure.

The water level measured in boreholes M 2 (4.6), M 5 (6.10) and M 6 (6.25) is substantially higher than that of the river. This shows that during the dry season there is a slight flow from the water table to the river.

7.2.3.4. Quality of loose rock

Loose rock consists mainly of debris and of the fluvial deposits of the Bafing. There are also encrusted laterite soils, and sandy mud due to slowly moving water.

The debris covering the foot of the cliff is very coarse and seems to pass straight into sound rock without any transition zone. The cover of mud or alluvium is very thin at the dam axis. Near the Ma 2 borehole where it appears to be thickest, it reaches less than 5m.

7.2.3.5. Construction materials

A few kilometers to the north of the dam there is a 2.5 km wide zone where deposits of fine material are found. The wells used to supply water to the villages are excavated in clayey and sandy silt, to depth of over 5 m. These deposits arise from the valley becoming filled by an accumulation of fluvial deposits upstream from the Manantali rock sill.

Rocky material can be won in quarries in the sandstone cliffs. This material can be used either as rockfill, or as aggregate for making concrete.

7.2.3.6. Storage

In the area affected by the storage the rocks are impervious. The faults which have been discovered there are closed and shallow, as boreholes carried out at the dam site have shown.

It is however possible that some of these fissures allow water to filter through, in general to an insignificant extent. If the storage level were increased to an elevation greater than 230 m.a.s.l., it is possible that these seepages might increase very markedly.

7.2.3.7. Geotechnical prospecting

Six boreholes with a total length of about 200 m have been carried out. The first four are situated along the dam axis, and the remaining two which are each 10 m deep are between the village of Soukoutali and the dam axis, their purpose being to check on the existence of a geological fault. This fault was clearly demonstrated, as shown by the geological profile and the borehole profiles.

Intact core samples were extracted from the two boreholes on the left and right bank (Ma 1 and Ma 2) and have been used for laboratory tests.

An exploration by seismic refraction tests has not been considered necessary for this dam site. In the course of subsequent investigations it would be useful to undertake a geophysical campaign including seismic and resistivity tests, so as to facilitate the study of faults and find how they affect the dam design in its details.

Three pits have been excavated upstream and downstream from the dam axis in areas considered to be likely sources of the necessary materials for the construction of the dam. These pits showed the depth of the bedrock. The investigation indicated that over an area of some 6 km2 the thickness of the alluvial layer varied between 2.30 m and 9 m. These alluvia are therefore available in sufficient quantity for constructing an earth or rockfill dam with impervious core. It should be noted, however, that the variations in thickness of the deposit will render its exploitation a costly business.

Seven samples of material extracted from these pits have been examined and tested in the laboratory. The average clay content amounts to 22% and the liquidity and plasticity limits are respectively 21 and 11%. A standard Proctor test has given a density of 2.01 t/m3 with an optimum water content of 12%. The internal friction angle of this material amounts to 30^{o} and its permeability is of the order of 10^{-8} cm/s. The coefficient of pore pressure according to Skempton is about 30%, which is relatively high. This material therefore has properties which make it possible to envisage using it for constructing the core of a dam. Detailed test results are given in Volume 9 of the present Report, entitled "Geological Appendix".

7.2.4. General layout

Local geological conditions do not imperatively dictate the precise siting of the dam. For this reason the dam axis was aligned in accordance with the topography of the area.

Moreover, both geological and geotechnical conditions equally favour either a concrete dam, or an earth or rockfill dam.

As part of the preliminary study, the results of which were published in the Interim Report of December 1968, different types of structure had been analyzed and compared among themselves. The main features of these different pre-projects were as follows:

- a homogenous earth dam of alluvial material with uniform talus slopes of 1:1.3
- a rockfill dam with central clay core, having talus slopes of 1:1.5.
- a rockfill dam with a concrete lining covering the water face again with talus slopes of 1:15.5.
- a buttress dam of classical type, consisting of 22 m wide elements with double buttresses.
- a multiple arch dam in plain concrete consisting of arches of 50 m span.

Provisional estimates for each of these projects and for two different heigts, have shown that the first four solutions studied differ among themselves only slightly as regards cost. By contrast, the last solution, i.e. the multiple-arch dam, appeared to offer a substantial saving. For this reason the last mentioned type of structure had been proposed in the Interim Report.

During the second phase of SENEGAL-CONSULT's studies and once the dimensions of the storage structure had been fixed, it appeared to be justified to take up again the comparison of the different types of structure, on the basis of unit prices, which due to more detailed studies, had been worked out with greater accuracy. In the course of this investigation, it became apparent that there were numerous arguments militating in favour of replacing the multiple-arch dam by a heavy buttress dam of plain concrete. In effect, the saving in concrete which a multiple-arch construction can bring about, is compensated by the additional cost of the formwork and the cost of the specialist work force which such a structure calls for. In addition, the construction of vaults demands accurate coordination in advancing the buttresses and the vaults themselves which must be carried out in such a way as to avoid asymmetrical side thrusts on the buttresses. By contrast, a buttress dam can be split up into different sections which for the most part can be carried out independently of one another, since each buttress is self-supporting. It should also be added that the choice of simple buttresses make it possible to envisage the re-use of the shuttering on a much wider scale than could be achieved in the case of a multiple-arch dam.

This is the reason the project presented in the present Report differs from that described in the Interim Report. It must however be stressed that there is less than 10% difference in cost between a heavy buttress dam and a mixed structure consisting of a rockfill dam with impervious core and a section of concrete dam. In these circumstances it is difficult to affirm in the present state of our knowledge that a concrete dam is really more economic. It will therefore be indispensable during the next phase of the studies, to take up again this comparative analysis, and probably prepare a detailed pre-project for each of the solutions.

In fact it is likely that the final choice can only be made when tenders for carrying out the work have been submitted.

7.2.5. Dam

The Manantali dam has been dimensioned for the accumulation of a live storage volume of 10000 million m3. The dead volume which has to be retained in order to compensate for the silting up of the reservoir, has been fixed at 1000 million m3 on the basis of the hypotheses outlined in Volume 5 of the present Report. It corresponds to the volume of material which could be deposited in the reservoir in 100 years, assuming an average concentration of solids capable of being deposited, equal to about 1 g/litre. Such an annual rate of deposition corresponds to an overall erosion of the catchment area of 0.35 mm/annum. Erosions of this magnitude have been observed in connection with siltation measurements on reservoirs in New Mexico (USA). However, the morphological characteristics of tne catchment basin of the Bafing lead one to expect that the overall erosion will be smaller.

The Manantali storage volume includes an extra slice above the normal storage level for the purpose of lopping flood peaks. Properly used, this slice allows reducing discharge at Bakel from a centennial flood to that of a decennial flood.

On these assumptions, the main characteristics of the storage and dam are the following:

- normal reservoir level	207.50 m.a.s.l.
- highest high-water reservoir level	214.00 m.a.s.l.
- lowest reservoir level	175.00 m.a.s.l.
- crown height	215.50 m.a.s.l.
- max. height above foundations	73 m
- width of valley at crown level	1480 m
- concrete volume of dam	1.390.000 m3
- brutto storage volume	11.1 10^9m3/s

The dam comprises 95 buttresses, i.e. 85 simple buttresses and 6 double buttresses. The buttresses are arranged in several groups of the same height and are practically founded on horizontal planes, except at the two ends where the buttresses are completed by two concrete gravity abutments, 18 m long on the left bank and 48 m long on the right bank.

The maximum width of the waterface of the normal buttress being 14 m, the dam as a whole comprises the following arrangement, going from the left bank to the right:

- left-bank gravity abutment	18 m
- 42 standard buttresses on left-bank	588 m
- 8 buttresses housing the intakes	112 m
- 1 double buttress housing the bottom outlet	28 m
- 5 double buttresses housing the spillway	140 m
- 39 standard buttresses on right bank	546 m
- right-bank gravity abutment	48 m
- total length of dam	1480 m

The 89 standard buttresses are 14 m wide on the upstream side. Six double buttresses are each 28 m wide on the upstream side; five of these constitute the spillway block and one houses the bottom outlet. These double buttresses are formed by joining two standard buttresses. The upstream and the downstream bore have identical slopes of 1 : 0.45 and their planes intersect at an elevation of 215.50 m corresponding to the crown level. The waterface has been designed for maximum strength under its own weight and the water pressure. It rests by way of an intermediate gusset on a 4 m thick triangular bulkhead. The foundation of the waterface is reinforced by a rectangular slab footing 14 m wide and 8.50 m long which is also connected to the bulkhead by a gusset. The foundation of the bulkhead is strengthened by a 6 m wide footing, extended to 10 m at its downstream extremity over a length of 12.15 m by means of a connecting gusset. The downstream end of the bulkhead is gradually wider from elevation 165 m to where it joins up with the foundation footing.

This arrangement represents a threefold advantage; the pressure on the rock foundation is reduced; applied forces are efficiently transferred to the concrete and the support section of the buttress, for as the reservoir fills, the position of the resistance on the horizontal sections changes only very little; and finally, the buttresses achieve a certain transverse rigidity, enabling them to withstand any possible lateral forces. The maximum pressure on the rock foundation does not exceed 19 kg/cm2, and the maximum stress in the concrete is less than 39 kg/cm2.

The upper portion of the water face rises vertically between elevation 208.85 and 215.50. In that zone, the total thickness of the waterface varies from 6 m to 3 m. The buttresses are to be built entirely from plain concrete.

The average depth of excavation has been fixed at 5 m under the waterface and at 3 m under the downstream end of the buttresses. Treatment of the foundation rock will comprise the following three stages :

The principal grout screen is vertical and has a maximum depth of 55 m. It extends along the dam underneath the waterface foundation of the buttresses and will be carried out by means of drillholes, the spacing of which will be fixed after a detailed investigation of the rock condition.

ontact grouting under the base of the buttress waterfaces will include two vertical drillholes per buttress, 15 m deep and situated upstream of the waterface, s well as two 15 m deep drillholes downstream from the main waterface and sloping t 30° to the vertical. In addition 4 drillholes each 15 m long and inclined at 0° to the vertical, will be carried out under the downstream footing of each uttress, two being raked in the upstream direction and two in the downstream irection.

ntact grouting under the foundation of the bulkhead wall of each buttress, will e done by 5 m long vertical drillholes arranged in a quincunx pattern in three ws, the drillholes of each row being spaced 10 m apart.

In order to ensure adequate drainage of the foundation rock downstream from the main grout screen, it is planned to carry out two types of boreholes, both 75 mm in diameter. These are, on the one hand two 25 m deep vertical boreholes, each on the longitudinal axis of the cell formed by two consecutive buttresses; on the other hand, one borehole will be carried out in the axial plane of each buttress, starting from the gallery in the upstream footing which descends underneath the buttress at 45° towards the downstream side. It will be extended as far as its intersection with the principal axial plane of the dam.

A 6 m wide road connecting the two banks, will be constructed on the crown of the dam. The parapet on the reservoir side will be 1 m high and will act as a supplementary protection against waves.

7.2.6. Control devices and spillway

In the final stage of the Senegal regulation, the spillway of the Manantali dam will only have its conventional role to play, i.e. to form an outlet for the storage in case of floods. By contrast, during phases of agricultural amelioration of the Lower Senegal Valley, it will be necessary to regulate and control the annual flood in such a manner as to make it possible to carry out the traditional after-flood cultivation. In this case it must be possible, if necessary, to bring about an artificial flood of 3500 m3/s at Bakel, even if the storage level at Manantali is close to its minimum operating level.

In order to fulfil this double role, two kinds of equipment have been provided, viz. intermediate gates and crest spillways.

The dimensions of the intermediate gates are fixed by those of the buttresses. In order to satisfy the conditions referred to above, it is necessary to have 5 gates each 5.60 m wide by 7 m high. The entry sill of these gates intermediate has been fixed at elevation 162. These 5 gates can discharge 5350 m3/s under a head corresponding to the normal storage level of 207.50 m.a.s.l., and 3500 m3/s for storage at elevation 181.50 m.a.s.l. When the reservoir is at its minimum level at èlevation 175 m.a.s.l., the five gates can still discharge 2800 m3/s. In fact it did not seem justified to dimension the intermediate gates for a discharge of 3500 m3/s under the minimum water level, for it is very unlikely that during a flood the storage level will remain at its minimum for long.

At the top of the five buttresses which accommodate the intermediate discharge gates, are spillways with fixed sills for discharging flood water. In addition to its double role of regulating the discharges and controlling the minimum flood, the Manantali reservoir will also act as a buffer during exceptionally high floods. The objective in lopping the flood peaks is to reduce the peak discharge of a centennial flood at Bakel to one corresponding to a decennial flood at the same place. Analysis of the hydrograph of the centennial flood has shown that it will be necessary to retain a volume of 2400 million m3 until the moment when the discharge at Bakel falls below that of the decennial flood. This condition is fulfilled by having crest spillways as provided in the present project. The crest spillway has five outlets each 24 m wide and with different sill heights.

The central outlet has its sill at elevation 207.50 m.a.s.l., corresponding to the normal storage level. The two intermediate outlets are arranged at elevation 209. m a.s.l., and the two top outlets at elevation 210.50 m.a.s.l. This arrangement of outlets, coupled with the appropriate operation of the intermediate sluice gates, makes it possible on the one hand to store a volume of 2400 million m3 above the normal storage level, and on the other hand, to regulate the discharge from the Manantali reservoir, in such a way that the discharge at Bakel does not exceed the decennial flood.

The crown level of the dam has been fixed in relation to the maximum water level reached in the reservoir during a millenial flood. For the purpose of calculation it was assumed that one of the five intermediate sluice gates had remained stuck in the closed position during the passage of the flood. In order to take the most unfavourable conditions into account, it was supposed that the millenial flood occurred during flood peak lopping, i.e. with the storage level at Manantali at elevation 212.50 m.a.s.l. and the extra storage of 2400 million m3 associated with operating the crest spillway being full. In that case, also taking into account the shape of the hydrograph of the flood entering the storage reservoir, and the fact that intermediate sluice gates would be opened progressively, it seems that a maximum reservoir level of 214 m.a.s.l. will be reached, that is to say 1.50 m higher than the level of the crest-overspill storage provided for abating the floods at Bakel.

The crown level of the dam has been fixed 1.50 m above the maximum level reached during the millenial flood. This excess height of 1.50 m should be adequate for the dam is a concrete structure which can be washed by waves without danger. Moreover, a 1m-high parapet gives additional protection against the waves. In addition, due to the shape of the reservoir, the dam is not at right angles to the main reservoir axis, but on the contrary is parallel to it, which reduces the height and force of the waves which could break against it.

The 5 intermediate outlets can be partially or completely closed with the aid of a sector control gate which can be operated by a servomotor. Beyond this gate, the outlet continues in the form of an open channel which slopes down gently to the level of the natural bed. A ski-jump placed at the end of this channel, project the discharge into a big trench, excavated from the river bed in order to get a good hydraulic drop, and in a fixed position. Into the same trench the discharges from crest spillways are projected by means of a ski-jump. A concrete apron extends the downstream foot of the dam in the area of the spillway, in order to obviate any possible erosion of the river bottom during the passage of smaller floods which would not be projected as far as the stilling trench.

The chosen layout thus permits an effective control of the Senegal floods. It does, however, require an increase in the height of the dam, which is not negligible.

If one relinquishes the imposed condition that the discharge of the centennial flood at Bakel is to be reduced to that of a decennial flood, the operating requirements for the intermediate discharge gates are considerably relaxed. It is also possible to give up the tiered layout of the crest spillways, and to fix the bottom of the spillways at the level of the normal storage, i.e. at 207.50 m.a.s.l. In that case, SENEGAL-CONSULT has recognized that during a millenial flood the gates would be opened progressively and that they would be fully open at the beginning of the sixth day of the flood. The maximum discharge evacuated by the intermediate openings and the crest spillways, in this case, reaches 5600 m3/s and the maximum level of the storage reservoir reaches an elevation of 210.4 m.a.s.l. The crown level of the dam would then be at the elevation 212.0 m.a.s.l., i.e. 3.50 m. above the level provided in the project described in the present chapter.

One could equally envisage a more rapid operation of the gate valves which would maintain the storage level constant until the moment when the natural discharge flowing into the reservoir became greater than the discharge capacity of 4 out of 5 of the intermediate discharge valves. In that case the maximum storage level during a millenial flood will reach 208.50 m.a.s.l., and the crown of the dam can be fixed at the elevation 210.00 m.a.s.l., i.e. 5.50 m lower than the structure described in this chapter.

The economic impact of the role which the Manantali dam could play in the control of exceptional floods will be dealt with in Section 7-2-19 below.

7.2.7. Intakes and penstocks

In order to facilitate the erection of the penstocks and avoid the risk of a delay in the concreting work, the intake structures are to be sited in the empty space between the bulkheads of two adjacent buttresses. Each penstock is supported by the bulkhead of one of the buttresses, which confers a certain advantage as regards the execution of the work; for it reduces to a minimum any interference between the two types of work (concreting and erection of penstocks) which in their nature are so different.

The intakes which are eight in number, are laid out for a combined discharge of 500 m3/s, i.e. 62.5 m3/s per intake. Because the penstocks are incorporated in the simple buttresses without any modification of their shape, the intake openings had to be rectangular. The dimensions are 3.20 m wide by 4.80 m high, which is a favourable proportion for sluice gates. The intake entrance sill is fixed at elevation 167 m.a.s.l., which leaves an adequate margin between the sill level and the minimum normal storage level in the reservoir at elevation 175 m.a.s.l., so as to prevent air entrainment into the penstocks while placing them at the same time at the highest possible level to stop silting up.

Each intake can be shut off by a sluice gate of classical type design which moves parallel with the waterface and can be operated by means of an oil-hydraulic servo-motor installed in a cabin under the crown of the dam. These valves can also be controlled from the power station and close automatically if a penstock ruptures, i.e. if the entry velocity surpasses a preset value. The penstocks have a diameter of 4 m each, and rest on brackets fixed to the bulkhead of the buttress.

7.2.8. Bottom outlet

The bottom outlet of the Manantali dam consist of two gates of 2.30 m width and 3.40 m height placed in a double buttress element of 28 m width. This is situated between the spillway block and intake block. The intake sill is fixed at elevation 155 m. Under the minimum head, corresponding to a reservoir level at elevation 175 m,

ey can evacuate a discharge of 300 m3/s. This is less than the design turbine
ow, but corresponds to the regulated discharge at Bakel during an intermediate
age of the development of the river Senegal basin. In fact, in the final stage of
e development, the Manantali reservoir will no longer participate directly in the
gulation for the Galougo dam which is situated downstream from Manantali, will
ay the principal role.

ch opening is provided with two vertical sliding gates, arranged in tandem, and
erated by an oil-hydraulic servo-motor. The upstream gate valve acts as emergency
te and as a standby to make it possible to overhaul the operating valve. Downstream
om the valves the two conduits come together in a slightly sloping common channel,
joining the level of the river bed on the right of the downstream foot of the dam.
ski-jump is provided at the end of this channel, in order to project the discharge
om the bottom outlet beyond the foot of the dam.

2.9. Construction stages

basic principle of the envisaged construction programme consists in maintaining
normal and continuous flow of the construction work, even if floods of the order
5000 m3/s, corresponding to a recurrence of once in 20 years, should happen during
e course of the work. If, however, floods above this level were to take place,
ey would merely cause a certain delay by flooding the work site, without endangering
e work already completed.

e first stage of the work will begin the moment the access road linking the working
ite with the town of Bafoulabé is completed. This will be timed for the beginning
f the dry season, in the course of which the foundations of those dam sections which
ontain the intakes will be excavated and then concreted to elevation 157.7 m. The
ork will be carried out in the shelter of a coffer dam capable of giving protection
gainst the low water during the dry season. The foundations of the dam sections on
he left bank will also be excavated and concreted during this period, in order to
ake it possible to concrete the upper portion during the first high-water period.
uring the second dry season, the blocks through which the water intakes and bottom
utlet will pass, will be concreted right up to crown level. The rest of the left-bank
xcavations and the concreting of the dam foundations to elevation 170 will also
e done during this period. In this manner it will be possible to complete concreting
he dam on the left bank during the second wet season. During the third dry season,
he dam foundations in the spillway zone will be excavated in the shelter of a temp-
rary cofferdam and concreted, to elevation 170. Similarly, the foundations of the
am on the right bank will be constructed during this period and the work carried up
to elevation 170. During the third flood season, with the discharge of the river
Bafing still flowing in the natural river bed, it will be possible to concrete
the upper portion of the spillways and part of the dam on the right bank. During the
fourth dry season, concreting of the dam on the right bank will be completed, with
the discharge flow of the Bafing being diverted through the intermediate outlets.

7.2.10. Hydroelectric power station

The hydroelectric power station of Manantali is situated on the left bank at the foot of the buttress dam. In plan, its position is near the spillway.

The station is of semi-outdoor design, with an exterior portal crane for erection and overhaul purposes; at its end facing the left bank it has a multi-purpose building comprising the control room, offices, stores, and a completely enclosed workshop into which the portal crane can enter completely. The workshop also includes installations for detanking the transformers which can be taken there in turn.

The utilized gross head is the difference between the upstream storage level which can fluctuate between elevations 175 and 207.50 m.a.s.l. (minimum and normal storage levels), and the downstream restitution level which depends on the discharge. The station has 9 vertical Francis turbines and is designed for a maximum turbine flow of 560 m3/s under minimum head.

The main characteristics of the power station are the following :

- Number of sets	:	8
- Turbine flow at maximum head	:	345 m3/s
- Turbine flow at minimum head	:	560 m3/s
- Guaranteed output	:	100 MW
- Installed capacity	:	150 MW

7.2.11. Mechanical equipment of power station

Each Francis turbine is coupled to a generator and comprises a steel intake spiral and a cylindrical slide valve which is inserted between the stay-bars of the spiral and the distributor vanes. This valve is controlled by oil-hydraulic cylinders and takes the place of an inlet valve of classical type (butterfly or spherical). Each turbine is fitted with a speed governor.

At the exit of each draught tube, stoplogs are provided together with a portal crane for operation.

The principal data of each turbine are the following:

- Net head, m	20.0	30.0	34.5	52
- Discharge, m3/s	27	72	63	43
- Output, MW	12.5	19.0	19.0	19.0
- Speed, rev/min	150	150	150	150

The thrust bearing carrying the vertical load of each set is mounted underneath the generator, either on separate beams, or on a pedestal forming part of the top of the turbine casing. Dismantling of the turbine is carried out through the generator stator.

Because of the climate, cooling the machines poses a special problem. At the present stage of the design studies, it is planned to provide a secondary clean-water circuit, with heat exchangers specially designed to facilitate cleaning.

An auxiliary set is provided to supply the requirements of internal and local services (lighting, oil pumps, drainage pumps, hoists, etc). This is a horizontal turbo-generator set and will give complete independence from the grid. It is placed above the highest flood level in the generator bay.

7.2.12. Electrical equipment

The electrical equipment of the power station comprises 8 vertical generators situated in the generator bay above the highest flood level. Each has a rating of 22.5 MVA, and is fitted with closed-circuit ventilation and watercooling of the air.

On the medium-voltage side, the generators are connected in pairs, by means of a busbar system fitted with circuit-breakers, to groups of three single-phase transformers. These transformers are situated outside the power station on the downstream side, and each group of single-phase units has a rating of 45.0MVA.

ransmission of the energy generated is carried out by means of 4 high-voltage onnections of three 220-kV cables, placed in a tunnel leading to the switchyard ach cable is connected at the power station end to one of the single-phase transormers, and ends at the other in an outdooor-type terminal box.

he overall efficiency of the electrical equipment of the sets under nominal operating onditions, is of the order of 96%.

addition the electrical equipment comprises:

- a medium-voltage system for connecting the internal station services to the local mains supply.
- a low-voltage distribution system for the internal services of the scheme
- all installations for control, indicating safety and measuring
- all installations for lighting and earthing.
- the communication system

7 .13. Switchyard and high-voltage transmission lines

T nsmission of the energy to the centres of consumption is at 220 kV by an high-v age overhead transmission line which starts at the switchyard. The latter is s uated near the dam.

Th 220-kV switchyard receives, dispatches and distributes energy to the different st ting points of the transmission line. It has two busbar systems with mixed phases.

Th switchyard installations comprise the following items:

- the high-tension equipment (apparatus and busbar system) required for the reception, dispatching,and sending out of energy
- the metallic support structures for the electrical equipment together with their foundations
- the auxiliary equipment for checking, controlling and actuating the equipment as well as for communicating with the power station. For the most part these are housed in a service building.

The switchyard is constructed on a flat area of land, appropriately drained and equipped with the necessary roads and paths for the movement of personnel and material, and fenced.

7.2.14. Power Generation

Calculation of the power production has been confined to establishing the guaranteed generation, i.e. the energy supplied by the power station at the guaranteed power during 8000 hours per annum, nine years out of ten. This guaranteed power has been fixed for the Manantali scheme for each of the three regulation stages which this dam makes possible.

The calculations were based on a mathematical model worked out by SENEGAL-CONSULT which makes use of the hydrological data from 66 years of observations. The table shown below quotes the figures which were calculated in Volume 5 of the present Report.

	1st stage	2nd stage	3rd stage
- Regulated flow at Bakel	100 m3/s	200 m3/s	300 m3/s
- Guaranteed flow at Bakel in August	3500 m3/s	3000 m3/s	not prescribed
- Guaranteed power	30MW	40MW	100MW
- Guaranteed generation	2400Wh	3200Wh	800GWh

7.2.15. Access to site

From Dakar transport will be carried out by the Dakar-Bamako railway as far as Dioubéba which is situated on the Bafing upstream from its confluence with the Bakoye. From there transport will be by road. The existing track does not however correspond to what is needed for the kind of construction work here contemplated. As a result it is indispensable to improve the track in order to render it suitable at all times for the heavy convoys supplying the building site with equipment and materials. A last stretch of 13 km still remains to be constructed in order to join the track and building site. The distance between Dioubéba and the site is about 68 km.

7.2.16. Re-siting of villages

The surface of the Manantali reservoir with storage at elevation 212.50 m, will be nearly 510 km2.

A census has been carried out by SENEGAL-CONSULT on the basis of serial photographs taken by IGN in 1967 for updating the 1:20 000 map of the reservoir area. This has shown that there are 68 villages in the inundation zone, representing (in accordance with the hypotheses outlined in Volume 1A of the present Report) about 3800 huts with a population of farmers and fishermen of nearly 6000 personnes.

These villages are listed in an appendix. Their re-siting will depend on political tribal, agricultural, and other factors.

7.2.17. Re-siting of roads

At present there are no trunk roads in the future inundation area, which will be inundated by the reservoir. On the other hand, as regards secondary roads, the project provides for the completion of a road system round the reservoir in the shape of 5 sections of secondary roads totalling 142 km.

Furthermore, in estimating the cost of the work, account has been taken of the construction of 50 km of cart tracks which will link the future villages on the shores of the lake to the road system.

These tracks will be routed according to the future development of the territory.

7.2.18 Construction Schedule (see Appendix 7.2.15)

The construction schedule has been arranged so as to give a rational utilisation of the site installations, i.e. by arranging for an orderly flow of the work at every stage. The requirements of the climate, as well as the construction methods, are also taken into account.

The programme for constructing the access road connecting Bafoulabé with the site at Manantali, has not been analyzed in detail in the present study because it is difficult to determine the optimum sequence of operations which varies depending upon how the contractors' equipment and plant can be employed. In order, however, to decide on a preliminary programme of expenditure, it appeared justified to assume that the construction of the access road could be carried out during the three years preceding the beginning of construction of the dam.

On the other hand, re-settlement of the populations would only occur shortly before the flooding of the reservoir. In fact, the villages which exist at present on the banks of the Bafing are dependent upon it, for it is the presence of water nearby which renders possible the life of these populations.

Construction work on the dam will begin at the start of the dry season with the setting-up of the site installations and carrying out the excavations on the left bank.

The sequence of operations has been outlined in detail in the section devoted to the stages of construction. In broad outline, it is planned first of all to carry out the concreting on the left bank, then on the right, in order to utilize in a rational manner, the facilities for placing the concrete. Concreting of the dam will be carried out in four campaigns each lasting one year. The volumes of concrete which are to be placed in these four campaigns are the following:

- 1st	concreting	campaign	270 000 m3
- 2nd	"	"	410 000 m3
- 3rd	"	"	450 000 m3
- 4th	"	"	260 000 m3

The execution of such a programme required installations with a capacity of the order of 40 000 m3 of concrete per month, or 2000 m3 per day.

The programme was set up so as to enable the foundations of the power station to be dug and concreted during the first stage, so that at least some of the sets could be completed by the time the dam was ready to be flooded. In this way erection of the hydroelectric equipment can start 2.$\frac{1}{2}$ years after the beginning of the work. The duration of the erection work will depend on the commissioning rythm which be adapted as much as possible to fit in with energy demands. In the framework of the present study, the optimum case has been envisaged regarding the duration of the work. The erection programme provides for sets to be commissioned in pairs, each commissioning being scheduled to occur 1.$\frac{1}{2}$ months after the last. This way of proceeding in an optimum as such as regards the construction of the sets in the factory as their erection on site.

In these circumstances, the commissioning of the last two sets of the Manantali power station could take place five years after the commencement of the work as such.

7.2.19. Construction cost

The cost of constructing the Manantali dam and power station has been based on the unit prices set out in Volume 1A of the present Report. These take into account the geographical situation of the site and in particular the magnitude of the transport costs by rail as far as Mahina and by road from Mahina to Manantali.

Improvement of the access road to the site, as well as the construction of secondary roads and the re-siting of villages which are to be rebuilt on the shores of the Manantali reservoir, have all been the subject of detailed studies. The resulting costs have been calculated on the basis of the unit prices which are also given in Volume 1A of the present Report.

Table 7-2-I shown at the end of the present chapter 7.2. summarizes the quantities pertaining to the chief categories of construction work as well as the estimated cost of each category of work. This refers to the solution proposed by SENEGAL-CONSULT by which exceptionally large floods at Bakel can be effectively controlled. The main heads of work, as well as miscellaneous and contingent items, have been grouped under each contract and are calculated on the basis of the prices given in Volume 1 A. In this account of the estimates another chapter is devoted to the cost of rebuilding roads and villages, and a third gives an estimate of the cost of studies and finance during the course of the work. These last-mentioned costs have been worked out on the basis of the prices shown in Volume 1A of the present Report.

For the fully equipped power station (150MW installed capacity) and taking into account the interest on capital payable during the course of the construction, the total investment required for building the Manantali dam and power station amounts to 115.4 million US \$.

However, the intention is to install sets in the power station only as required by the demand for energy. Thus, there will be an initial stage during which it will be possible to guarantee at Bakel a regulated flow of at least 100 m3/s and a flood in the month of August of at least 3500 m3/s; at this stage it will be possible to supply a guaranteed output of 30MW, calling for the installation of 3 sets in the power station. Supposing that interest-free finance can be obtained for this first stage, the necessary investment will be as follows:

<u>Stage 1, 30MW guaranteed, interest-free finance</u>

Dam - civil engineering work	59.5 10^6 US \$
equipment	3.7 10^6 US \$
Power station	
- civil engineering work	3.5 10^6 US \$
- equipment (3 sets)	8.3 10^6 US \$
onstruction cost	75.0 10^6 US \$
oads and resiting of villages	11.3 10^6 US \$
ost of studies	7.0 10^6 US \$
apital charges	NIL
Total investment	93.3 10^6 US \$

he additional investment to complete the installation of equipment in the power tation, amounts to 9.3 million US \$, including the further costs due to opening new working site.

s explained in section 7.2.6. above, by renouncing complete control of exception- lly heavy floods, it becomes possible to reduce the height of the storage dam 5.50 m. Such a modification of the project would yield a substantial saving thout lessening the importance of the role played by the Manantali dam in river gulation and power production. The chief relaxations in the quantities are the llowing:

Excavations	50 000 m3
Concrete	210 000 m3
Formwork	80 000 m2

The total of the resulting possible savings amounts to 7.0 million US $.

In these circumstances, the investment required up to and including the first development stage of the power station would be 86.3 million US $.

One could equally envisage postponing to a later date the installation of the first sets of the power station, and building in the first stage only the foundations of the power station as far as this is strictly necessary.

Under this assumption, and always assuming the idea of complete flood control is given up, and interest-free finance can be obtained the total investments required for constructing the Manantali dam would be reduced to 76.5 million US $.

7.2.20. Investment programme

The investment programme is represented graphically in Table 7.2.III shown at the end of the present chapter. It is based on the construction schedule described in section 7.2.18 and covers the following extreme cases :

(a) Full flood control (crown of dam at elevation 215.5 m.a.s.l.) installation of all 8 sets in the power station, interest charged on capital.

(b) Abandonment of full flood control (crown of dam at elevation 210.0 m.a.s.l.) installation of power station equipment postponed until later, no interest charges during construction.

These programmes take into account a delay of three months between the completion of work and payment being made for it. as well as sums retained under guarantee clauses and released 24 months later. Moreover, the fact is taken into account that the access road to the site will be completed just as the work is about to commence and that it will be paid for at that time.

7.2.21. Energy production and generating cost

The Manantali scheme falls into the category of mixed schemes of which the main objective is to help regulate the flow of the river Senegal. Its part in this regulation will however be modified as agricultural development in the Lower Valley goes through different stages. In fact, as has been said at the beginning of the present chapter, different stages of the regulation are being planned by SENEGAL-CONSULT in the course of which the regulated flow at Bakel will increase in stages as well as the guaranteed generation of the power station, while there will be a progressive reduction in the size of the artificial flood which can be guaranteed in the month of August.

The guaranteed energy output will increase in stages and will reach successively the following values :

Stage 1 :	regulation 100 m3/s	at Bakel	240GWh
Stage 2 :	regulation 200 m3/s	at Bakel	320GWh
Stage 3 :	regulation 300 m3/s	at Bakel	800GWh

Generating costs for the guaranteed output have been calculated for the final stage of the Manantali scheme, i.e. on the basis of a guaranteed annual output of 800GWh. These generating costs have been worked out from two different viewpoints. In the first case, they take into account the annual charges relating to the complete scheme, in the second case only the charges relating to the power station equipment. Annual charges have been worked out on the basis of the assumptions and prices given in section 4.3.5. of Volume 1A of the present Report; they are listed in Table 7.2.IV appended at the end of the present chapter.

On the assumptions referred to above, the generating cost at station terminals works out as follows:

(a) on the basis of the charges for the full scheme :

9.1 mills of a US $ per kWh

(b) on the basis of the charges for the power station only:

3.0 mills of a US $ per kWh

7.2.22. Present state of project

In conformity with its mandate, SENEGAL-CONSULT has worked out a summary pre-project for the Manantali site which enabled it to make a preliminary estimate of the cost of a scheme capable of contributing, under the most advantageous economic conditions, to the optimum regulation of the river Senegal.

It is clear, however, that this is only a preliminary study made with the aim of finding the most satisfactory combination of schemes which can be built in the Upper Senegal basin. The construction costs referred to thus remain relative in character and are valid only under present economic conditions. It will therefore be necessary to update these costs when the construction dates for this scheme have been decided, in order that a plan for financing the scheme may be set up. It will also be advisable to review the general concept of the scheme bearing in mind any decisions which may be taken in future regarding the desired degree of flood control.

Table 7.2.I

MANANTALI PROJECT

Normal reservoir level : 207.5 m.s.m.

Dam crest level : 215.5 m.s.m.

Cost Estimate

1.	CONSTRUCTION COST OF DAM AND POWER PLANT	Unit	Quantity	Unit Price US $	Price US $
1.1.	Dam and ancillary structures				
	Civil works				
	- Temporary diversion	forfait			1,000,000
	- Excavations				
	. in soft ground	m^3	305 000	1.05	320,000
	. in rock	m^3	410 000	3.05	1,250,000
	- Concrete, shuttering reinforcement				
	. mass concrete	m^3	1,335,000	31.5	42,100,000
	. reinforced concrete	m^3	55 000	43.5	2,400,000
	. steel shuttering	m^2	430 000	4.70	2,020,000
	. wood shuttering	m^2	100,000	9.35	935,500
	. reinforcement	t	4,300	380.--	1,630,000
	- Curtain				
	. grouting	m^2	68 500	14.--	960,000
	. drainage	m^2	3 500	6.--	20,000
	Administration				1,295,000
	Contingencies				6,370,500
	Total civil engineering				59,480,000

Equipment

- 5 spillway gates	t	490	3,040.--	1,489,600
- 3 intake gates	t	305	3,040.--	927,200
- 2 bottom outlets	t	85	3,040.--	258,400
- entry grilles	m^2	1,040	120.--	124,800
- screens	t	560	500.--	280,000
- stoplogs	m^2	300	400.--	120,000
- control equipment	total			90,000
				3,290,000
Administration				82,000
Contingencies				328,000
Total equipment				3,700,000

1.2. Power plant and switchyard

Civil engineering

- Excavations				
. in rock	m^3	37,975	3.05	115,800
. in soft ground	m^3	14,900	1.05	15,700
- Embankment	m^3	21,875	2.20	48,100
- reinforced concrete	m^3	23,300	43.50	1,013,600
- plane formwork	m^2	3,750	4.70	17,600
- curved formwork	m^2	2,200	9.35	20,600
- reinforcement	t	1,400	380.--	13,300
- built volume	m^3	38,200	50.--	1,910,000
				3,673,400
Administration				91,800
Contengencies				734,800
Total civil engineering				4,500,000

Equipment			
- turbines with cylindrical sleeve valves, speed governor, including installation	8	533,000.--	4,264,000
- cooling system, auxiliary set, stoplogs and portal hoist			366,000
- overhead crane			130,000
- generator, including reserve and installation	8	557,000.--	4,456,000
- single-phase transformator including reserve	13	56,000.--	728,000
- auxiliary services and command			2,768,000
- switchyard, including high and low voltage equipment			724,000
			13,436,000
Administration			336, 00
Contingencies			1,847,900
Total equipment			15,620,000

2. COST OF RECONSTRUCTION OF ROADS

2.1. Access to site

Main road			
- new road	km	70,000.--	910,000
- improvement of existing roads	km	55,000.--	3,030,000
- works			50,000
Administration and contingencies			610,000
Total			4,600,000

2.2. Road re-routing

Main roads				
Secondary roads				
- new roads	km	40	35,000.--	1,400,000
- improvements to existing roadways	km	102	25,000.--	2,550,000
- works		50	15,000.--	325,000
Carriage roads	km			750,000
Administration and contingencies				775,000
Total				5,800,000

2.3. Reconstruction of villages

- villages	3,758	195.--	733,000
Administration and contingencies			167,000
Total			900,000

3. DEVELOPMENT COST

3.1. Hydrology, geology, drillings, geotechnics, geometer works	1,900,000
3.2. General project and tendering	2,800,000
3.3. Execution plans	900,000
3.4. Consulting services	1,400,000
Total	7,000,000

4.	INTEREST DURING CONSTRUCTION	13,800,000
5.	COST OF PROJECT ADMINISTRATION	notional

<u>TOTAL INVESTMENT</u>	<u>115,400,000</u>

Table 7.2.II

MANANTALI PROJECT

Estimate of the investment required

Cost summary

	Alternative described		
	complete development 10^6 US \$	regulation alone 10^6 US \$	power plant alone 10^6 US \$
1. CONSTRUCTION COST OF DAM AND POWER PLANT			
<u>Dam and ancillary structures</u>			
- civil engineering	59.5	59.5	--
- equipment	3.7	3.7	--
<u>Power plant and switchyard</u>			
- civil engineering	4.5	--	4.5
- equipment	15.6		15.6
Total	83.3	63.2	20.1
2. CONSTRUCTION COST OF ROADS, RAILWAYS AND VILLAGES			
Site access roads	4.6	4.6	--
Re-routing of roads	5.8	5.8	--
Reconstruction of villages	0.9	0.9	--
Total	11.3	11.3	--
3. DEVELOPMENT COST	7.0	5.5	1.5
4. INTEREST DURING CONSTRUCTION	13.8	10.8	3.0
5. COST OF PROJECT ADMINISTRATION		notional	
<u>TOTAL INVESTMENT</u>	115.4	90.8	24.6

Table 7.2.III

MANANTALI PROJECT

INVESTMENT SCHEDULE

A: Project with flood control, complete equipment of the power plant and interest during construction

B: Project without flood control, postpassed equipment of the power plant without interest during construction

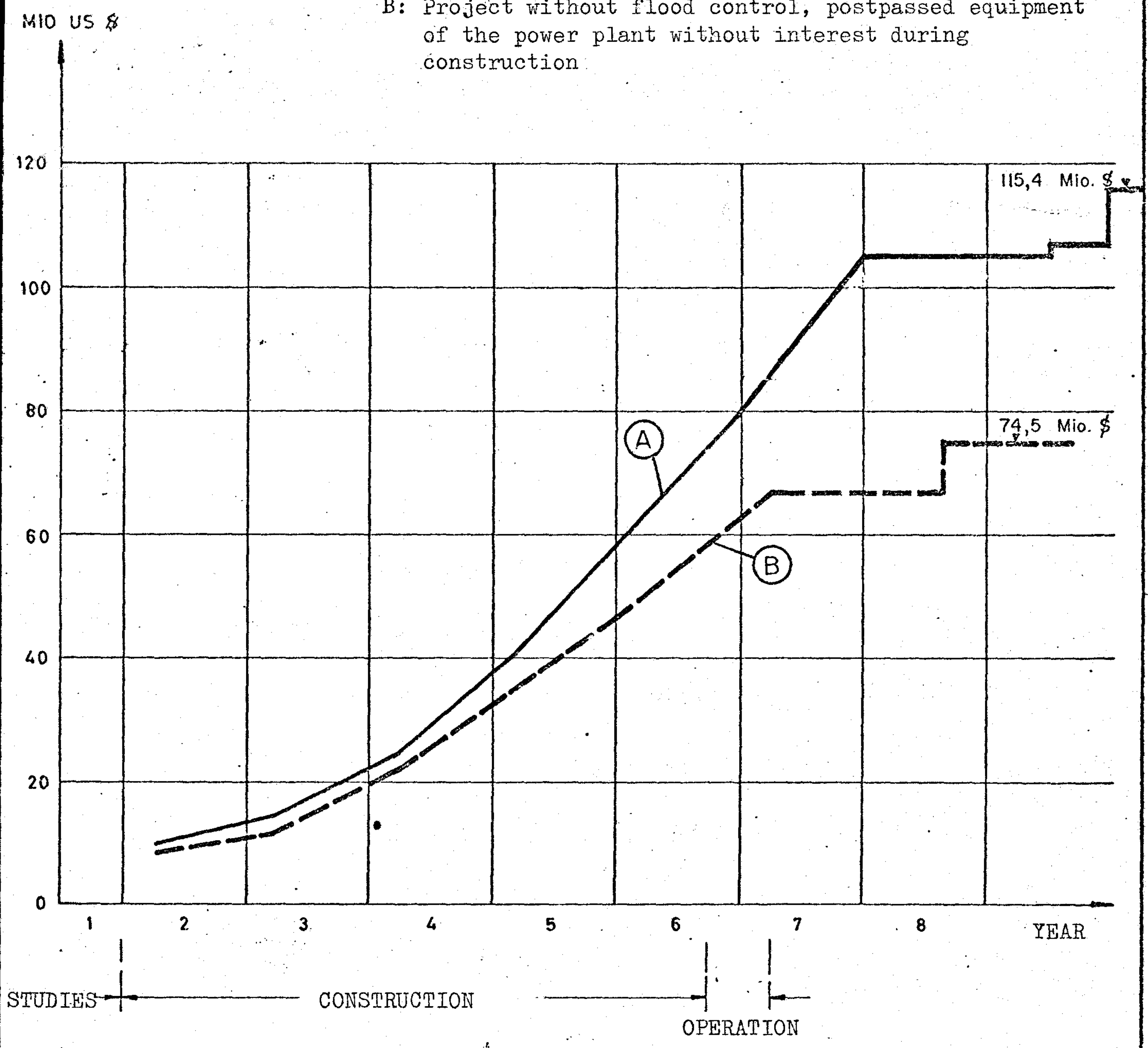


Table 7.2.IV

MANANTALI PROJECT

Estimate of annual charges

	Alternatives described		
	complete development US $	regulation alone US $	power plant alone US $
1. FINANCIAL CHARGES	6,355,000	4,635,000	1,720,000
2. REPLACEMENT CHARGES	152,000	49,000	203,000
3. OPERATING COSTS	315,000	45,000	270,000
4. MAINTENANCE COSTS	438,000	180,000	258,000
5. COST OF ADMINISTRATION		notional	
TOTAL	7,260,000	4,809,000	2,451,000

REPUBLIC OF MALI Table 7.2.V/a

CENSUS OF THE VILLAGES IN THE MANANTALI RESERVOIR AREA

No of plan 1 : 20000	Index and name of villages		Number of huts	Level
1	M 1	Soukoutali	85	167
	M 2	Serhoto	90	166
	M 3	Dialakoto	50	166
	M 4	Liliko	40	178
	M 5	Kéniékéniéko	150	167
	M 6	Souroufouga	35	167
	M 7	Kouroukondi	70	168
2	M 8	Barlakourou	125	177
	M 9	Tinntila	165	182
	M 10	Bamafélé	170	167
	M 11	Goumbalan	130	182
	M 12	Kouniakari	60	177
	M 13	Maréna	230	173
	M 14	Badioké	130	168
	M 15		10	168
	M 16		10	202
	M 17		3	193
	M 18		5	194
	M 19		7	217
	M 20		110	196
	M 21		20	207
	M 22		30	207
	M 23		15	213
	M 24		8	202
	M 25		12	192
	M 26		15	202
	M 27	Samantoutou	45	193
	M 28	Ougoundinnko	160	204
	M 29		33	202
	M 30		45	202
	M 31		12	193
3 N	M 32	Kéniéba	230	162
	M 33	Sandegnan	30	162
	M 34	Farabandi	130	182
	M 35	Nigui	125	185
	M 36		10	190
	M 37		8	186
	M 38		10	175
	M 39		40	180-200
3 S	M 40	Ganfan	117	183
	M 41	Tondidji	125	188
	M 42		10	166
		To be carried forward	2905	

REPUBLIC OF MALI Table 7.2.V/b

CENSUS OF THE VILLAGES IN THE MANANTALI RESERVOIR AREA

No of plan 1 : 20000	Index of name of villages		Number of huts	Level
		Report	2905	
3 S	M 43		25	168
	M 44		10	169
	M 45		5	192
	M 46		3	180
	M 47	Madina	55	179
	M 48		20	187
	M 49		13	190
	M 50		8	190
	M 51		30	203
	M 52		180	167
4 N	M 52a	Firia	160	202
	M 53	Nounkala	40	218
4 S	-	-	-	-
5 N	M 54		3	218
	M 55	Banbouta	80	177
	M 56		15	181
	M 57		3	181
	M 58		8	183
	M 59		8	167
	M 60		6	180
	M 61		8	173
	M 62	Kouroundi	35	180
	M 63		20	180
	M 64		45	190
	M 65		7	197
	M 66		35	207
	M 67		7	204
	M 68		18	225
	M 69		2	225
	M 70		25	217
5 S	M 71	Goungoudala	80	215
	M 72		20	213
	M 73		3	220
	M 74		15	230
	M 75		4	230
	M 76		25	220
	M 77		13	230
6 N	M 78		63	175
	M 79		5	170
	M 80		7	173
	M 81		10	180
		To be carried forward	4024	

REPUBLIC OF MALI Table 7.2.V/b

CENSUS OF THE VILLAGES IN THE MANANTALI RESERVOIR AREA

No of plan 1 : 20000	Index of name of villages		Number of huts	Level
		Report	2905	
3 S	M 43		25	168
	M 44		10	169
	M 45		5	192
	M 46		3	180
	M 47	Madina	55	179
	M 48		20	187
	M 49		13	190
	M 50		8	190
	M 51		30	203
	M 52		180	167
4 N	M 52a	Firia	160	202
	M 53	Nounkala	40	218
4 S	-	-	-	-
5 N	M 54		3	218
	M 55	Banbouta	80	177
	M 56		15	181
	M 57		3	181
	M 58		8	183
	M 59		8	167
	M 60		6	180
	M 61		8	173
	M 62	Kouroundi	35	180
	M 63		20	180
	M 64		45	190
	M 65		7	197
	M 66		35	207
	M 67		7	204
	M 68		18	225
	M 69		2	225
	M 70		25	217
5 S	M 71	Goungoudala	80	215
	M 72		20	213
	M 73		3	220
	M 74		15	230
	M 75		4	230
	M 76		25	220
	M 77		13	230
6 N	M 78		63	175
	M 79		5	170
	M 80		7	173
	M 81		10	180
		To be carried forward	4024	

REPUBLIC OF MALI Table 7.2.V/c

CENSUS OF THE VILLAGES IN THE MANANTALI RESERVOIR AREA

No of plan 1 : 20000	Index and name of villages		Number of huts	Level
		Report	4024	
6 N	M 82		4	177
6 S	M 83		30	215
7 N	-	-	-	-
7 S	M 84	Sitaféto	62	225
	M 85		9	220
	M 86		13	221
8 N	M 87	Diba	42	217
	M 88		4	213
	M 89		7	217
	M 90		13	217
8 S	M 91		3	217
	M 92		14	222
	M 93		10	216
	M 94		84	217
	M 95		10	217
	M 96		33	217
	M 97		26	222
	M 98	Sitaninnkoto	130	224
	M 99	Tiémoko	30	219
	M 100		9	220
	M 101		15	220-225
	M 102	Koba	70	228
	M 103	Kologo	170	230
9 N	M 104		21	222
	M 105	Bafé	62	226
	M 106	Souméoglo	65	227
	M 107		33	227
	M 108		35	228
	M 109		7	228
9 S	M 110		15	227
	M 111		10	230
	M 112		8	230
	M 113	Bafing-Makana	(90)	235
10	-	-	-	-
		Total (level 230 m.s.m.)	5068	
		Total (level 215 m.s.m.)	3758	

Chapter III

GOURBASSI SCHEME

7.3. GOURBASSI SCHEME

7.3.1. General

The chief morphological characteristic of the Falémé basin is the absence of a well-marked valley, making it difficult to find a site for a dam. The first aerial reconnaissances as well as a study of topographic maps have shown it to be possible to envisage the creation of a storage basin by building a dam across the river Falémé on the reach between the villages of Gourbassi and Farikounda.

On that part of its course, the Falémé forms the frontier between the republics of Senegal and Mali; the area which lends itself for the construction of a dam is situated about 240 km upstream from the confluence of the Falémé with the river Senegal.

An optimisation study, the results of which are given in Volume 5 of the present Report, has shown that optimum regulation of the river Senegal would be obtained with a discharge of 500 m3/s at Bakel. In order to achieve this objective, the most economic solution is to build a storage reservoir of 31.9 million m3 at Galougo. However, the Galougo scheme alone cannot satisfactorily protect the Lower Valley from floods and inundations. It is therefore necessary to provide another reservoir on the Falémé. In these circumstances, the economic optimum appears to consist in creating on the Falémé at Gourbassi a reservoir of 2100 million m3. This would have its normal storage level at an elevation of 94.0 m.a.s.l., and would make it possible to complete in a harmonious manner the regulation of the river Senegal.

The average annual discharge of the Falémé at the Gourbassi site amounts to 167 m3/s for a catchment area of 17100 km2. The minimum and maximum discharges observed during the period 1902-1968 amount respectively to 71m3/s and 264 m3/s. The characteristics of the Falémé floods which have been determined from a statistical analysis of the observed floods, are as follows: the millenial flood is 3400 m3/s, the centennial flood 2710 m3/s, and the decennial flood 2000 m3/s.

7.3.2. Topography and morphology

In the area envisaged for the construction of a dam, the Falémé meanders in a general north-westerly direction. The valley is not specially pronounced and this zone is characterised by the presence of numerous hills separated by peneplains.

By reason of the complex morphological character of the region, there exist a certain number of dam sites which from the viewpoint of topography are more or less equivalent. The first site envisaged was close to the village of

Farikounda in the elbow of the river. At that point a quartzitic hill forms a well-marked embankment on the right shore. By contrast, the left shore rises only very gradually and moreover, consists of a lateritic spur. Detailed reconnaissance of this site has also shown that the river bed is not rocky and that there is a risk of erosion at the foot of the dam.

A study of the topographic map showed there was a second site which at first glance appeared interesting. It is situated in the first elbow of the Falémé downstream from the village of Gourbassi. The first geological reconnaissance showed that it was possible to build an earth dam or rockfill dam, and a topographical survey was carried out. However, when a topographical map was drawn up to a scale of 1:2000, contrary to the indications of the 1:50 000 IGN map it showed that the height of the hills on the two banks was unsufficient for the construction of a storage dam. In the meantime, a first drilling campaign had also shown that the subsoil was worse than expected.

Analysis of the topographical maps of the storage area, produced by SENEGAL-CONSULT on a scale of 1:200 000 has shown that there exists a third site, between the two sites initially envisaged, on the reach of the Falémé which runs north.

A second drilling campaign undertaken during the winter of 1968-69 made it possible to finalize the siting of the storage dam; within the framework of the study described in the Interim Report of December 1968, this could only be done approximately. The configuration of the terrain which is characterised by the presence of numerous hills separated by gently sloping valleys, limits the maximum storage level to an elevation of 103 m.a.s.l. Above this level the secondary dams would become so large as to jeopardise the economy of the project.

The storage dam described in the report has a total length of 1510 m and a maximum height of 35 m. It requires the construction on the left bank of a secondary dam with a length of 600 m and a maximum height of 17 m above the natural level of the terrain.

7.3.3 Geology and tectonics

The rocks forming the basement of the Falémé basin are part of pre-Cambrian series more ancient than any others in the entire Senegal basin. These series it is assumed belong to the Birrimian era and are part of a metamorphic socle which is very strongly folded and eroded, while the basins of the other tributaries of the Senegal are situated in later series which have not undergone folding.

The geology of Gourbassi is therefore more complex than that of the other sites, and for that reason it has been the object of three geological investigations supported by numerous petrographical and cristallographical analyses and two drilling campaigns. The future stage of the project, moreover, will require still more detailed investigations accompanied by more extensive reconnaissance of the subsoil. The present report therefore does not pretend to solve all the problems presented by geology in the construction of a storage reservoir at Gourbassi.

No trace has been observed of any major landslides either in the area of the dam or in that of the reservoir.

7.3.3.1. Stratigraphy, lithology

The rocks which are present form part of a slightly metamorphic series (epizone to mesozone), generally of detritic origin (sandstones, pelites, jaspers and radiolarites), sometimes of volcanic origin (andesites, tuffs, or pillow-lavas) or of chemical origin (limestones or dolomites).

Pre-Cambrian orogeneses have produced strong folding of the series (direction N-S to NE-SE, dip 70° W to 70°). One also finds shear-folding (with a dip of 50°) as well as strong and deep weathering of the outcrops. All these factors render a reconnaissance of the terrain difficult.

In practice, the rocks have been classified according to their appearance under the microscope, in their in-situ condition, i.e. taking weathering into account. The following varieties have been distinguished:

1. Light-coloured rocks

   1.1. Strongly sericitic schists, often deeply weathered with a strong proportion of kaolinite (FA 2). In this class are included quartzites, greywackes, and breccias, all of which have a schistose structure and contain a high percentage of sericite. This variety is very widespread in the area under study.

   1.2. Slightly seriticized quartzites, with a brown-beige patina and containing numerous rhombohedra of dolomite. This variety is rare (FA 12).

   1.3. Limestone or dolomite with beige or blueish patina, a variety which has been observed only at one point in the area.

2. Rocks with deep patina

   2.1. Quartzite (radilarites), greywackes and jaspers. Coloured black, violet, or dark brown; massive texture. These rocks are very widespread in the zone under study.

2.2. Meta-andesite, agglomerates, and green-coloured chloritized tuffs; locally, dolerites or metamorphosed basalts of heavy texture.

In addition, one also finds numerous veins of quartz.

Lateral transition from one variety to another is extremely rapid as evidenced by a detailed profil in a direction along the river, downstream from the dam on the right bank.

Intensity and depth of weathering vary according to the nature of the rocks: weathering is more accentuated in schistose rocks (category 1.1) than in the massive rocks containing less of phyllitic minerals (categories 1.2 and 2.2). The maximum depth of weathering reached is 57 m (by borehole FA 5). Weathering shows itself at Gourbassi either by a kaolinisation, in the form of pure clay, or by a ferralitisation, in the form of lateritic crusts with the development of ferruginous concretions or linings.

7.3.3.2. Tectonics

In the Falémé basin, the Birrimian on the whole is very strongly worked and deformed by tectonic forces. The exact age of the formation of the Birrimian chain is not known, but it is certainly earlier than the pre-Cambrian quartzites of the Tambaoura cliffs which are superimposed on the Birrimian with a well-marked angular discordance.

In the area of the dam, the bedrock dips at an angle of 45 to 80°, in a direction ranging from north-south to north-east-south-west. The folds are rarely visible on the outcrops, but on the other hand, it is possible to recognize two pronounced cleavage systems on most of the soundest outcrops. Approximately 35 km south of the dam, i.e. in the area occupied by the reservoir, one can see on a limestone outcrop folds from two different ages. Each of these folds presents two distinct cleavages which run parallel with the axial plane of the respective fold.

These observations which have been corroborated by previous, geological reports, furnish evidence of intensive folding of the metamorphic series of the Falémé basin. There are morphological indications that some faults are present in the region under study. These are indicated on the geological map in their suspected positions.

7.3.3.3. Quality of bedrock

Because of the very heterogenous basement rock, the choice of the dam site epended principally on geological conditions. After examining numerous topo-raphically feasible variants, and carrying out an initial campaign of four boreholes (FA 1, FA 2, FA 3, and FA 5) in a zone which turned out to be unfavourable, the choice finally fell on a better area where a second campaign of three boreholes (FA 11, FA 12 and FA 14) has been undertaken.

In that area, the rock which crops out in the river bed can be considered to be of good quality; it consists of comparatively homogenous, dark quartzites, which should be sufficiently strong to support the weight of the concrete spillway structures. The dip of the strata and their orientation at right angles to the direction of flow in the reservoir reduce the risk of seepages.

On the right bank there are numerous outcrops. They consist of various quartzites, schists and meta-vulcanites which are of very heterogeneous composition. On the surface they are physically broken up, but do not appear to have weathered chemically in depth. Thus their geotechnical properties, i.e. their strength and especially their permeability change very rapidly, but can be considered satisfactory. It will be possible to site a rockfill dam there without fear of settlements. By contrast, the amount of settlement varies substantially depending upon the type of underlying rock.

On the left bank, the weathered cover is thicker, so that it has not been possible to be precise about the characteristics of the rocks to be encountered there. It may be assumed that they are similar to those on the right bank of which they would appear to form more or less an extension.

The strong heterogeneity of the strata, their heavy fissurisation and the deep weathering into clay are all factors which will have to be taken into account in detail when constructing the foundations and carrying out injections.

7.3.3.4. Construction materials

The alluvium along the thalweg and the clay from the weathering of the bedrock are both suitable for constructing the core of an earth or rockfill dam. A first estimate of the available quantities of material has been carried out following a reconnaissance to determine the boundaries of the area from which materials may be obtained. This has shown that approximately 3 km upstream from the dam there are 20 to 25 million m3 of silty clay, claybearing silt, and clay-bearing silt mixed with fine sand.

Materials for the rockfill can be excavated in sufficient quantity from the neighbouring hills which are formed either of broken rock or of lateritic crusts. By contrast, the rocky materials from excavating the foundations in the bed of the Falémé have to be reserved for the production of aggregates for making concrete. One can also utilize the outcrops of the basin where the bedrock is sound and free from phyllitic minerals, such as for instance the quartzites to the north-east of borehole FA 11. On no account must schists and rocks in process of weathering into clay be used as aggregates.

7.3.3.5. Storage Reservoir

Up to now, very little is known regarding the watertightness of the storage basin. It is possible that losses through percolation or slow underground seepages could occur on a relatively large scale. These underground leakages do not necessarily take place in a direction parallel to the present course of the river Falémé. One positive factor limiting the possible seepages is the high proportion of clay from the weathering which stops up the cavities caused by fractures.

7.3.3.6. Geotechnical prospecting

Four boreholes, FA 1, FA 2, FA 3, and FA 5, have been carried out in an initial prospecting campaign along the axis of the planned dam structure in the bend of the Falémé downstream from the village of Gourbassi. These boreholes represent a total length of 164 m. Seven intact core samples have been extracted from the clay.

Twenty-two Lugeon-type permeability tests have been carried out in the course of this reconnaissance. The nature of the terrain made it necessary to use a special sampling core drill with a sharp-edged casing.

Detailed results concerning the profiles are appended in the volume devoted to geotechnics.

The entire area is covered by a layer of laterite 3 to 6 m thick which is occasionally masked by a thin layer of silt.

Underneath the laterite is a varying thickness of banded clay, the product of the decomposition of underlying schists which themselves are weathered in places and get sounder with depth. The boundary between these two formations is moreover rather vague.

It has been found that the absorptions observed in the clays are on the whole larger than the absorptions found in the schists. This is due to the fact that the clays are easily ruptured by low pressure because of their low strength. These large absorptions are obtained after rupturing has occurred. The water level observed in boreholes FA 2 and FA 3 corresponds fairly well to the level of the river. As for borehole FA 5, which during drilling continually lost all its water, it was not possible to observe the level accurately because the borehole kept falling in.

The problems of the foundation cannot be resolved on the basis of the results from the above-mentioned first four boreholes. Moreover, the excellent quality of the construction materials did not justify carrying out geotechnical tests. For this reason it was decided to limit the geotechnical tests during the first campaign to those required for a study of the foundation problems, and not, as with the other dam sites, carry out tests to establish the characteristics of the construction materials.

Tests carried out on core samples of mixed material from the boreholes have shown a clay content of 10 to 15%. The clayey fraction (4 samples) was analyzed at the Mineralogical Institute of the University of Geneva. These analyses have shown the presence of kaolinite, illite, chlorite and quartz, all of which are minerals which do not swell in the presence of water. The liquidity limit of the analyzed material amounts to 40% and the plasticity limit to 25%. The Proctor standard test gives a density of 1.66 t/m3 with an optimum water content of 20%. The internal friction angle is about 29% and the permeability coefficient is of the order of magnitude of 10^{-7} cm/s. The pore pressure coefficient after Skempton is 10%.

Tests carried out with intact samples gave exactly the same results, except that a value of 15^{o} was obtained for the angle of internal friction, with a cohesion c' = 0.05 kg/cm2.

In the course of the second prospecting campaign which was undertaken during the winter of 1968-69, three boreholes (FA 11, FA 12, and FA 14) totalling 120.70 m in length, as well as 18 Lugeon-type permeability tests were carried out in the area of the definitive project.

Detailed results from these boreholes are shown onthe appended profiles in the volume devoted to geotechnics.

There is one feature which these boreholes have in common : this is the large mechanical resistance of the rocks which are encountered. It is, however, difficult to correlate the different boreholes on account of the heterogeneity of the rocks. In fact, in the case of borehole FA 11, almost the only strata traversed (under a cover of sand), were quartzites intersected by a bank of dolerite between +52 and +49 m; the quartzites were only slightly metamorphosed and rather compact.

Borehole FA 12 traversed only fissured light-coloured quartzite. Borehole FA 14 (on the left bank) encountered hard seritic schists alternating with black metamorphosed quartzites, which at the base were very much fissured.

On the right bank, except in the dolerites of borahole FA 11, the water absorptions were very low (less than 7 Lugeon units).

On the left bank, the water absorption which is at first very low (less than 5 Lugeon units) becomes very large with increasing depth (from 72 to 145 Lugeon units).

The observed water levels are nearly the same as the level of the river.

7.3.3.7. Conclusions

Despite geological conditions which at first sight seemed unfavourable, the two reconnaissance campaigns carried out at Gourbassi made it possible to select a satisfactory site for a storage dam.

7.3.4. General layout

A study of the numerous variants which are topographically possible, and an analysis of the results of the second drilling campaign carried out in the winter 1968-69 have shown that the most suitable site for a concrete dam would be in the river bed in the reach upstream from the island, i.e. 400 m upstream from the axis envisaged in the Interim Report. In that zone the rock crops out in the river bed and it appears to be possible to build a concrete dam at right angles to the line of the river.

The geological reconnaissance campaign had shown that the materials which make up the banks of the river Falémé are of adequate quality for building a rockfill dam. The height of the dam does not exceed 40 m and for this reason it is possible to envisage a homogeneous dam constructed of materials taken from deposits found in the vicinity of the village of Gourbassi. Comparison of a structure of this type with a rockfill dam has however shown that the latter solution is cheaper. In fact, the fine-grain materials are such that the dam profile can have only a very gentle talus, and horizontal drainage layers must be incorporated at different levels. Furthermore, the deposits from which such material is obtainable lie within a radius of 3 km while the rockfill can be obtained from quarries in the immediate proximity of the site.

The layout of storage dam studied in the present report is as follows:

- a concrete dam which incorporates the spillway and intakes is constructed in the river bed, and is flanked on either side by a rockfill dam.
- an ancillary dam, also built as a rockfill dam, closes off a saddle on the leftside of the valley.

7.3.5. Storage dam

The crown level of the dam has been fixed at 99 m.a.s.l., which takes into account a maximum additional rise in the water level of 1.50 m during a millenial flood a wave height of 1.50 m and a supplementary freebord of 2 m.

he characteristics of the structure described in the present chapter are the ollowing:

- storage volume	2.1 10^9 m3
- normal storage level	94.0 m.a.s.l.
- maximum storage level	95.5 m.a.s.l.
- crown elevation level	99.0 m.a.s.l.

ain dam

- sill height of spillway	84.3	m.a.s.l.
- maximum height of dam	35	m
- crown length	1 510	m
- length of concrete portion of dam	230	m
- length of rockfill portion of dam	1 280	m
- concrete volume	119 000	m3

Secondary dam

- maximum height of dam	21	m
- crown length	600	m
- total rockfill volume	1 489 000	m3
- total core and filter volume	371 000	m3

7.3.5.1. Rockfill dam

The dam section consists of two supporting shells of rockfill and a central watertight core. The waterface talus has a slope of 1:1.8 and the airface talus one of 1:1.7. Transition zones consisting of gravel of progressive granulometry separate the core from the upstream and downstream support bodies. Over the entire area of the dam, the cover of loose material will be removed and the rockfill support shells will rest either on the bedrock, or on the weathered rock if this should turn out to be of adequate quality, notably as regards settlement.

Equally it is not impossible that subsequent investigation may reveal that loose material need not be removed over the total dam area, and that in particular it may be possible to found the downstream support body on the in-situ material, while on the other hand the excavations for the core have to be carried down to the solid rock in order to make sure the structure is watertight.

The necessary rockfill will be won in quarries in the neighbouring hills, and in particular the hillside downstream from the site of the reservoir and on the right bank of the Falémé.

7.3.5.2. Concrete dam

The concrete portion of the dam is situated in the river bed. It consists of a 230 m long gravity dam rising to a height of 35 m above its foundations. Its profile will be of the classical kind, with a batter of 0.05 on the upstream wall face and one of 0.75 on the downstream face. Because the height is not great, a buttress dam would not offer any economic advantage. In fact, the saving in concrete which such a design would render possible, is relatively small and is balanced by the extra cost of the formwork. The dam is subdivided into blocks having a width of 14 to 16 m, depending upon the equipment housed in them. The joints between the concrete dam and the rockfill dams which flank it are made by means of spur walls penetrating into the body of the rockfill.

Stability calculations have shown that the structure is safe from sliding and overturning, and that the maximum foundation pressure does not exceed 15 kg/cm2, a value which is perfectly acceptable having regard to the geological conditions which are met with.

The dam rests on rock which crops out from the river bed as well as on the upstream part of the island, and only minor excavations will have to be done, except on the right bank where a certain quantity of loose material must be removed.

In order to channel the flow of water during the different construction stages, it is necessary to construct retaining walls in the shape of wings at right angles to the dam axis. It is the purpose of these walls to retain the foot of the rockfill, but they also make it possible to shorten the spurs which form the junction between the dam sections; for this reason they will not cause any increase in the construction cost.

7.3.6. Flood discharge installations

The scheme described in the present report was envisaged as forming part of a regulation of the river Senegal amounting to 500 m3/s at Bakel. Its contribution to this regulation is of the order 100 m3/s. This is the discharge at Gourbassi either from the hydroelectric power station or the bottom outlet. Because the storage reservoir is of modest size and it is impossible to control the Senegal floods solely by regulating the discharge of the Falémé, no provision has been made for creating artificial floods with the aid of the Gourbassi scheme. Accordingly, the sole purpose of the spillway is to form an outlet from the reservoir for flood water, and it consists solely of a crest spillway dimensioned to correspond to the millenial flood.

The spillway dimensions have been chosen so as to allow a discharge of 3400 m3/s when the Gourbassi storage reservoir is at its normal level of 94 m.a.s.l. The spillway comprises 4 openings each 10 m wide, fitted with gates having a total height of 9.7 m. The spillway sill is set at an elevation of 84.30 m.

Each opening is fitted with a segment gate having a height of 6.70 m, surmounted by a flap of 3 m height, which permits small quantities, i.e. up to 600 m3/s, to be discharged without opening the main gates.

During the passage of a millenial flood which reaches 3400 m3/s as it enters the storage reservoir, the level in the reservoir will only rise by 1.20 m above the normal storage level. This has been calculated on the basis of the flood hydrograph and a programme of opening the gates progressively in accordance with the rise in the reservoir level; the calculation takes into account the additional storage in the reservoir. The same analysis also demonstrates that the maximum spillway discharge during a millenial flood reaches 2280 m3/s at the maximum storage level attained.

Thusduring a millenial flood, the maximum discharge released at Gourbassi will be only about 15% greater than the present decennial flood. Heavy floods, up to the strength of a centennial flood, can therefore be reduced without difficulty to values corresponding to a present-day decennial flood.

A stilling basin has been provided downstream from the dam, with the object of destroying the greater part of the energy of the water before it is returned to the river.

7.3.7. Intakes and penstocks

There are three intakes feeding the sets with which the hydroelectric power is equipped. These intakes are situated in the right-hand part of the concrete dam, to the east of the portion assigned to the spillway and the bottom outlet. The intakes feed penstocks of 3 m diameter and measuring approximately 25 m in length to the turbine inlet. The penstock axes are spaced 14 m apart which corresponds to the spacing of the sets in the power station. Grilles are fitted to the inlet funnels, and each intake can be closed by a gate valve installed in the penstock and controlled from a valve chamber situated inside the dam. It is possible to operate the valves either from the power station or from the valve chamber itself, and they close automatically if the penstock ruptures, i.e. if the velocity of entry exceeds a preset value.

Lateral grooves in each inlet funnel allow the insertion of stoplogs during valve overhauls. Because of the low height of the dam, it is intended to place the stoplogs with the help of frogmen or divers.

7.3.8. Bottom outlet

The bottom outlet of the Gourbassi dam is sited in the central portion of the concrete dam, between the spillway and the intakes. It consists of a conduit 3 m in diameter which under a head corresponding to the minimum storage level in the Gourbassi reservoir gives a discharge equal to the design flow of the power station. This will make it possible to furnish the full amount of water to the river Senegal at Bakel, even if the power station is completely out of operation.

The bottom outlet conduit is shut off by 2 gate valves, the upstream one functioning as a safety device and standby to enable the service valve to be overhauled. The energy of the water discharged by the bottom outlet is dissipated in the spillway stilling basin.

7.3.9. Construction stages

The basic principle of the planned construction works consists in giving a normal and continuous flow of the work, even if in the course of the work, there should be floods of the order of 2300 m3/s, corresponding to a recurrence of once in 20 years. If, however, floods greater than this should take place the only consequence would be flooding of the working site, resulting in a certain delay, but without endangering the work already done. Hydrological study has shown that for a discharge of 2300 m3/s the water level reaches an elevation of 76 m.a.s.l. during the most critical construction stage.

In the first stage of construction, the river will be diverted into the left portion of its bed, and the whole of the concrete dam will be constructed in the shelter of temporary cofferdams.

The secondary dam as well as the sections of the main dam situated above an elevation of 78 m.a.s.l., can be carried out in this first stage without any special precautions. The dam on the right bank will be carried out up to an elevation of 78 m.a.s.l. during a dry period in this first stage, and above this level it can be completed without interference by the river discharge.

The second construction stage will consist in diverting the Falémé into the right portion of its bed, through a gap left in the concrete dam. The dam section in the river bed will be built during a dry season in the shelter of cofferdams of low height. Above an elevation of 79 m.a.s.l. the work can be brought to a conclusion independently of the discharge of the river.

In the third and final stage, the feeble flow of water during the dry season will be discharged through the bottom outlet, while the last three spillway blocks are being constructed.

7.3.10. Power station

The Gourbassi power station is situated on the right bank of the Falémé at the foot of the concrete dam. In plan, its position is bounded on one side by a wall separating it from the spillway, on the other side by the talus of the dam on the left bank.

The power house is of conventional design. The generator bay includes a workshop area. The hall is a steel structure, fitted with rails for a travelling crane. The lower portion of the walls is of masonry, above, the walls and roof are of corrugated aluminium sheet. Equipment for detanking the transformers is in the right-hand part of the power station where each of the transformers can be taken. This part of the station is at a higher level than the generator bay proper and includes a space reserved for offices, stores, and the control room.

The utilized gross head is comprised between the upstream water level which can vary during floods between elevations 85.0 m.a.s.l. and 94 m.a.s.l. and the restitution level downstream which can range from 66.8 m.a.s.l. to 75.3 m.a.s.l., depending on the discharge. The power station is fitted with 3 vertical Francis turbines designed for a total maximum design flow of 90 m3/s under the minimum head.

The main characteristics of the power station are as follows:

- number of sets	3
- normal turbine discharge	60 m3/s
- design turbine flow at minimum head	90 m3/s
- guaranteed output at switchyard terminals	13MW
- installed capacity at switchyard terminals	20MW

7.3.11. Mechanical equipment of power station

Each of the Francis turbines is coupled to a generator and has a steel intake spiral and cylindrical sleeve valve, placed between the staybars of the spiral and the distributor vanes. This valve is operated by oil-hydraulic cylinders and takes the place of a turbine inlet valve of classical design (butterfly or spherical). Each turbine is fitted with a speed governor.

Stoplogs together with portal hoists for handling them, are provided at the outlet of each draught tube.

The main operating data of each turbine are the following :

- net head	m	17.6	22	26	28
- discharge	m3/s	30	33	30	28.5
- output	MW	4.7	6.4	7.0	7.0
- speed	rev/min	187.5	187.5	187.5	187.5

The thrust bearing supporting the vertical load of each set is mounted underneath the generator, either on separate beams or on a pedestal forming part of the upper turbine casing. Dismantling of the turbine is carried out through the stator of the generator.

Because of the climate, cooling the machines presents a particular problem. Special arrangements will have to be made when the work is carried out. In the present state of the studies, a secondary clean-water circuit is being provided, with specially designed heat exchangers to facilitate cleaning. An auxiliary set is provided for supplying internal and local services (lighting, oil and drainage pumps, hoists, etc.). This will be a horizontal set driven by a water turbine. In this way the station will be completely independent of the grid. The auxiliary set is placed in the generator bay above the highest flood level.

7.3.12. Electrical equipment

The electrical equipment of the power station includes three vertical generators situated in the generator bay lower than the highest flood level. Each set has a rating of 8.2 MVA, and is cooled by watercooled airing closed-circuit ventilation system.

The three generators are connected through a medium-voltage busbar system equipped with circuit breakers, to a set of three single-phase transformers situated outside the power house on the upstream side. Each set of three single-phase units has a rating of 24.5 MVA.

The energy produced is transmitted at high voltage by a connection comprising three groups of three 110V overhead conductors. Each conductor is connected at the power station end to a single-phase transformer, at the other end it is anchored to the input portal of the 110 kV switchyard where it ends in an outdoor terminal box.

Overall efficiency of the electrical equipment working under nominal conditions is of the order of 96 %. In addition, the electrical equipment includes :

- the medium-voltage installation for connecting the internal station services to the local mains supply,

- the low-voltage distribution system for the internal station services,
- all installations for control, indicating, protection, and measurement,
- all telecommunication installations.

.3.13. Switchyard and h.t. lines

he transmission of energy to the centres of consumption takes place by means of 110 kV overhead transmission line which starts from a switchyard situated near he dam.

he 110kV switchyard receives, dispatches and distributes the energy of the ifferent starting points of the transmission line. It has two busbar sets with ixed phases.

he switchyard installations comprise the following elements :

- the high-voltage equipment (apparatus and busbar systems) required for the reception, dispatch, and transmission of energy.
- the metal structures together with their foundations, needed for carrying the electrical equipment.
- auxiliary equipment for monitoring, control and operation, and liaison with the power station; this is mostly situated in a service building.

e switchyard is sited in a flat plot of land, which is properly drained and qu ipped with roads and paths for the movement of material and personnel, and enc s.

7.3.14. Energy production

In the final stage of the Senegal regulation, the Gourbassi scheme will be capable of producing 104 GWh per annum with a guaranteed power output of 13MW available for 8000 hours per annum, during 9 years out of 10.

These output figures are the result of a simulation test undertaken with the aid of a programme worked out by SENEGAL-CONSULT which makes use of the hydrological data of 66 years of observation.

The mode of operation of the power station is described in Volume A1.

7.3.15. Access to site

Transport as far as the village of Mahina takes place by the railway from Dakar to Bamako. From there onwards, transport is by the existing road via Dialafara. It will however be necessary to improve this road over a length of 146 km, in order to permit all-the-year-round traffic of heavy convoys to supply the working site with equipment and materials. Starting from Koussili, an 8 km long spur road has to be built to gain access to the working site. The total distance between Mahina and the working site is 154 km.

7.3.16. Re-siting of villages

The storage reservoir which the Gourbassi dam will create will submerge an area of about 250 km2. A census has been carried out by SENEGAL-CONSULT on the basis of aerial photographs taken in 1967 for updating the 1 : 20 000 map of the scheme ' s storage basin. According to this census, it is expected that 22 villages will be submersed, representing about 1000 huts and a population, which on the basis of 2 persons per hut, can be estimated at 200 persons in accordance with the hypotheses stated in Volume 1 of the present Report.

These villages which are listed and their sizes given in an appendix, will be shifted either to the periphery of, or downstream or upstream from the storage reservoir. The distribution of these villages will depend on political, tribal, agricultural and piscicultural factors.

7.3.17. Displacement of roads

As regards trunk roads, a new road section with a length of 30 km will have to be constructed commencing at the Gourbassi dam, to re-establish the connection between Kayes and Saraya.

Furthermore, in estimating the cost of the road works, account has been taken of the construction of 50 km of cart tracks destined to connect the future villages established on the shores of the reservoir, with the road system.

The exact routes on these tracks will be decided according to the future development of the territory and the siting of the new villages.

7.3.18. Work schedule

The work schedule has been arranged in such a manner as to ensure the rational use of the site installations, i.e. so as to obtain a regular sequence of the individual work phases. Moreover, account is taken on the one hand, of the climatological requirements, and on the other, of the different modes of construction.

With the present access road to the Gourbassi site it is not possible to envisage large-scale construction work taking place on the Falémé requiring major transports. It will therefore be necessary to build a new access road to the dam site before construction of the Gourbassi dam begins. The construction programme of this road has not been analyzed in detail in the present study. It is in fact difficult to determine the best work sequence to adopt, as this depends on the possibilities of deploying the contractors' equipment. In order nevertheless to lay down a preliminary programme of expenditure, it seemed fair to assume that the construction of this access road could be carried out during the two years preceding the start of the construction work on the dam.

By contrast, the displacement of the population which at present lives in the villages which will be drowned when the storage reservoir is being flooded, will only occur a short while before the flooding. In fact, those villages which are at present established on the banks of the river depend on it, for it is the nearness of water which allows these populations to live.

Construction of the dam will begin at the start of the dry season, by installing the site equipment and constructing a temporary bridge across the Falémé. As soon as a system of site roads has been set up, and temporary cofferdams have been constructed for drying out the right-hand half of the bed of the Falémé, a start will be made with clearing away the top cover over the entire extent of the secondary dam. Excavation in that zone will only start when it has been dried out, in such a manner, as to permit the concreting of the dam foundations before the beginning of the Falémé floods. It is in fact preferable to have completed the concreting of the foundation before an exceptionally heavy flood occurs and drowns the working site. Concreting the dam and power house will take until the end of the second rainy season. By then, placing the rockfill of the secondary dam on the left bank, as well as of the main dam on the right bank, will both have been completed.

After the passage of the second year's floods, the first temporary cofferdam will be removed and a new cofferdam constructed for drying out the other half of the Falémé bed, on the left bank. This second cofferdam will have a comparatively modest height, as the rockfill for the dam on the left bank can be placed during the dry season. The temporary cofferdam therefore only needs to channel the water of the Falémé through the opening which has been left in the concrete dam. The height of the dam on the left bank is sufficient for the passage of the third year's flood. After this flood is over, the low water during the winter is discharged through the bottom outlet, and concreting the remaining three blocks of the dam is completed; the rockfill dam on the left bank is also finished.

Concreting the barrage and power station, as well as placing the rockfill of the dams, will be completed at the beginning of the fourth rainy season. It will then also be possible to start filling the reservoir, using a portion of the water from this first flood. Erection of the hydroelectric equipment of the power station can commence at the beginning of the third year's rainy season, i.e. 2.$\frac{1}{2}$ years from the start of construction. The duration of the erection work depends upon the rythm of commissioning the sets. This will conform as much as possible to energy requirements. Within the framework of the present study, the optimum case as regards the construction period has been assumed. The erection programme thus envisages the sets being put into service one after the other, in such a way that the last set will be ready to go into operation 4 months after the first has been put into operation at the same time as the storage reservoir was first being flooded.

7.3.19. Construction costs

The costs of construction of the dam and the hydroelectric power station of Bourbassi have been worked out on the basis of unit prices given in Volume 1A of the present report. These unit prices take into account the geographical position of the site, and the transport costs which are incurred by rail as far as Mahina and by road between Mahina and Gourbassi.

The cost of placing the rockfill of the dam takes into account the distance of the quarry areas. Improvement of the access road, as well as the rebuilding of secondary roads, and the shifting of villages which are to be rebuilt on the shores of the storage reservoir, have been the object of detailed studies, and the resulting costs have also been calculated on the basis of the unit prices given in Volume 1A of the present Report.

Table 7.2.I at the end of the present chapter summarizes the quantities in the main work categories, as well as the estimated cost of each quantity. The main heads of work, as well as miscellaneous and contingent work, have been regrouped under the different contracts which have been priced at the rates shown in Volume 1A. A second chapter is devoted to the cost of constructing roads and villages. A third chapter then gives an estimate of the cost of the studies and of finance during the course of the work, evaluated on the basis of the rates shown in Volume 1A. Taking into account the hypotheses made by SENEGAL-CONSULT concerning the unit prices per unit as well as the conditions of obtaining finance, the total investment required for carrying out the work described in this chapter amounts to 44.8 million US $.

7.3.20. Investment programme

The investment programme of investment is represented in Table 7.2.24. It has been set up on the basis of the work schedule described above, taking into account the lapse of 3 months between completion of work and payment being made for it, as well as the retention of guarantee sums which are kept back at the time of the provisional completion and released 24 months later.

7.3.21. Power production and generating cost

The Gourbassi scheme comes into the category of schemes whose main purpose is to contribute to the regulation of the discharges of the river Senegal. This scheme nevertheless makes it possible to generate a not inconsiderable quantity of base-load energy, amounting to a 130 million kWh, and available at the guaranteed power level of 13MW. The generating cost of the energy produced by Gourbassi has therefore been calculated from two different points of view, taking into account on the one hand the annual charges relating to the scheme as a whole and on the other, the annual charges relating only to those items of work connect-ed with energy production. The calculation of annual charges has been carried out on the basis of the rates shown in Volume 1A of the present Report.

Table 7.3.IV shown at the end of the present chapter, gives in detail the calculation of the annual charges. The generating cost has been calculated for the production of base-load, guaranteed during 8000 hours per annum, in 9 years out of 10. Taking into account the hypotheses presented previously, the generating cost of base-load energy produced at Gourbassi at the station terminals, works out as follows (1 mill = 0'001 US $) :

(a) on the basis of charges for the complete scheme : 26.5 mills

(b) on the basis of charges for the power station only : 7.0 mills

7.3.22. Present state of the project

As required by its mandate, SENEGAL-CONSULT has worked out a summary pre-project for the Gourbassi site, which enabled it to produce a preliminary estimate of the cost of the scheme, as well as to ensure, in combination with other schemes, an optimum regulation of the river Senegal under the most advantageous economic conditions.

It is clear, however, that this is only a preliminary study with the object of finding the most judicious combination of structures which can be built in the Upper Senegal basin. The construction costs which are mentioned are relative in character and are valid only under present-day economic conditions. In order to make it possible to carry out a study of the finance, it will thus be necessary to update the costs when the construction dates are known.

Table 7.3.I

GOURBASSI PROJECT

Normal reservoir level : 99 m.s.m.

Cost Estimate

		Unit	Quantity	Unit Price	Price
1.	CONSTRUCTION COST OF DAM AND POWER PLANT				
1.1.	Dam and ancillary structures			US $	US $
	Civil works				
	- Temporary diversion				400,000
	- Excavations				
	. in soft ground	m^3	290,000	1.05	304,500
	. in alterated rock	m^3	180,000	2.10	378,000
	. in rock	m^3	290,000	3.05	884,500
	- Dam embankment				
	. material from quarry	m^3	1,350,000	3.75	5,062,500
	. core	m^3	330,000	3.40	1,122,000
	. filter	m^3	280,000	4.95	1,386,000
	- Concrete, shuttering, reinforcement				
	. mass concrete		108,000	33.--	3,564,000
	. reinforced concrete	m^3	27,000	44.50	1,201,500
	. steel shuttering	m^2	32,000	4.70	150,400
	. wood shuttering	m^2	6,000	9.35	56,100
	. reinforcement	t	1,350	380.--	513,000
	- Curtain				
	. grouting	m^2	98,000	14.50	1,421,000
	. drainage	m^2	4,830	6.--	29,000
					15,472,500

			carried forward	15,472,500
Administration				411,500
Contingencies				3,078,000
Total civil engineering				18,962,000

<u>Equipment</u>

- 4 spillway gates	m^2	560	750.--	420,000
- 3 intake gates	t	45	3,040.--	136,800
- 1 bottom outlet	t	25	3,040.--	
- entry grilles	m^2	100	120.--	12,000
- screens	t	190	500.--	95,000
- stoplogs		150	400.--	60,000
- control system, iron work, light and command				60,000
				859,800
Administration				21,200
Contingencies				86,000
Total equipment				967,000

1.2. <u>Power plant and switchyard</u>

<u>Civil engineering</u>

- Excavations				
. in rock	m^3	6,500	3.05	19,800
. in soft ground	m^3	7,150	1.05	7,500
- Embankment	m^3	23,000	2.20	52,600
- reinforced concrete	m^3	4,900	44.50	218,000
- plane formwork	m^2	800	4.70	3,800
- curved formwork	m^2	300	9.35	2,800
- reinforcement	t	295	380.--	112,000
- built volume	m^3	11,800	50.--	590,000
				1,006,500

	carried forward			1,006,500
Administration				25,300
Contingencies				137,200
Total civil engineering				1,169,000

Equipment

- turbine with cylindrical sleeve valves, speed governor, including installation	p	3	44,500.--	1,333,500
- cooling system, auxiliary set, stoplogs and portal hoist				300,000
- overhead crane				46,000
- generator, including reserve and installation	p	3	260,000.--	780,000
- single-phase transformator including reserve	p	4	43,000.--	172,000
- auxiliary services and command				1,019,000
- switchyard, including high and low voltage equipment				145,000
				3,795,500
Administration				94,900
Contingencies				379,600
Total equipment				4,270,000

2. COST OF RECONSTRUCTION OF ROADS

2.1. Access to site

Main road				
- new road	km	8	70,000.--	560,000
- improvement of existing roads	km	146	55,000.--	8,030,000
- works				250,000
Administration and contingencies				1,330,000
Total				10,170,000

2.2. Road re-routing

Secondary roads				
- new roads	km	30	35,000.--	1,050,000
Carriage roads				
- new roads	km	50	15,000.--	750,000
Administration and contingencies				270,000
Total				2,070,000

2.3. Reconstruction of villages

- villages	case	963	195.--	188,000
Administration and contingencies				37,000
Total				225,000

3. DEVELOPMENT COST

3.1.	Hydrology, geology, drillings, geotechnics, geometer works	757,000
3.2.	General project and tendering	1,135,000
3.3.	Execution plans	376,000
3.4.	Consulting services	564,000
	Total	2,832,000

4. INTEREST DURING CONSTRUCTION

5. COST OF PROJECT ADMINISTRATION

notional

TOTAL INVESTMENT 44,813,000

Table 7.3.II

GOURBASSI PROJECT

Estimate of the investment required

Cost summary

	Alternative described		
	complete development 10^6 US $	regulation alone 10^6 US $	power plant alone 10^6 US $
1. CONSTRUCTION COST OF DAM AND POWER PLANT			
Dam and ancillary structures			
- civil engineering	19.0	19.0	--
- equipment	1.0	1.0	--
Power plant and switchyard			
- civil engineering	1.2	--	1.2
Total	25.4	20.0	5.4
2. CONSTRUCTION COST OF ROADS, RAILWAYS AND VILLAGES			
Site access roads	10.2	10.2	--
Re-routing of roads	2.1	2.1	--
Reconstruction of villages	0.2	0.2	--
Total	12.4	12.4	--
. DEVELOPMENT COST	2.8	2.4	0.4
INTEREST DURING CONSTRUCTION	4.2	3.6	0.6
COST OF PROJECT ADMINISTRATION		notional	
TOTAL INVESTMENT	44.8	38.4	6.4

Table 7.3.III

GOURBASSI PROJECT

INVESTMENT SCHEDULE

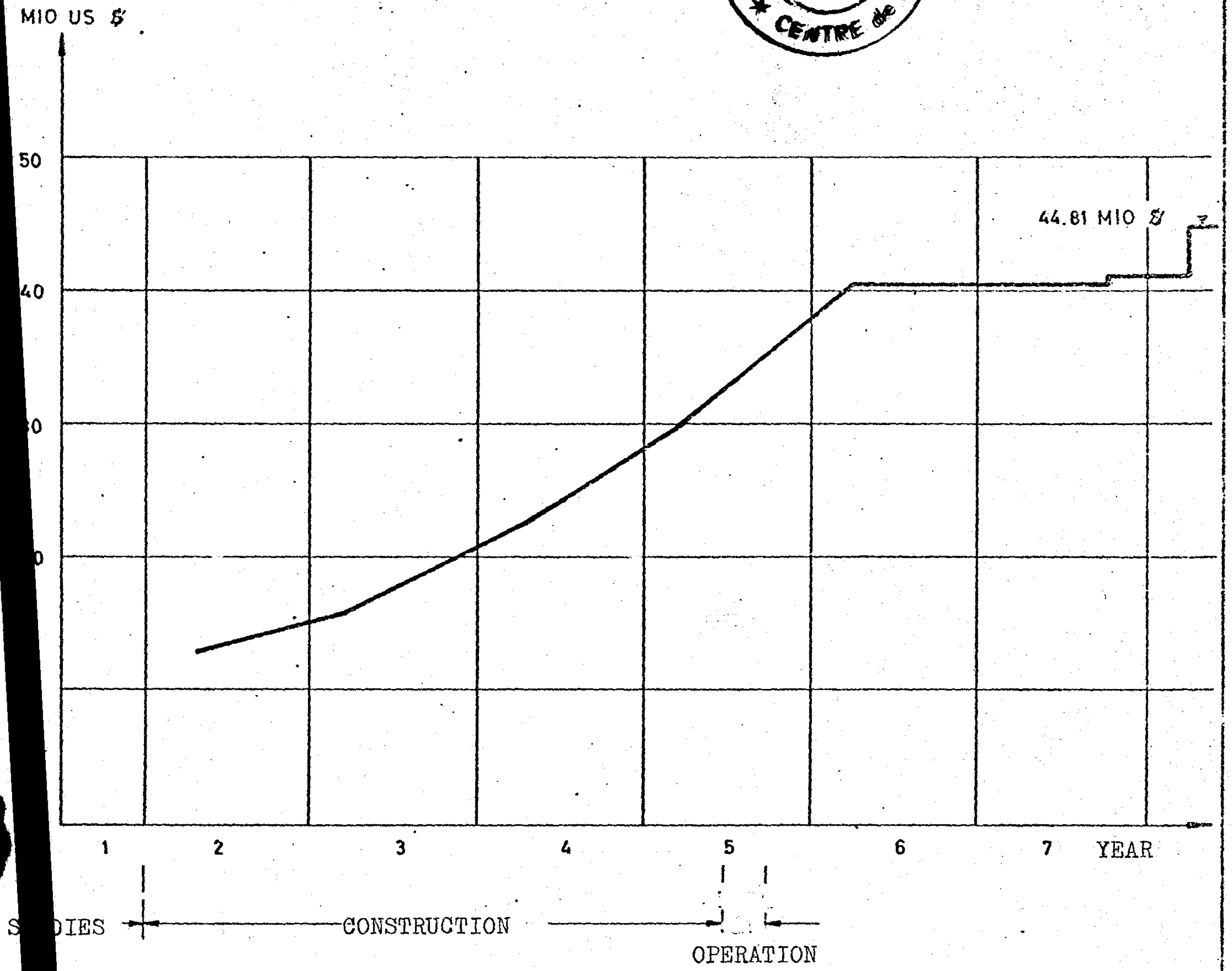


Table 7.3.IV

GOURBASSI PROJECT

Estimate of annual charges

	Alternatives described		
	complete development US $	regulation alone US $	power plant alone US $
1. FINANCIAL CHARGES	2,363,000	1,912,000	451,000
2. REPLACEMENT CHARGES	69,000	13,000	50,000
3. OPERATING COSTS	195,000	45,000	150,000
4. MAINTENANCE COSTS	135,000	65,000	70,000
5. COST OF ADMINISTRATION		notional	
TOTAL	2,762,000	2,035,000	727,000

Table 7.3.V

CENSUS OF THE VILLAGES IN THE GOURBASSI RESERVOIR AREA

No of plan 1 : 20 000	Index and names of villages		Number of huts	Level
1 N	G 1	Farabantourou	25	101
	G 2	Niakabana	22	82
1 S	G 3	Gourbassi	140	85
	G 4	Saïsoutou	150	82
	G 5	Santakoto	20	92
	G 6		9	87
	G 7		4	100
	G 8		4	98
	G 9	Kassagueri	30	101
2	G 15	Tinntiba		
	G 16	Linnguékoto	95	105
3 N	G 10	Samé	30	86
	G 11		17	87
	G 12	Bahé	20	82
	G 13	Kolonba		
	G 14	Kanfaranie		
3 S	G 17	Wassandara	96	85
	G 17a			105
	G 18		10	82
	G 19	Souroukoto	30	82
	G 20		20	80
	G 21	Kọlobo	30	85
	G 21a			92
	G 22		25	85
	G 23		80	94
	G 24		16	87
4	G 25	Koressinngui	10	110
	G 26		10	100
5	-	-	-	-
6 N	G 27	Moussala	55	92
	G 28	Kéniéko	60	95
	G 29	Lingogoto	30	91
	G 30	Ylimalo	25	94
	G 31	Daorala	100	102
	G 32		40	100
	G 33		18	91
6 S	G 34	Djidian-Kéniéba	120	107
	G 35	Frandi	50	103
7	G 36	Baboto	55	107
8	G 37	Banbadji	65	107
9 E	G 44	Métédia	11	102
9 O	G 38	Kolia	60	98
	G 39		20	103
	G 40	Mahina-Miné	55	101
	G 41		20	104
	G 42	Koundamé	60	100
10 E	G 43		40	107
10 O	-	-	-	-
		Isolated huts, approx.	60	
		Total (level230 m.s.m.)	1837	
		Total (level95.0 m.s.m.)	963	

Chapter IV

BOUREYA PROJECT

7.4. BOUREYA PROJECT

7.4.1. General

The Upper Bafing is comparatively regular in its hydrological character; at the frontier which separates Guinea from Mali, its flow in the average year reaches nearly 300 m3/s. Because of the large drop in level on Guinean territory, this river is thus bound to play a very considerable part in the country's energy policy.

As mentioned previously, preliminary studies have already been carried out in the region, in order to find the size of the hydroelectric potential of the Guinean basin of the Upper Bafing. While one could think of numerous schemes which could be constructed purely for power production, there are only very few sites suitable for major storage schemes which can play a part in the regulation of the river Senegal itself.

Reconnaissances carried out by SENEGAL-CONSULT showed that two sites were of special interest, since they appeared to have those characteristics which the regulation requires. These sites, going in an upstream direction, were Boureya and Koukoutamba. Preliminary studies undertaken by SENEGAL-CONSULT have been reported on together with the results obtained in an intermediate Report dated August 1969 devoted to the Upper Bafing.These studies have shown, however, that neither of these two sites could compete with the storage reservoirs on Mali territory, chiefly because of the paucity of the natural discharges. Nevertheless, the reservoirs on Guinean territory are capable of making a considerable difference especially as regards the production of electrical power in the final stage of the Senegal development.

At the request of SENEGAL-CONSULT, prospecting work both of a geological and topographical character, was carried out at the two sites envisaged, but could be completed only in part. In fact, on the orders of the Guinean government, the drilling reconnaissances had to be interrupted before the prospecting work at the Koukoutamba site had been completed and that at the Boureya site had even started. On the other hand, existing topographical maps to a scale of 1:200 000 gave indications which in the case of the Koukoutamba reservoir led to the preparation of a larger-scale map but covering an insufficient area.

The United Nations did, however, authorize SENEGAL-CONSULT in June 1970 to proceed with the preparation of a supplementary map of the Koukoutamba reservoir area, in order to make it possible to carry out a comparative analysis of these two sites which had been discovered on the Upper Bafing. These additional cartographic data have made it possible to confirm the conclusions of earlier studies. It became clear that in fact without a shadow of doubt neither of the two sites was capable of competing with the sites which had been studied in Mali, either at any intermediate or in the final stage of the development of the river.

Moreover, an optimisation study, the results of which are presented in Volume 5 of the present Report, has shown that from the point of view of energy production the two sites are practically equivalent.

The specific investment per unit of guaranteed power reaches minimum values which are practically identical for the two sites for a guaranteed power output of 85 MW. The generating cost of guaranteed energy does not allow therefore one site to be eliminated in favour of the other. It must be stressed, however, that this generating cost remains high, and that other sites in the Guinean Upper Bafing basin may have more favourable characteristics and allow energy to be produced more economically, although they may not have any large storage capacities.

It is therefore probable that the hydro-electric development of the Guinean Upper Bafing basin will commence with the construction of power stations along the course of the Tené or the Kiowa which are tributaries of the Bafing with large waterfalls. The construction of dams on the Bafing itself would then be carried out only after these more favourable possiblities had been developed.

In default of an absolute criterion of choice between the Boureya and Koukoutamba sites, SENEGAL-CONSULT finally gave preference to the former site for the following reasons, although none of them can be considered decisive:

- The volume which can be stored at Boureya is large and the cost per m3 stored is lower than at Koukoutamba.
- The discharge of the Bafing is greater at Boureya than at Koukoutamba.
- A reservoir at Boureya can therefore play a more important role, although still a modest one, than one at Koukoutamba, in connection with the regulation of the river Senegal at Bakel.

The Boureya project described in this chapter is based on the results of an optimisation calculation which was carried out on the assumption that the scheme is intended essentially for energy production. This scheme therefore has no particular role to play in connection with flood protection, and it is unnecessary to reserve an additional slice of storage volume for flood control.

The Boureya dam will make it possible to obtain a total storage of 4900 million m3 at the normal storage level of 381 m a.s.l.. The maximum height of the dam above its foundations is 64.0 m (crown height 386 m a.s.l.).

At the Boureya site, the mean annual discharge of the Bafing amounts to 237 m3/s, the mean annual discharge in a dry year being of the order of 150 m3/s. Estimates for the millenial flood give a value of 4550 m3/s, the centennial flood amounts to 3650 m3/s, and that of the decennial flood to 2600 m3/s. The catchment area is 14 860 km2.

7.4.2. Topography and morphology

In the area of the dam, the Bafing flows in an easterly direction, having originally flowed northwards as far as a point 2 km upstream from the site, and resuming its northerly direction 3 km downstream from it. The bedrock over which the river flows consists of gneiss and granite, and forms a succession of low sills giving rise to small rapids. Upstream from the Boureya site the Bafing follows the western slope of a hill forming part of a mountain chain, the Fello Boureya. At the site selected, it traverses this chain at right angles in a gorge which is a good place for building a barrage. Downstream from the site, the valley broadens and the hillsides slope more gently.

The right bank is thus formed by the slopes of the Fello Boureya. These hillside are rounded, which is characteristic of granitic rock. The decay products of the granite have accumulated by soil-creep at the foot of the slopes in the form of sand.

On the left bank, lateritic terraces descend successively towards the river.

The river bed consists of sandy alluvium, and in the vicinity of the selected site its maximum width is 750 m; isolated monadnocks some 20 m high dominate the plain; these have originated from the erosion of lateritic plateaux left intact by the river.

Along the axis of the projected dam, the following morphological picture presents itself:

- The bottom is at an altitude of 340 m and has a width of 150 m.
- The hills on the right bank rise up to a height of 410 m with a slope of 40 %, then drop to 390 m in a slight depression where the rock crops out, only to rise again progressively to more than 420 m. In the south, the hills are generally higher, except in the region of Afia, about 10 km from the site.
- On the left bank, a plain of lateritic soil extends at an altitude of 345 m up to a distance of 600 m north of the river. This is followed by a plateau of encrusted laterites which rises gently from 410 to 450 m altitude; to the north, there is a marked steep rise of the terrain to a glacis situated at an altitude of more than 600 m.

Such a morphology, comprising hills of soft contours separated by small saddles presents disadvantages for the construction of a storage scheme.

7.4.3. Geology and geotechnics

7.4.3.1. Surface Geology

On opposite sides, the region's boundaries are defined, to the west and north, by its contact with the Lower Cambrian massif of the Fouta Djallon, and to the south and east, with the pre-Cambrian socle of Lower Guinea. These two geological entities are traversed by large doleritic intrusions of post-Cambrian age. A strong lateritisation which extends throughout the region, masks the rock which can only be seen in a few places; it is however much more marked on the left bank (Fouta Djallon massif) than on the right bank where rock outcrops have been uncovered by erosion far more frequently.

The area of the dam is situated in a granitic-gneissose series forming a link in the Fello Boureya chain and belonging to the pre-Cambrian socle. This chain is bounded to the east and south by Lower Cambrian sandstones which can be seen about 10 km south of the dam and 4 km to the east. At a point 2 km north of the dam, the metamorphic series disappears beneath the laterite which probably covers Lower Cambrian sediments. A large doleritic dyke runs from north to south along the eastern shoulder of the granitic-gneissose hill.

7.4.3.2. Stratigraphy and lithology

In the vicinity of the dam three kinds of rock may be distinguished, i.e.:

- fine-grained granites,
- coarse-grained granites in conjunction with gneiss,
  and
- dolerites.

Subject to more detailed studies later, it seems that the fine-grained granite was formed during a second phase of granitisation, after the coarse-grained granite had been laid down in the Birrimian socle.

A fine-grained granite is found in the western flank of the hill on the right bank, as well as in the river bed where it is clearly visible. It can readily be recognized by its rose colour and black patina, and is a calcium-alkaline granite with microcline, quartz, plagioclase and biotite. In this area, coarse-grained granites and gneisses are very frequently found in association; they are differentiated from each other only by their texture. This is why for simplicity's sake, they have been mapped in the same category. Very widely distributed on both sides of the river, they form the largest portion of the Fello Boureya. There is great similarity in their composition: they consist of quartz, plagioclases, and potash feldspars, much biotite and in addition a little muscovite; and they can be sericitized or chloritized. Their structure is very coarse occasionally pegmatitic in the case of the granites, and granoblastic in that of the gneisses. They are traversed by aplitic veins which follow the Birrimian direction (see 5.3.3.3.) and are several decimetres thick, or by predominantly dark-coloured (melanocratic) veins from the dolerite dyke.

There are still other metamorphic rocks belonging to the pre-Cambrian socle of which one finds some isolated for outliers, but these do not directly affect the dam as far as one can see at the present stage of the reconnaissances.

The dolerite dyke extendsover a width of more than 200 m downstream from the dam. It consists of a dark-coloured dolerite with black patina, and is fine-grained with an ophitic texture. The minerals are mainly plagioclases and slightly chloritized augite.

7.4.3.3. Tectonics

The absolute age of the period when the varions geological entities were laid down is is unknown at present; however, the alignment (N to NNW) of the existing features shows a predominance of the Birrimian direction.

The granitic-gneissose socle is certainly syntectonic; the fold-axis, banding and schistosity follow a north-northwesterly direction. Fracture-cleavage in general presents itself in the form of two symmetrically oriented systems at 40° - 60° to the Birrimian direction.

The fine-grained pink granite is more homogeneous and does not achelit any oriented texture. It is probably post-tectonic, and appears to be less fractured than the socle.

As regards the doleritic dyke, this too follows a meridional direction, which can be explained by the intrusion of the dyke into a Birrimian fissure having occurred during the bulging upwards of the Fouta Djallon. The dolerites are fractured along two principal systems which are fendered very clearly visible by the action of superficial weathering.

7.4.3.4. Quality of bedrock

On the right bank and in the river bed, the bedrock is of suitable strength and stability for a heavy structure to be sited there; this is not so on the left bank where the substratum consists of soils and alluvia on which concrete structures must not be founded.

The contact line between the granite-gneiss and the fine-grained granite runs from north to south, in a direction tangential to the long axis of the barrage, which will therefore be founded astride these two petrographic entities. This should not cause any special difficulties, the contact being - according to the initial theories about the terrain - of a petrographic and not a tectonic nature.

In general, the geological units have been strongly folded in the course of the Birrimian orogenesis which thus worsened the quality of the existing rocks. The principal direction of the fractures is tangential to the axis of the barrage, while the secondary fractures run obliquely to it. In the river bed, erosion has exposed the network of fractures which is thus more clearly visible than on the rocky slopes. However, no major fracture zone has been discovered on the site.

The granites and gneisses are strong and impermeable rocks when fresh which is generally the case in the river bed. On the right slope, the outcrops show a certain degree of weathering (kaolinisation of the feldspars and oxydation of the micas). Their machanical disintegration, into granitic sand, does not appear to be excessive; one should bear in mind, however, that in zones of weakness such disintegration can continue down into the depths notably in depressions and along fractures. The granitic sand forms poor, beige-coloured soils, of sandy granulometry and giving poor agricultural yields.

The dolerites are very strong and impermeable when they are not too rund fractured. Their chemical alteration generally does not go very deep. The soils which are formed are red in colour, of the consistency of clays, and give a good agricultural yield.

7.4.3.5. Reservoir

The basin traverses geological strata which are generally considered to be impermeable; however, there are certain regions, notably where granites have become strongly weathered and are not covered by a lateritic veneer; this is where considerable losses could occur.

Another source of seepages could be depressions which separate the hills from the basin. The largest depression is near Afia, 10 km south of the dam; the rock basement there consists of quartzites partially covered by laterite. This depression is situated at an elevation of approximately 412 m. It may be work will have to be carried out there to make the reservoir watertight, if later investigations should show that seepages could occur and if the storage level were to be higher than envisaged in the present report.

7.4.3.6. Construction materials

The dolerites form an excellent material whether for the rockfill of a dam or as aggregates for concrete. Superficial fracturing helps with the quarrying without lowering their geotechnical properties, as the chemical changes do not penetrate deeply. The dolerites can be won from quarries situated downstream from the dam on both river banks; except that the right bank must not be stripped in the vicinity of the dam for fear of excessively dislocating the body of rock downstream from the structure.

The fine-grained granites are also suitable for making concrete, but the granite-gneisses contain too much mica (20 %).

Fill materials for the supporting body of the dam exist in great variety and abundance throughout the region (dolerite, granite, gneiss, and even lateritic concretions).

Laterite concretions can be utilized for making a watertight core. They exist in large quantities in the plain and on the left-bank slopes of the Bafing.

According to a laboratory classification carried out on 9 samples at the Federal Polytechnic in Zurich, the soils on the left bank are favourable for forming the core. Samples were taken from three pits; the next section deals with their geotechnical properties.

7.4.3.7. Geotechnical prospecting

No rotary drilling was carried out at Boureya, but four pits were dug at the request of SENEGAL-CONSULT, by inhabitants of the neighbouring villages, in order to reconnoitre the subsoil of the plain on the left bank. These pits were respectively 1.50 m, 2m, 3.8 m and 4 m deep, and did not reach the bedrock, not even weathered bedrock.

Three of them traversed concretions of lateritic soil. Situated 300 m downstream from the dam axis, and 230 m from the river, the fourth pit encountered alluvia of sand and clay saturated by the water table. In February 1969 the level of the water table was at an elevation of 342 m, i.e. 2 m above the level of the river.

The tests carried out on 9 samples from the trial pits taken by hand at different depths, showed a clay content varying between 10 and 25 %. The liquidity limit of the material analyzed ranged from 25 to 40 %, and the plasticity limit varied between 10 and 18 %. It therefore appears that the properties of this material vary depending upon the site and depth of the pont from which the samples are taken. Samples of material which can be considered suitable for constructing a watertight core have been mixed and a standard Proctor compaction test carried out on this mixture gave a density of 1.88 t/m3 at an optimum water content of 13 %.

The material has an internal friction angle of 30°. Detailed results of the analysis are given in Volume 9 of the present Report; they show that the material is suitable for constructing a dam core.

7.4.4. General layout

The choice of the site of the storage dam, was dictated principally by topographical condition. In fact, it was a matter of finding which alignment of the dam axis would give the minimum volume of fill. Geological conditions, in particular the existence of a thick lateritic cover on the left bank, immediately ruled out any solution employing a concrete dam only. On the other hand, the magnitude of the floods which have to be discharged while the work is in progress, made it prohibitive to construct tunnels for this purpose. This is why the present solution imposed itself. This provides a concrete dam across the river bed and on the right bank, completing it by a rockfill dam on the left bank. It is possible to construct the concrete dam in two stages, by diverting the river during the first stage into one half of its bed, while discharging it during the second stage through openings left in the already constructed portion. The dam has to be supplemented on the right bank by a small secondary rockfill dam.

7.4.5. Dam

The crown level has been fixed et 386.0 m a.s.l.. This takes into account an additional rise in the water level of 1.50 m, and an extra height of dam of 2 m more. The dimensions and data of the intermediate variant shown on the drawings appended to the present Report are as follows:

- normal storage level — 381 m a.s.l.
- minimum storage level — 357 m a.s.l.
- live storage volume — 4.1×10^9 m3
- crown level — 386 m a.s.l.
- maximum height of dam above foundations — 64 m

- width of valley at crown level	
(a) main valley	1 300 m
(b) side valley	250 m
- length of concrete dam	500 m
- length of rockfill dams	
(a) main dam	960 m
(b) secondary dam	260 m
- volume of concrete dam	835 000 m3
- volume of rockfill dams	6 320 000 m3
- area of grout screen	53 000 m2

The cross-section of the main rockfill dam, as well as that of the secondary rockfill dam on the right bank, consists of three separate zones. These three zones are going from the upstream to the downstream side, a relatively pervious support shell, an impervious core, and lastly again a relatively pervious support shell. The transition between the support shells and the core is ensured by special transition layers which act as filters.

The core will be constructed of laterite concretions, but its dimensions cannot definitely be decided except on the basis of information obtained by carrying out additional reconnaissance in those areas where a study of the surface geology has located likely suitable materials. The upstream and downstream support shells can be constructed either from materials obtained from quarries or equally from materials of lateritic origin. It will be a matter of finding the most economic solution on the basis of further reconnaissances, and after having determined the size of the available deposits.

In the absence of sufficiently precise data, it appeared to be reasonable to choose a gradient of 1:2 for the slopes of the dam. This figure is based on the results of tests carried out as part of the studies of other dams built of lateritic materials. If quarry rock were to be used for the support bodies, the slopes could be increased, which would reduce the volume of the dam. Any gain from such a reduction would however in all probability be cancelled out by the greater cost of extracting the material.

The dam axis curves towards the upstream side, and the crown width has been fixed at 10 m. The core is anchored in the laterites and alluvia at a depth of some 10 m. Additional prospecting will probably reduce this depth, but it seems best to draw up the project on this basis because of the very variable characteristics of the laterites found in the trial pits. The watertightness of the subsoil under the core has been ensured by a grout curtain which will be extended beyond the lateritic or alluvial cover into the bedrock.

The concrete portion of the dam is situated in the river bed and on the right bank. It consists of a buttress dam with a crown length of 500 m and a maximum height of 64 m above the foundations. This structure is made up of 20 buttress elements each 22 m wide, and 4 elements of 16 m width housing the intake penstocks leading to the hydroelectric power station at the foot of the dam.

The standard elements have a 22 m wide massive waterface, and two 5 m thick buttresses spaced 7 m apart. The elements housing the intake penstocks are of the same type, and consist of a massive waterface 16 m wide and two buttresses each 4 m wide and spaced 4 m apart. The batter of the upstream face is 0,5 and that of the downstream face of the buttresses varies from 0.2 at the top section to 0.45 in the central and 0.67 in the lower portion. The elements are separated from one another by joints fitted with seals. Where as in the central zone the gaps between the buttresses are open on the downstream side, in the transition portion from the concrete dam to the rockfill dam they are covered by prefabricated slabs of reinforced concrete. The ends of the support bodies of the rockfill dam rest against these slabs. The gaps between the buttresses are also closed in those elements housing the spillway so that floodwater can pass over.

Over the entire working site, the sound rock must be reached by cutting away the top cover to a depth of 4-5 m. The waterfaces of the elements are extended downwards by an 8 - 10 m deep foundation wall containing a gallery from which contact grouting and the grout screen will be carried out.

7.4.6. Spillway

The spillway is sited on the left of the concrete dam. This layout conforms with the local topographical conditions. The only purpose served by the spillway is to let floods out of the reservoir.

The spillway comprises four openings each 19 m wide, with a sill 7.5. m lower than the normal storage level in the reservoir. Each opening is closed by a segment gate. If one of the gates becomes blocked in the closed position during a millenial flood, the capacity of the remaining three openings is 300 m3/s with an additional rise in the reservoir level by 1.5 m above the 381 m level, taking into account the effect of the additional storage volume. Such a discharge corresponds to that which now occurs downstream from the barrage during a 20-years flood.

A stilling basin has been placed at the foot of the dam in order to obviate the danger of erosion.

7.4.7. Water intakes and pressure penstocks

The four water intakes feeding the sets installed in the hydroelectric power station are housed in dam elements situated in the right-hand half of the Bafing. Their sill is at an elevation of 338 m a.s.l. and their axis is the same as that of the element which houses it. Each intake funnel is fitted with a grille and has grooves enabling a stoplog to be fitted if necessary. The penstocks have an internal diameter of 3.70 m and are spaced 16 m apart; they are housed between the buttresses downstream from the dam.

7.4.8. Bottom outlet

The bottom outlets are arranged inside the centre elements of the spillway. They comprise two conduits of 3 m diameter which can discharge a flow of 280 m3/s under a head corresponding to the minimum storage level. Each bottom outlet is fitted with two gate valves, one of which acts as a safety device and standby.

During the final stage of construction, as will be shown below, the bottom outlets will serve to divert the river temporarily.

7.4.9. Power station

The Boureya power station is situated on the right bank of the Bafing, at the foot of the concrete dam. Its situation in plan is between the spillway wall on the right bank and the natural talus of the river.

The power house is of semi-outdoor type with an external portal crane for erection work. At its end facing the right bank of the river, is a building comprising the control room, offices stores, and a workshop which is fully enclosed but gives complete access to the crane. The workshop also has facilities for de-tanking transformers, where every transformer can be taken. The gross head is the difference between the upstream storage level at an elevation of 381 m and the downstream restitution level which can vary between 326 and 334 m a.s.l. depending on the discharge.

The power station is equipped with vertical Kaplan turbines and has a maximum design turbine flow of 410 m3/s at minimum head.

The main data of the power station are the following:

- number of sets		5
- normal turbine flow	:	273 m3/s
- rated flow under minimum head	:	410 m3/s
- guaranteed output, at switchyard terminals	:	85 MW
- rated output of installation at switchyard terminals	:	130 MW

7.4.10. Mechanical equipment of power station

Each Kaplan turbine is coupled to a generator and consists of a steel intake spiral and a cylindrical sleeve-valve placed between the stay bars of the spiral and the distributor vanes.

This valve is operated by hydraulic oil cylinders, and takes the place of a turbine inlet valve of classical design (butterfly). Each turbine is fitted with a speed governor with a drum cam actuating the distributor ring which is controlled as a function of the head. Stoplogs with manoeuvring hoists are provided at the outlet of each draught tube.

The main operating data of each turbine are as follows:

- net head in m	:	27	41	51
- discharge in m3/s	:	82	82	82
- output in MW	:	17,9	29,8	37,8
- speed in rev/min	:	214,3	214,3	214,3

The thrust bearing carrying the vertical loads of each of the sets is mounted underneath the generator on a pedestal which forms part of the upper turbine casing. The turbine can be dismantled through the generator stator.

An auxiliary set is provided for supplying the internal and local services (lighting, oil and drainage pumps, hoists, etc.). This horizontal waterturbine-driven set renders the station completely independent of the local mains supply. It is situated in the turbine bay above the highest flood water level.

7.4.11. Electrical equipment of the power station

The electrical equipment of the power station comprises 5 vertical generators installed in the turbine hall, each with a rating of 48 MVA at 214.3 rev/min, and equipped with closed-circuit ventilation, the air being watercooled.

Each of the generators is connected by a medium-voltage busbar system to a set of three single-phase transformers which stand in line outside the power house on the downstream side. Each set of three single-phase units has a rating of 48 MVA.

The energy produced is transmitted at high voltage by five connections each consisting of three 220 kV cables and placed in a tunnel leading to the switchyard. Each cable is connected at the station end to a single-phase transformer, and at the other end to an outdoor-type terminal box.

The overall efficiency of the electrical equipment of the sets operating under nominal conditions is of the order of 96 %.

The electrical equipment comprises in addition:

- a medium-voltage installation for connecting the internal station services to the local mains supply.
- a low-voltage distribution installation for the supplying the internal services of the scheme.
- the entire system of control, indicating, protection, and measurement installations.
- the entire telecommunication installations.
- the entire installations for lighting and earthing.

7.4.12. Switchyard and h.t. lines

Transmission of energy to the centres of consumption is carried out by a 220 kV high-tension overhead line which starts from a switchyard near the dam.

This 220 kV switchyard receives, dispatches, and distributes energy to the starting points of the line. It comprises two busbar systems with mixed phases.

The switchyard installations consist of the following items:

- the high-tension equipment (apparatus and busbar systems) required for the reception, dispatching, and transmission of the energy.
- the steel structures together with their foundations on which the electrical equipment is mounted.
- auxiliary equipment for monitoring, controlling and operating the installation and for maintaining liaison with the power station; these items of equipment are mostly installed in a service building.

The switchyard is constructed on a flat plot of land which is appropriately drained, equipped with the necessary roads and paths for the movement of material and personnel, and fenced.

7.4.13. Stages of construction

The basic principle underlying the construction schedule is to make it possible for the work to proceed normally and continuously through the various stages of construction, even if floods of the order of 3000 m3/s should occur, such as are likely to happen once in 20 years. However, should there be floods in excess of this magnitude, they would only cause a certain delay by flooding the work site, but would not damage either the work already completed or partially completed, nor the site installations.

An analysis of the discharge conditions has shown that with a flood of the order of 3000 m3/s, during the most critical stage of the construction, a water level would be reached corresponding approximately to an elevation of 348 m a.s.l. The height of the retaining wall which protects the foot of the dam on the left bank has been based on this elevation.

The portion of the dam which is situated above an elevation of 350 m a.s.l. can be constructed in the first stage without any particular precautions as far as inundations are concerned. The same applies to the concrete portion of the dam on the right bank, as far as it lies above this level, as well as to the secondary rockfill dam on the right bank. An appropriately sited temporary cofferdam will make it possible to isolate and dry out the left half of the river. A start can then be made with the necessary excavations and the concreting of the right-bank retaining wall, the stilling basin, and the dam foundations up to the level of the bottom outlets.

During this first stage, the river will discharge through the right-hand half of its bed. The second stage which will commence during a dry season, consists in demolishing in the first instance the temporary enclosure, and then channelling the watercourse over the dam foundations which had been constructed during the first stage. A new temporary cofferdam is then built, making it possible to construct the hydroelectric power station and the last section of the concrete dam. During the final stage which also takes place during the dry season, the spillway is concreted while the low-water discharge of the river is diverted through the openings of the bottom outlet.

7.4.14. Access to site

Construction materials as well as equipment will be transported from Conakry by the railway which connects this town with Kankan. The transports go by rail as far as Dabola, from there a wellmaintained road passing through Dingueraye leads to Boné, a distance of 138 km from Dabola. The existing track which connects Boné with the village of Dabatou will have to be improved over a length of 22 km in order to enable it to carry all round the year the heavy traffic for supplying the working site with equipment and materials.

Finally, there remains a final 30 km long section of road to be constructed between Dabatou and the working site.

The total distance between Dabola and the working site is thus about 190 km.

7.4.15. Development of the territory

The largest possible storage reservoir which could be built at Boureya would drown an area of about 420 km2. At present there are about 40 villages and hamlets in this area, with from one dozen to about 200 huts. The total population of this territory has been estimated at 2800 persons for whom new villages will have to be built on the shores of reservoir.

The region occupied by the reservoir is traversed by numerous tracks connecting the various villages. It will therefore be necessary to re-establish access between the noninundated villages on the one hand, and to develop on the other hand new tracks for connecting the new villages to the network of existing tracks.

The project provides for about 145 km of tracks to be upgraded into secondary roads, and 20 km of new secondary roads.

7.4.16. Construction costs

The cost of constructing the Boureya dam described in the present chapter has been worked out on the basis of unit prices set out in Volume 1A of the present Report. Theseprices take into account the geographical situation of Boureya and transport costs by rail and road.

Detailed studies have been undertaken in order to obtain an estimate of the cost of improving the access road to the site and constructing main and secondary roads. There are moreover the costs which result from displacing those villages which are to be rebuilt on the shores of the reservoir. These costs have also been estimated on the price basis given in Volume 1A of the present report.

Table 7.4.I given at the end of this chapter summarizes the quantities of the main categories of work, as well as the estimated cost of each. The main heads of work, as well as miscellaneous and contingent work, have been regrouped under the different contracts and costed on the basis of the rates set out in Volume 1A.

Taking into account the assumptions which SENEGAL-CONSULT had to make both regarding the unit prices and the conditions of financing the scheme, the total investment required for building the Boureya dam and power station as described in the present chapter, amounts to 113 million US $.

7.4.17. Energy output and generating cost

In the final stage of the river Senegal regulation, the Boureya scheme as described in the present chapter will be capable of producing 680 GWh per annum with a guaranteed power output of 85 MW available during 8000 hours per annum in 9 years out of 10.

The figures are the result of a mathematical simulation carried out by computer with the help of a programme worked out by SENEGAL-CONSULT, using the hydrological data from 66 years of observations.

The Boureya storage dam has been dimensioned with the sole aim of producing energy in the most economic manner. The annual operating costs should therefore be met entirely out of the procees of energy sales.

The calculation of these annual charges, details of which are given in Table 7.4.III. at the end of this chapter, has been carried out on the basis of the Assumptions and prices indicated in Chapter 4 of Volume 1A of the present Report. These annual charges amount to 8.49 US $ per annum. In these circumstances, the generating cost of guaranteed energy is 12.5 mills per kWh (1 mill = 0.001 US $).

.4.18. Present state of project

he summary pre-project for the Boureya site presented in this chapter could unfortnately not be worked out on as full a basis as was done in the case of the dams on ali territory when determining their characteristics. In fact the lack of number of ssential geological and geotechnical data forced SENEGAL-CONSULT to make certain ssumptions, mainly with regard to the possibility of using lateritic materials for e construction of the dam.

Only a supplementary campaign of reconnaissance would make it possible to determine exactly to what extent these materials can be used, their geotechnical characteristics, and the cost of extracting and preparing them.

This is why it has not been judged practicable to work out a detailed construction and investment programme in the way this was done in the cas of the other schemes proposed by SENEGAL-CONSULT.

Table 7.4.I

BOUREYA PROJECT

Normal reservoir level : 381 m.s.m.
Dam crest level : 386 m.s.m.

Cost Estimate

1. CONSTRUCTION COST OF DAM AND POWER PLANT	Unit	Quantity	Unit Price	Price
1.1. Dam and ancillary structures	___	___	US $	Mio US $
Civil works				
- Temporary diversion				
- Excavations				
. in soft ground	m^3	1,630,000	1.05	0.45
. in rock	m^3	415,000	3.05	1.71
- Dam embankment				
. rockfill	m^3	750,000	3.75	1.27
. pit run	m^3	4,250,000	3.20	13.61
. core	m^3	830,000	3.90	3.24
. filter	m^3	490,000	4.45	2.18
- Concrete, shuttering, reinforcement				
. mass concrete	m^3	820,000	34.50	28.25
. reinforced concrete	m^3	15,000	46.--	0.69
. steel shuttering	m^2	230,000	4.70	1.08
. wood shuttering	m^2	60,000	9.35	0.56
. reinforcement	t	400	380.--	0.15
- Curtain				
. grouting	m^2	53,000	15.--	0.80
. drainage	m^2	2,200	7.--	0.01
				56.81

				Mio US $
		carried forward		56.81
Administration				1.42
Contingencies				6.30
Total civil engineering				64.53
Equipment				
- 4 spillway gates	m^2	608	750.--	0.46
- 4 intake gates	t	151	3,040.--	0.46
- 2 bottom outlets	t	106	3,040.--	0.32
- entry grilles	m^2	420	120.--	0.05
- screens		220	500.--	0.11
- stoplogs		150	400.--	0.06
- control equipment				0.08
				1.54
Administration				0.04
Contingencies				0.15
Total equipment				1.73

1.2. Power plant and switchyard

Civil engineering

- Excavations				
. in rock	m^3	101,200	3.05	0.31
. in soft ground	m^3	21,600	1.05	0.02
- Embankment	m^3	10,500	3.20	0.02
- reinforced concrete	m^3	28,000	46.--	1.29
- plane formwork	m^2	3,000	4.70	0.01
- curved formwork	m^2	8,000	9.35	0.08
- reinforcement	t	1,800	380.--	0.68
				2.41

				Mio US $
		carried forward		2.41
Volume built	m^3	31,000	50.--	1.55
Administration				0.10
Contingencies				0.50
Total civil engineering				4.56
Equipment				
- turbines with cylindrical sleeve valves, speed governor, including installation	p	5	980,000.--	4.90
- cooling system, auxiliary set, stoplogs and portal hoist				0.37
- overhead crane				0.18
- generator, including reserve and installation	p	5	620,000.--	3.10
- single-phase transformator including reserve	p	16	53,000.--	0.85
- auxiliary services and command				1.80
- switchyard, including high and low voltage equipment				1.20
				12.40
Administration				0.31
Contingencies				1.25
Total equipment				13.96

2. COST OF RECONSTRUCTION OF ROADS

Mio US $

2.1. Access to site

Main road				
- new roads	km	30	70,000.--	2.10
- improvements of existing roads	km	40	55,000.--	2.20
- works	p	10		0.25
				4.55
Administration and contingencies				0.68
Total				5.23

2.2. Road re-routing

Secondary roads				
- new roads	km	20	35,000.--	0.70
- improvement	km	145	25,000.--	3.62
				4.32
Administration and contingencies				0.65
Total				4.97

2.3. Reconstruction of villages

- villages	1,800	195.--	0.35
Administration and contingencies			0.77
Total			0.42

		Mio US $
3.	DEVELOPMENT COST	
3.1.	Hydrology, geology, drillings geotechnics, geometer works	1.89
3.2.	General project and tendering	2.86
3.3.	Execution plans	0.95
3.4.	Consulting services	1.41
	Total	7.11
4.	INTEREST DURING CONSTRUCTION	10.38
5.	COST OF PROJECT ADMINISTRATION	notional
	TOTAL INVESTMENT	112.89

Table 7.4.II

BOUYERA PROJECT

Estimate of the investment required

Cost summary

	Entire realization of project Mio US $
1. CONSTRUCTION COST OF DAM AND POWER PLANT	
<u>Dam and ancillary structures</u>	
- civil engineering	64.5
- equiment	1.7
<u>Power plant and switchyard</u>	
- civil engineering	4.6
- equipment	14.0
Total	84.9
2. CONSTRUCTION COST OF ROADS, RAILWAYS AND VILLAGES	
Site access roads	5.2
Re-routing of roads	5.0
Reconstruction of villages	0.4
	10.6
3. DEVELOPMENT COST	
4. INTEREST DURING CONSTRUCTION	10.4
5. COST OF PROJECT ADMINISTRATION	notional
<u>TOTAL INVESTMENT</u>	112.9

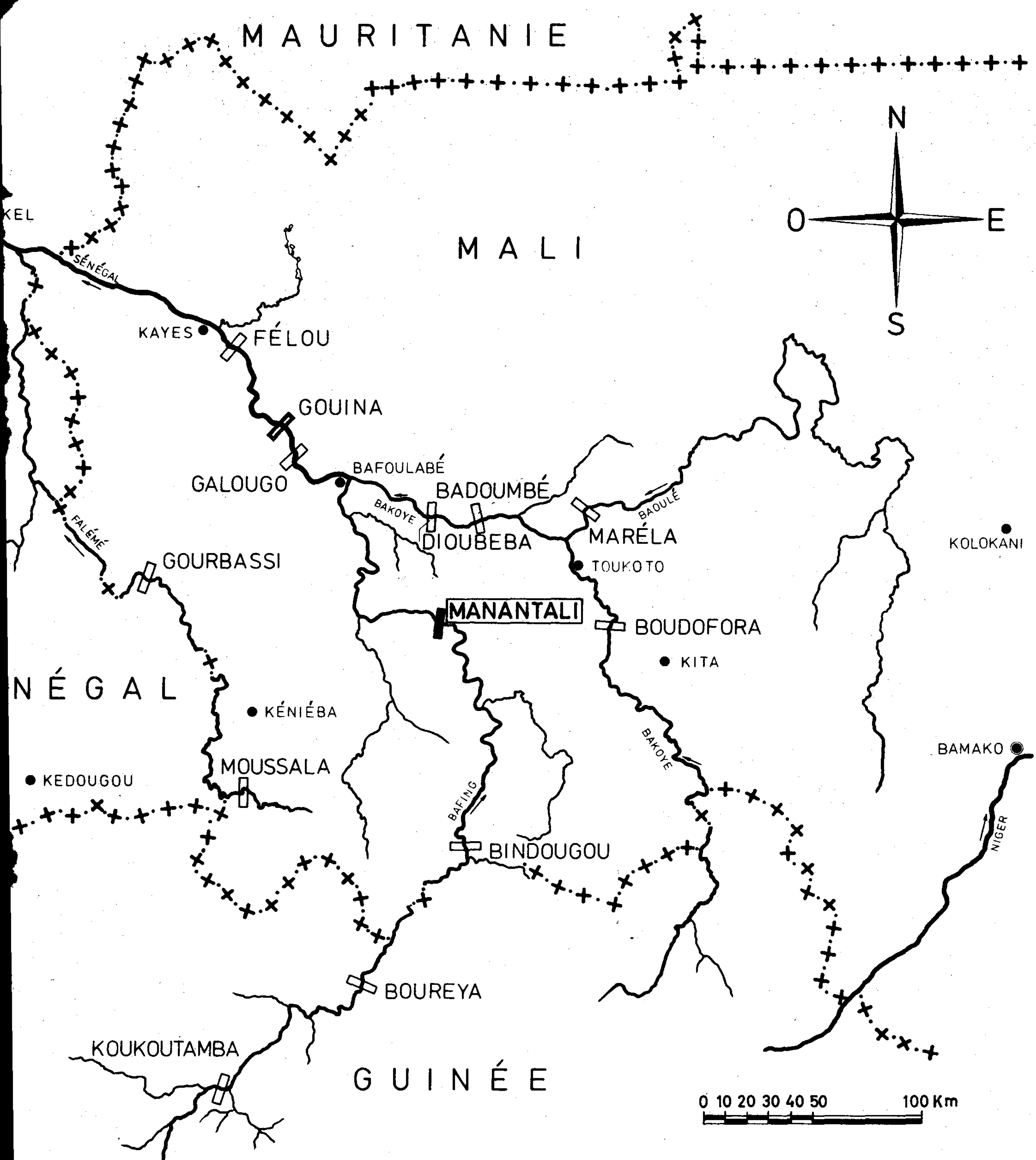
Table 7.4.III

BOUYERA PROJECT

Estimate of the investment required

Cost summary

	Entire realization of project Mio US $
1. FINANCIAL CHARGES	7.60
2. REPLACEMENT CHARGES	0.21
3. OPERATING COSTS	0.24
4. MAINTENANCE COSTS	0.44
5. COST OF ADMINISTRATION	notional
TOTAL	8.49



UNITED NATIONS	NATIONS UNIES
SENEGAL RIVER PROJECT	PROJET D'AMENAGEMENT DU FLEUVE SENEGAL

MANANTALI

GENERAL MAP
CARTE GÉNÉRALE

SENEGAL — CONSULT	SCALES ECHELLES 1:2 500 000	DATE DATE 15. 4. 70
SOCIETE GENERALE POUR L'INDUSTRIE SA GENEVE	APPROVED APPROUVE	NR NO 90 111
MOTOR-COLUMBUS BADEN ZINDER NEW YORK	MODIFIED MODIFIE 1.7.71 A	7-2-01
ELECTRO-WATT INGENIEURS-CONSEILS SA ZURICH		