1,000 m^3(natural gas) = 1.1 TEP
1 ton (heavy oil) = 1 TEP

Based on the assumed utilization factor of 80% for base-load thermal plants, 50% each for Ghannouch I and La Goulette II steam plants and 10% for gas turbine plant, natural gas consumption for a period from 1985 to 1990 for generation purpose will be estimated as follows;

(a) Base load thermal

	Installed cap.(MW)	gas consumption(10^6m^3)	Cumulative total(10^6m^3)
Ghannouch I	58	60	60
La Goulette II	95	90	150
Sousse	300	460	610
"X" steam	150	230	840
"Y" steam	150	230	1,070
"Z" steam	150	230	1,300
(b) Gas turbine			
Bouchemma	62	18	18
Ghannouch II	44	13	31
TG(A)	198	59	90

Approximately 1.4 billion cubic meters of natural gas will be consumed every year for both steam and gas-turbine power plants.

This means that there will be no problem from a view point of supply capacity, if the development of Mishar Project will be smoothly implemented. On the other hand, if there will be any trouble in the development of Miskar Project, some steam power plants and TG(B) power plant must depend on heavy oil and gas oil full.

4.2 OPERATION SEQUENCE AND OPERATION MODE

Economic feasibility of Kasseb Power station will be proven when the generating cost of the power station including capital cost, full cost, operation and maintenance cost as well as replacement cost is less than the generating cost of alternative gas turbine power plant.

Therefore, in cose Kasseb Project will be realized, the operation of Kasseb power station in the peaking hours will take precedence of the operation of TG"B" which be constructed after Kasseb power station. However for the existing gas turbine power plant (TG"A") with its comparatively small capital cost the operation will take precedence over Kasseb power plant from the sole comparison of full cost.

For the applciable fuel, there will be three assumption as follows in relation to available volume of natural gas as well as marginal supply cost of the gas;

Case "X" : Supply of natural gass is sufficient enough to cover the domestic dimand (cost of gas equals to supply cost of 25DT/TEP)

Case "Z" : Supply of natural gas is for a part of domestic demand only. (Some base-load thermal will utilize imported natural gas or heavy oil of 35DT/TEP and gas turbine will use gas oil of 50DT/TEP)

Case "Y" : Marginal supply cost of gas for slight load hours in the midnight will amount to 50% of that of peaking hours

Operation sequence of each power station for the obove cases will be as follous;

Operation sequence

1	Base-load steam (on nuclear)	Base-load steam(on nuclear)
2	Ghannouch I	Ghannouch I
3	La Goulettle II	La Goulette II
4	Hydro	Hydro
5	Bouchmma	Bouchemma
6	Ghannouch II	Ghannouch II
7	TG"A" (natural gas)	Kasseb
8	Kasseb	TG"A"(gas oil)
9	additional gas turbine(TG"B")	TG"B"(gas oil)

4.3 DAILY LOAD CURVE AND AVAILABLE POWER AND ENERGY PRODUCTION OF KASSEB POWER ATATION

4.3.1 Daily load curve

Daily load curves for the months from 1978 to 1990 were made available by STEG to the mission.

According to the curves it can be judged as follows;

(1) Maximum demand of a year will be realized in December followed with November and March. And minimum demand will be Observed in June. (In crease in Winter season is considered due to the load for heating)

(2) Peaking demand will be obsered most early at 19:00 in Winter (November, December and January) and gradually occurred later as it becomes warmer and warmer (April: 20:00, July : 21 : 00 - latest peaking hour). It tends to occur earlir and in September at 20 : 00.

(3) Duration of peaking hours in Winter is a little bit longer than those in summer (Winter : 5.5 h, summer : 3.1 h)

(4) The above tendencies do not change up to the year of 1990.

It was supposed by the mission that the following tendencies will be realized in accordance with the elapse of the years.

(1) Big demand will be increased

(2) Shape of peaking load will become dull

(3) Rate of minimum load against maximum load will be increased

However, in actual, the said tendencies can not be observed up to 1990, and almost all the load curves represent similar shape.

Load graphs were not especially prepared for Sundays and holidays, but their peaks would be less than 80% of weekdays while the morning peaks wouls be approximately 60% so that the load curves will be extremely flattened. Pumped up on these days would not be required to be used for power generation the same day and can be held for use on weekdays, which would be more desirable.

4.3.2 Available power and energy production of Kasseb power station

Based on the operation sequence mentioned in 4.2 and daily load curve in 4.3.1, operation wodes of Kasseb power station for 10 year period from 1985 to 1994 are abtained as shown in Fig 4-1 through 4-11.

The available power and annual energy production of Kasseb power station at the end of each year will be as follows:

Tableau 4-1 PUISSANCE EXPLOITEE (Fin d'année)

Année	Hypothèse "X"		Hypothèse "YZ"	
	Production	Réserve	Production	Réserve
1985	-	75	75	-
1986	-	150	71 (150)	79 (-)
1987	-	150	150	-
1988	63	187	250	-
1989	77	273	275	75
1990	-	350	133	217
1991	109	241	263	87
1992	229	121	350	-
1993	141	209	295	55
1994	350	-	350	-

Note: Chiffres entre parenthèses indiquent la puissance exploitée et celle en réserve à la fin juillet 1986.

Tableau 4-2 ENERGIE PRODUITE ET L'ENERGIE POUR LE POMPAGE

Année	Hypothèse "X" Production	Hypothèse "X" Réserve	Hypothèse "YZ" Production	Hypothèse "YZ" Réserve
1985	-	-	42	65
1986	-	-	102	157
1987	-	-	135	207
1988	32	49	249	383
1989	39	60	273	420
1990	-	-	81	124
1991	56	86	240	369
1992	172	264	369	567
1993	71	109	267	410
1994	369	567	369	567

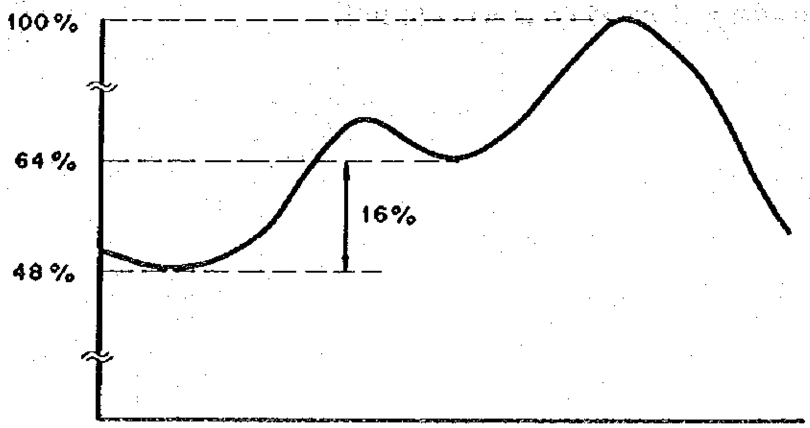
4.4 INTERNATIONAL POWER EXCHANGE

When the various electric power resources will be developed and strengthened in accordance wide the criteria of reserve capacity and development schedule mentioned in Chapter 3, a part of required power for pumping up for Kasseb power station must be supplied as follows from international tie line at the slight load hours in the mid-night.

	Year	Required power	hour
	1986(Jan.-July)	23 - 30 MW	2.6
1988	1988	44 - 138 MW	5.0
	1989	24 - 72 MW	4.3

The above firgure for required power is the one for the end of December of each year when the demand and supply balance is most serious.

4.4.2 Time Pump-up with 4 Units Possible



The time that pump-up with 4 units will be possible, considering that the shape of the daily load curve does not change, is when pumping power will fit into the 16% part of the figure above. Since the pumping power for the 4 units of Kasseb is (90 + 120) × 2 = 420,

16% . 420 MW
100% 2,625 MW

In effect, it will be when maximum demand becomes approximately 2,600 MW, and this, according to the load forecast of STEG, will be roughly around the year 2001. (Hypothese moyenne of 2001 is 2,580 MW.) However, what would require attention here is that because the upper reservoir capacity is not infinite, giving primary consideration to raising the load factor of base thermal, pump-up with the 4 units of Kasseb for long hours in the night cannot be done.

If the shape of the daily load curve does not change in the future, it is thought desirable for the STEG System for pump-up at Kasseb to be done in the night over a long period of time with less than full input (in other words, pump-up with 3 units or 2 units), and power generation carried out during peak hours for a short time at full output.

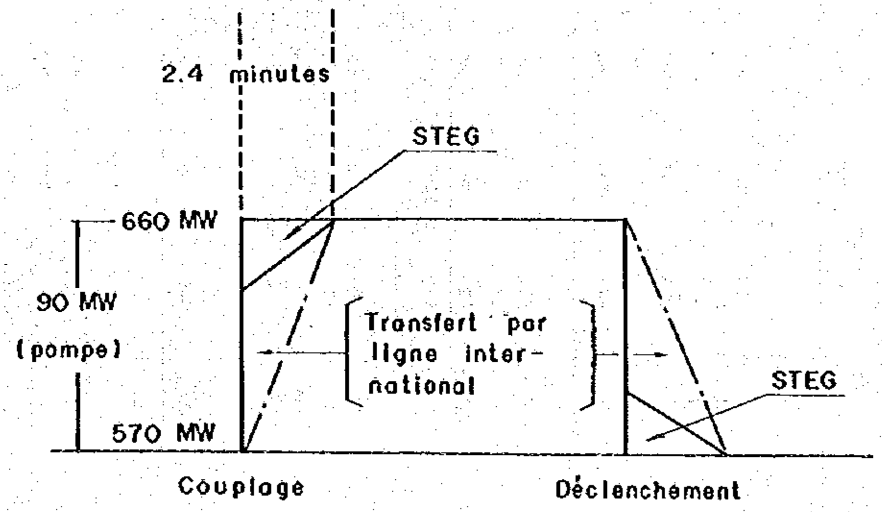
4.5 OUTPUT SHARING OF BASE LOAD THERMAL AT PUMPING

Fig. 4 - 13 shows the assumed situation in December, 1987. It is ecomical and preperable from a view-point of operation to share the output required in midnight and at pumping with the respective percentage of installed capacity against the total capacity of 753 MW composed of Sousse, X, Y, Ghannouch and Goulette II power stations.

At the parallel on release with power system of pumping unit, it is necessary to operate the above base - load thermal power stations to produce the increase of output (on decrease of output) as much as possible.

At the time of the solid operation, incoming or outgoing of certain amount of electric power from or to Algeria cannot be avoided. The flow of electric power will be as follows:

The load pattern at the time of parallel or release with the power system will be as shown in solid line of the following illustration, based on the severe assumption that the load will be suddenly charged from zero to full.



The increase of output of base - load thermal power stations of STEG will be possible up to 7%/minute from specifications. However, in consideration of allowance and handling delay in operation, 5 %/minute will be conceivable. Then the time T required to imcrease the output from 570 MW to 660 MW based on the total installed capacity of base - load thermal power stations (753 MW composed of X - 150 MW, Y - 150 MW, Sousse - 150 MW × 2, Ghannouch - 58 MW and Goulette II - 95 MW) will be as follows;

$$T = \frac{660 - 570}{753 \times 0.05} = 2.39 = 2.4 \text{ minutes}$$

Therefore, the actural change of output will be as shown in dotted line in the above illustration. At this moment, due to the drop of frequency of the system, electric power of $k.\Delta f$ will be supplied from the total power system. If system constant K will be the same volue for both STEG and Algeria systems 90 MW $\times \frac{3}{4} = 67.5$ MW (instant) will flow in from Algeria in consideration of output sharing to the percentage to the system capacity (corresponds to the portion covered by oblique lines in the above illustration). Electric power at this moment will be;

$$90 \times \frac{3}{4} \times \frac{2.4}{60} \times \frac{1}{2} = 1.35 \text{ MWh.}$$

68 MW will be equivalent to 3.6 % of total system capacity of Algeria ($\frac{68}{570 \times 3} = 3.6\,\%$)

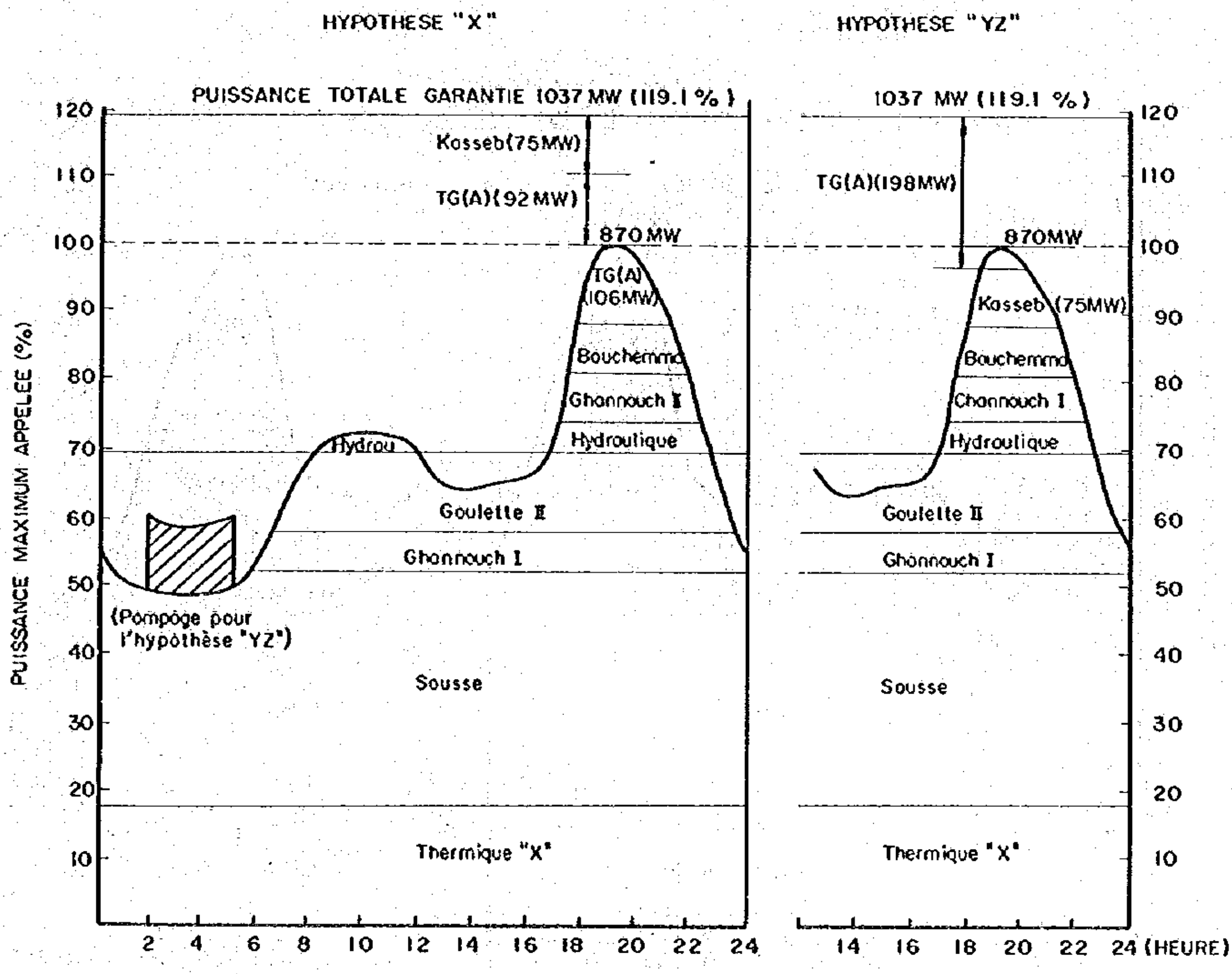
It is noteworthy that the above - mentioned checking are performed based on the severer considerations. In actual, load pattern of pumping unit at the time of parallel with power system dose not represent stair - steps, but gradually increase with a gradient more than 90 MW/minute. At the release with power system, the frequency of the system with raise, but there will not be serious effects to the customers in comparison to that of frequency drop, in consideration of expectable governer effect of generators connected to the power systems of both STEG and Algeria.

Tableau 4-3 ORDRE D'EXPLOITATION DES CENTRALES ET LA PUISSANCE EN RESERVE

(MW)

Hypothèse	Description	1985	1986 Jan.-Juillet	1986 Août-Déc.	1987	1988	1989	1990	1991	1992	1993	1994
	Puissance maximum appelée	870	920	970	1.060	1.160	1.280	1.390	1.520	1.640	1.790	1.940
HYPOTHESE "X"	Ordre d'Exploitation											
	1° Nucléaire (ou thermique)	-	-	-	-	-	-	300	300	300	600	600
	2° Thermique vapeur	603	603	753	753	753	903	855	855	855	855	855
	3° Hydraulique	40	40	40	40	40	40	40	40	40	40	40
	4° Turbines à gaz (TG"A")	227	277	177	267	304	260	195	216	216	154	-
	5° Centrale de Kasseb	-	-	-	-	63	77	-	109	229	141	350
	6° Turbines à gaz additionnelles (TG"B")											95
	Total	870	920	970	1.060	1.160	1.280	1.390	1.520	1.640	1.790	1.940
	Puissance en réserve											
	Turbines à gaz	92	27	127	37	-	-	65	100	200	200	305
	Centrale de Kasseb	75	150	150	150	187	273	350	241	121	209	-
	Total	167	177	277	187	187	273	415	341	321	409	305
	Puissance totale garantie	1.037	1.097	1.247	1.247	1.347	1.553	1.805	1.861	1.961	2.199	2.245
HYPOTHESE "YZ"	Ordre d'Exploitation											
	1° Nucléaire (ou thermique)	-	-	-	-	-	-	300	300	300	600	600
	2° Thermique vapeur	603	603	753	753	753	903	855	855	855	855	855
	3° Hydraulique	40	40	40	40	40	40	40	40	40	40	40
	4° Ghannouch II et Bouchemma	121	106	106	106	106	62	62	62	62	-	-
	5° Centrale de Kasseb	75	150	71	150	250	275	133	263	350	295	-
	6° Autres-turbines à gaz	31	21	-	11	11	-	-	-	33	-	95
	Total	870	920	970	1.060	1.160	1.280	1.390	1.520	1.640	1.790	1.940
	Puissance en réserve											
	Centrale de Kasseb	-	-	79	-	-	75	217	87	-	55	-
	Turbines à gaz	167	177	198	187	187	198	198	254	321	354	305
	Total	167	177	277	187	187	273	415	341	321	409	305
	Puissance totale garantie	1.037	1.097	1.247	1.247	1.347	1.553	1.805	1.861	1.961	2.199	2.245

Graphique 4-1 SCHEMA D'EXPLOITATION (DECEMBRE 1985)

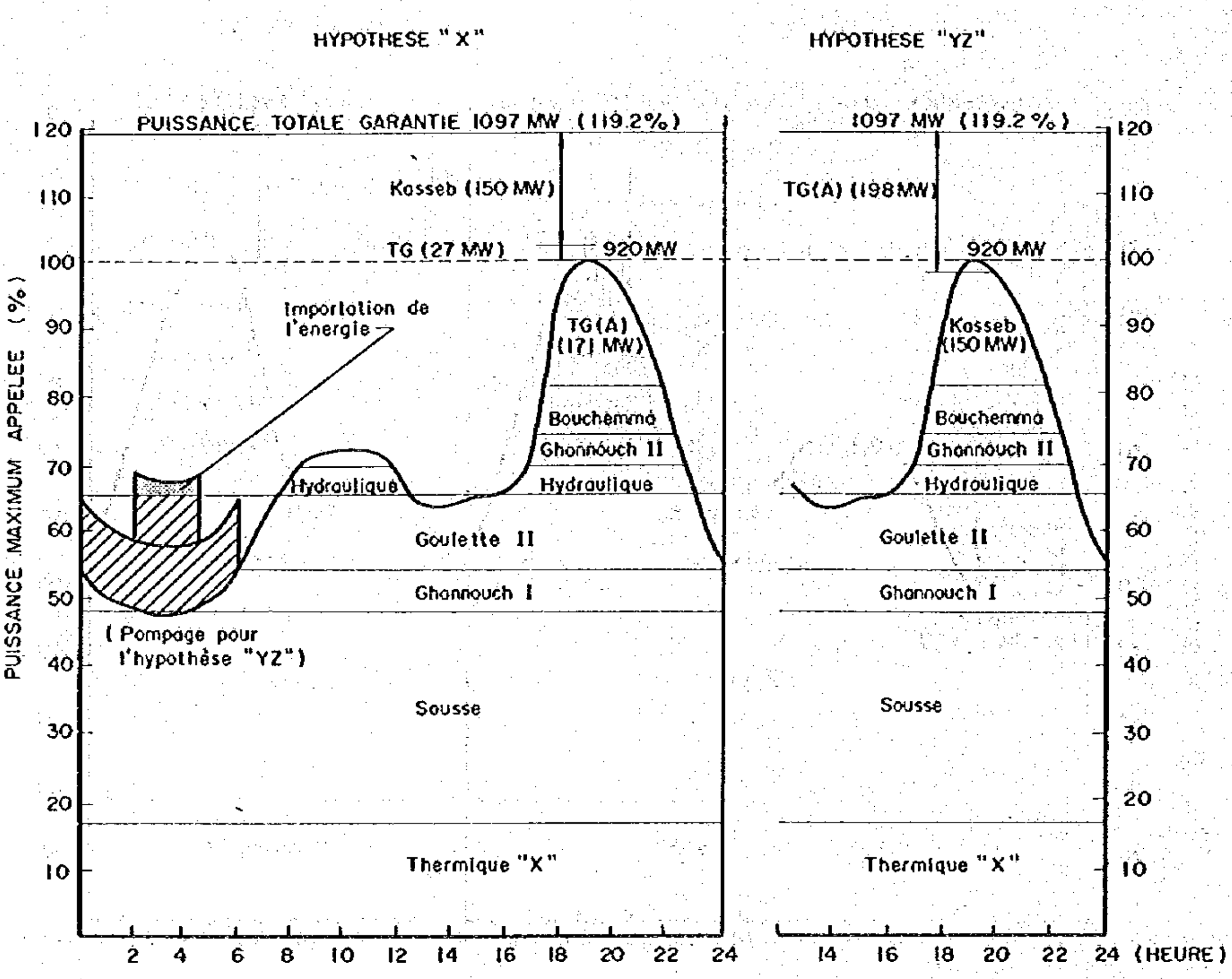


Note :

- Durée de temps de pompage 3,2 heures
- Energie pour le pompage 288 MWh
- Production en régime de turbinage 188 MWh

TG(A) ······ Tunis Sud (66 MW), Sfax (44 MW), Menzel Bourguiba (44 MW), Korba (22 MW) et Metlaoui (22 MW) totalisent 198 MW.

Graphique 4-2 SCHEMA D'EXPLOITATION (JUILLET 1986)

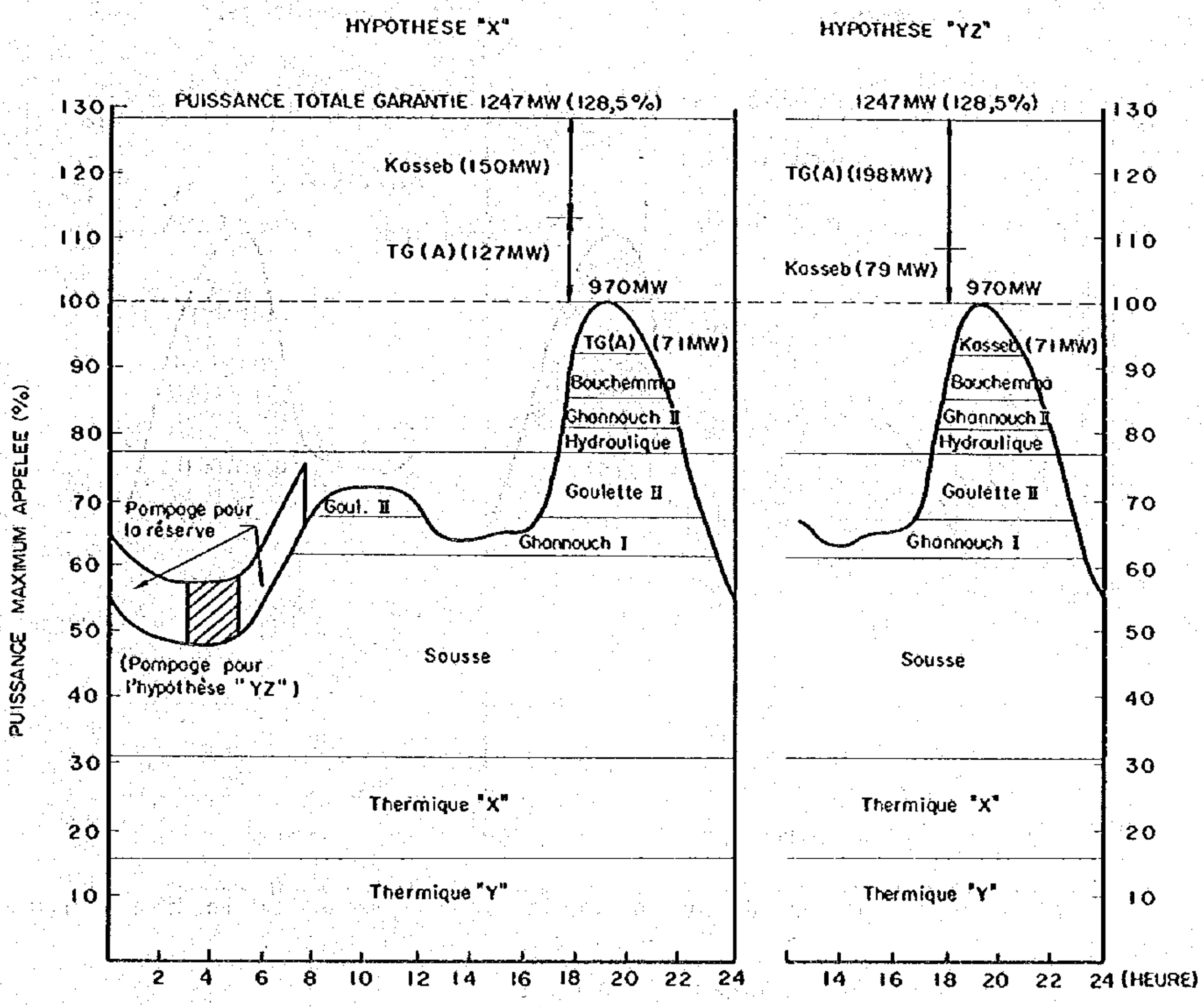


Note :

- Durée de temps de pompage 8.6 heures
- Energie pour le pompage 770 MWh
- Production en régime de turbinage 500 MWh

L'importation de l'énergie (de 23 à 30 MW x 2.6 heures) est nécessaire.

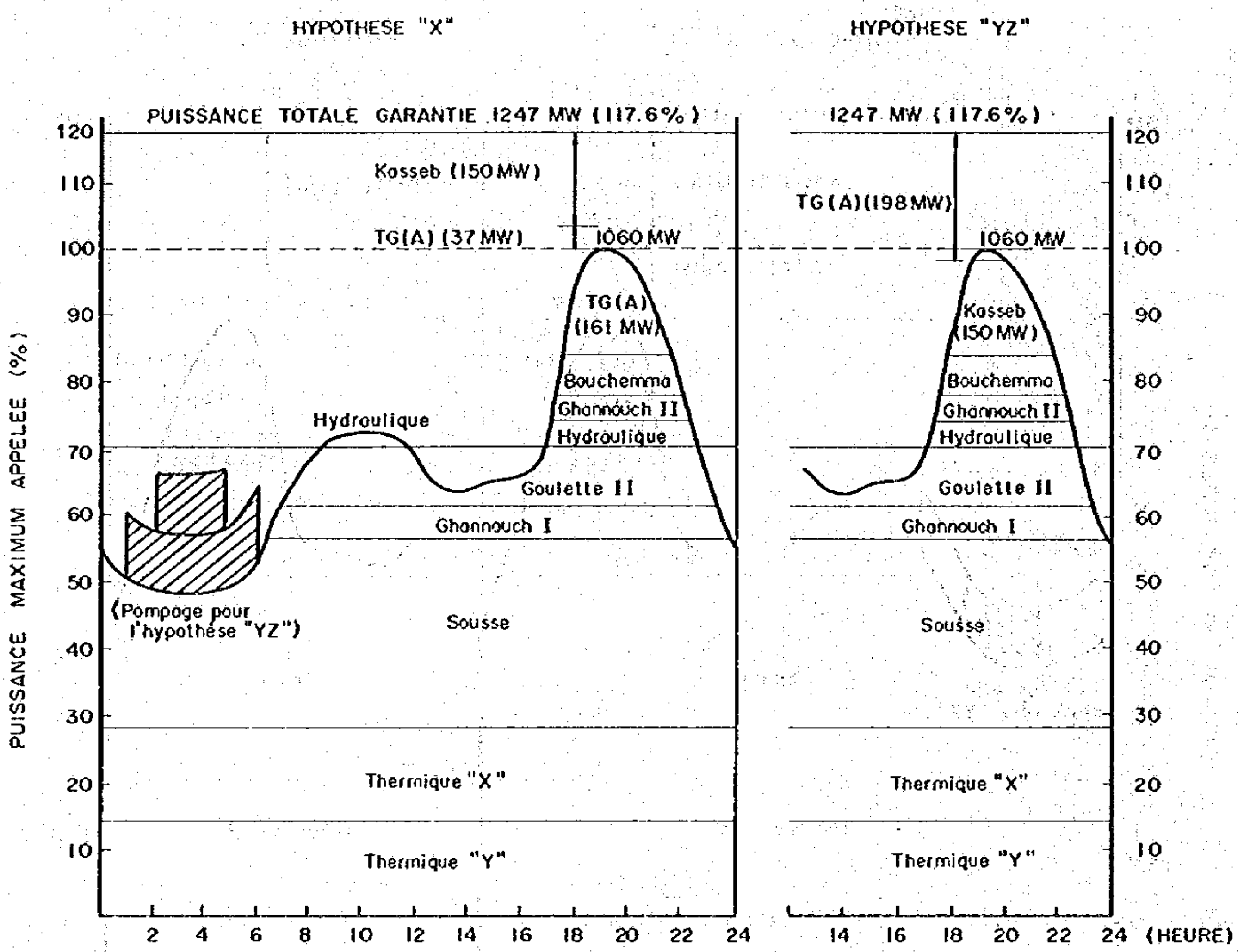
Graphique 4-3 SCHEMA D'EXPLOITATION (DECEMBRE 1986)



Note :

- Durée de temps de pompage 2 heures
- Energie pour le pompage 180 MWh
- Production en régime de turbinage 117 MWh
- Pompage pour la réserve 90 MW x 5,7 heures = 513 MWh

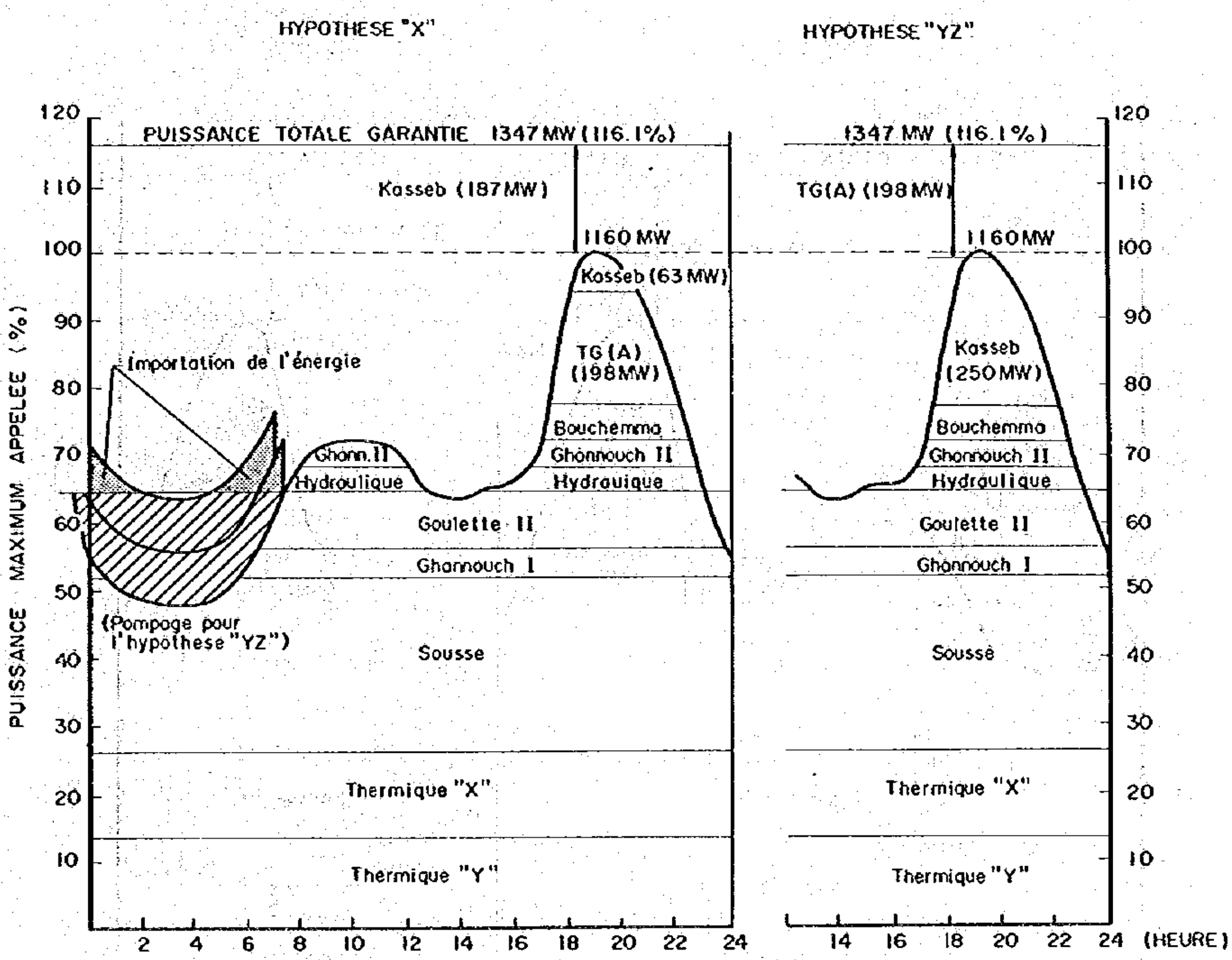
Graphique 4-4 SCHEMA D'EXPLOITATION (DECEMBRE 1987)



Note :

- Durée de temps de pompage 7.7 heures
- Energie pour le pompage 693 MWh
- Production en régime de turbinage 450 MWh

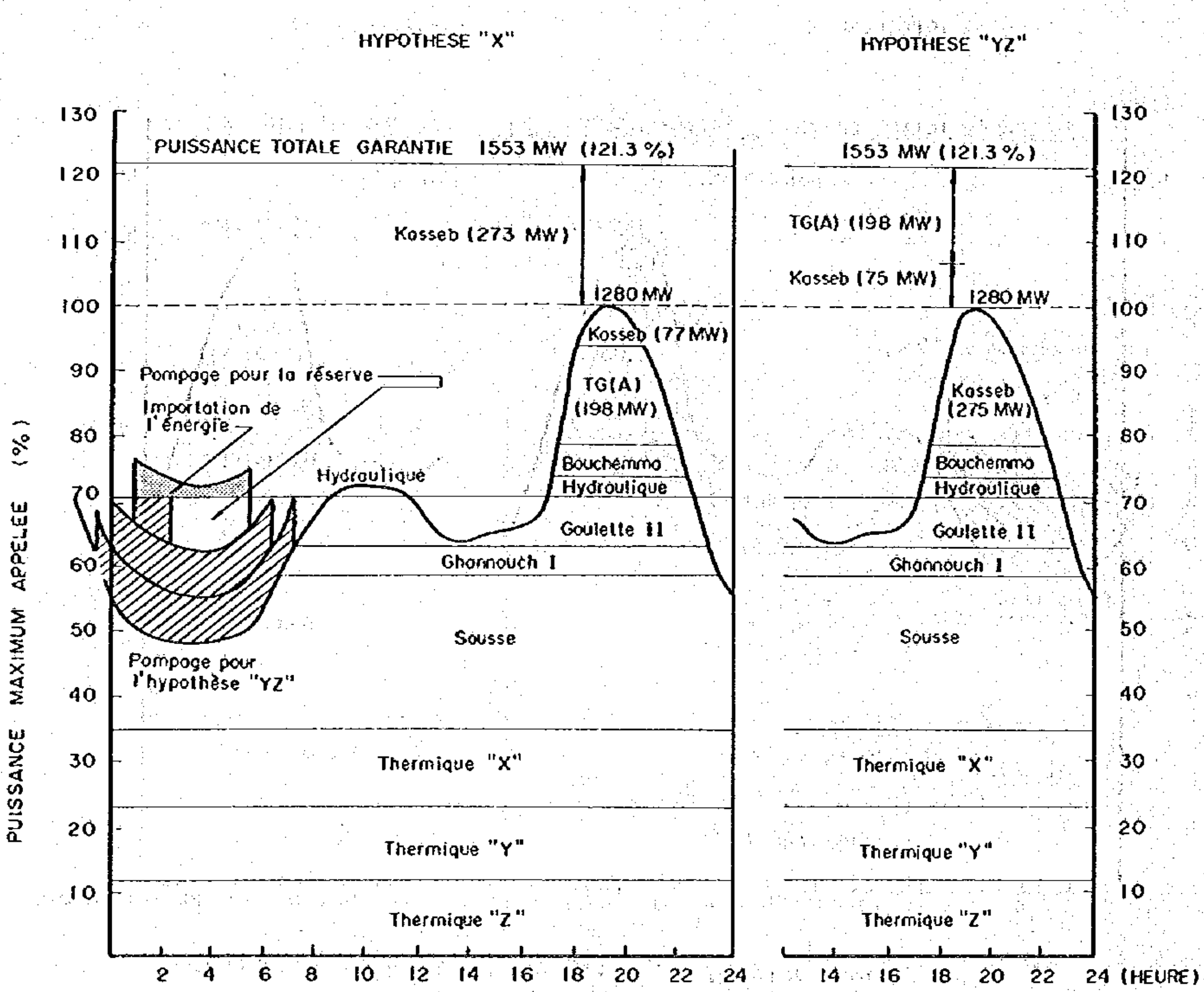
Graphique 4-5 SCHEMA D'EXPLOITATION (DECEMBRE 1988)



Note :
- Durée de temps de pompage 14.2 heures pour 2 pompes de 90 MW chacune
- Energie pour le pompage 1280 MWh
- Production en régime de turbinage 830 MWh

L'importation de l'énergie (de 44 à 138 MW) est nécessaire.

Graphique 4-6 SCHEMA D'EXPLOITATION (DECEMBRE 1989)

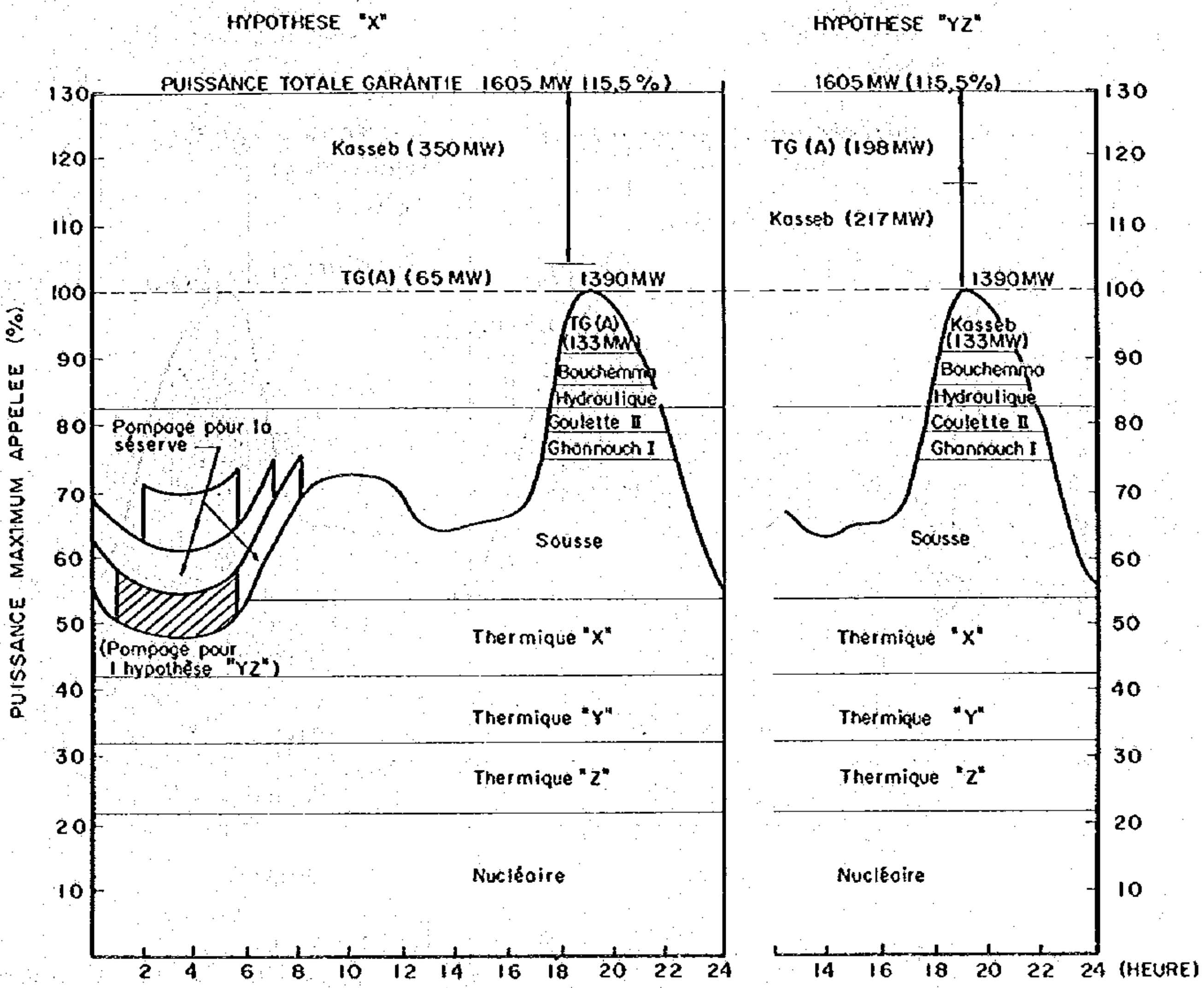


<u>Note</u> :

- Duréede temps de pompage: 14 heures pour 2 pompes de 90MW chacune; 1.2 heures pour une pompe de 120 MW
- Energie pour le pompage: 1400 MWh
- Production en régime de turbinage: 910 MWh
- Pompage pour la réserve: 120 MW x 3,1 heures = 372 MWh

<u>L'importation de l'énergie (de 24 à 72 MW x 4.3 heures) est nécessoire.</u>

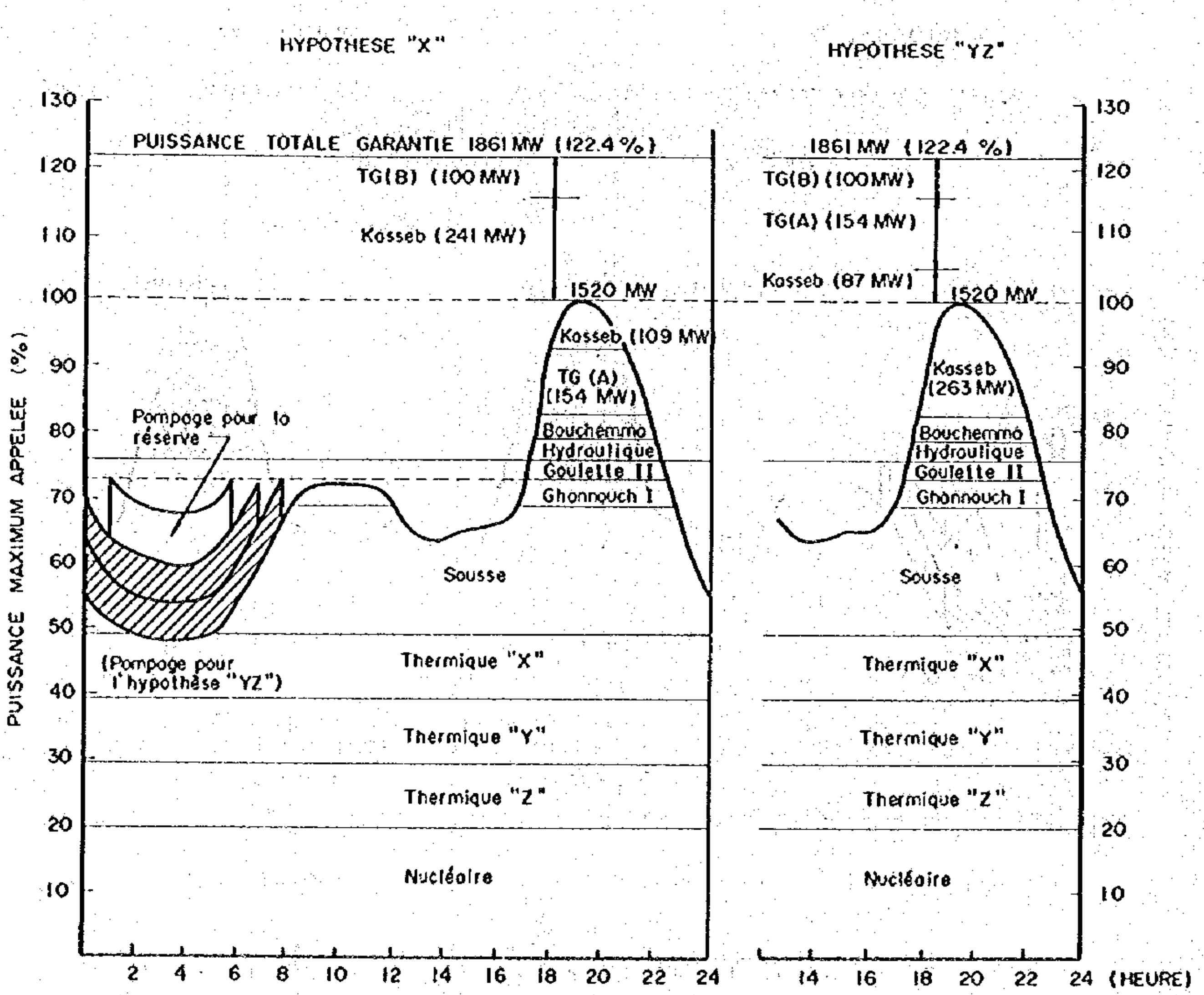
Graphique 4-7 SCHEMA D'EXPLOITATION (DECEMBRE 1990)



Note :

- Durée de temps de pompage — 4,6 heures
- Energie pour le pomage — 415 MWh
- Production en régime de turbinage — 270 MWh
- Pompage pour la réserve — (90 MW x 10,5 H) + (120 MW x 3,5 H) = 1360 MWh

Graphique 4-8 SCHEMA D'EXPLOITATION (DECEMBRE 1991)

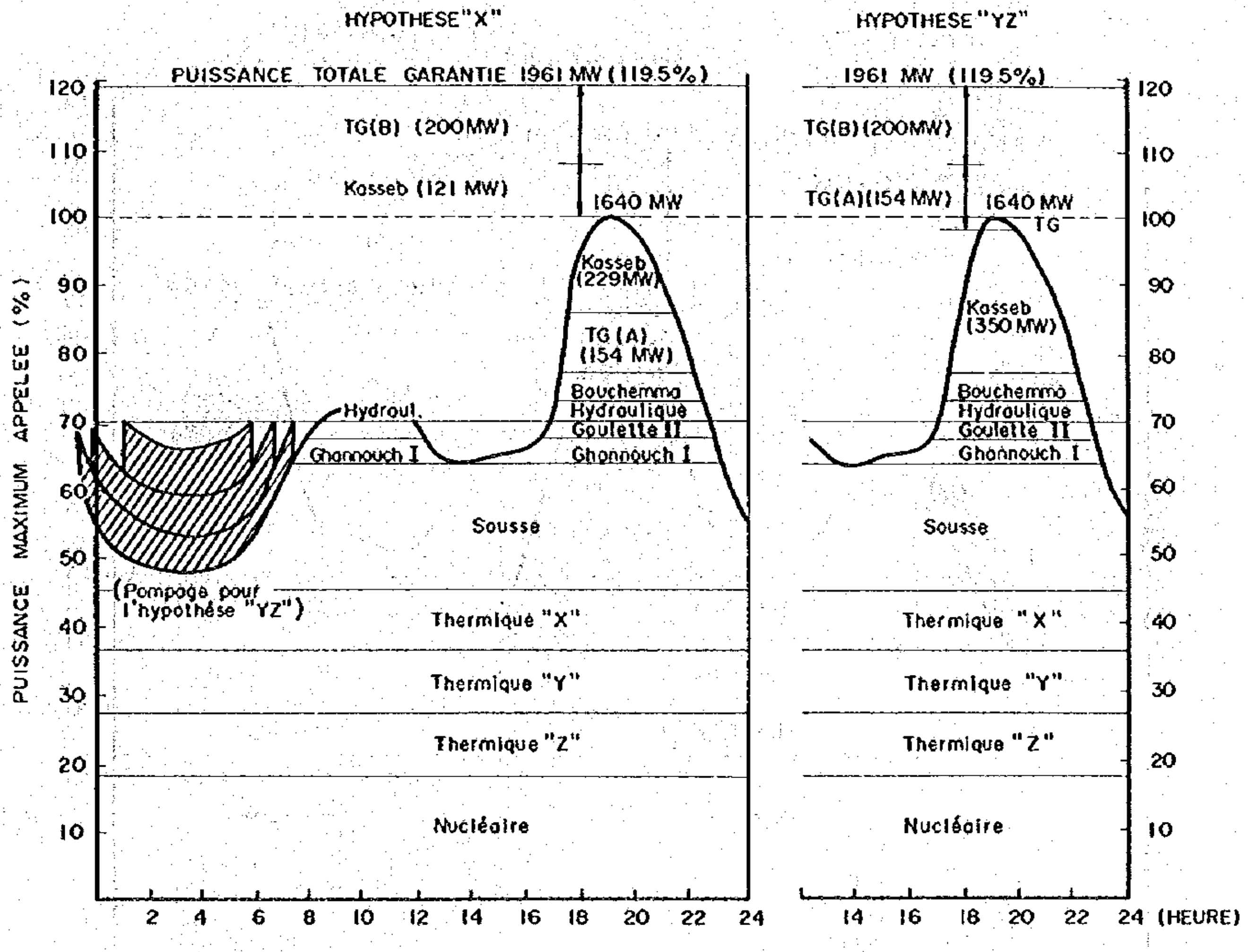


Note :

- Durée de temps de pompage 13.7 heures
- Energie pour le pompage 1233 MWh
- Production en régime de turbinage 800 MWh
- Pompage pour la réserve 120 MW x 4.5 heures = 540 MWh

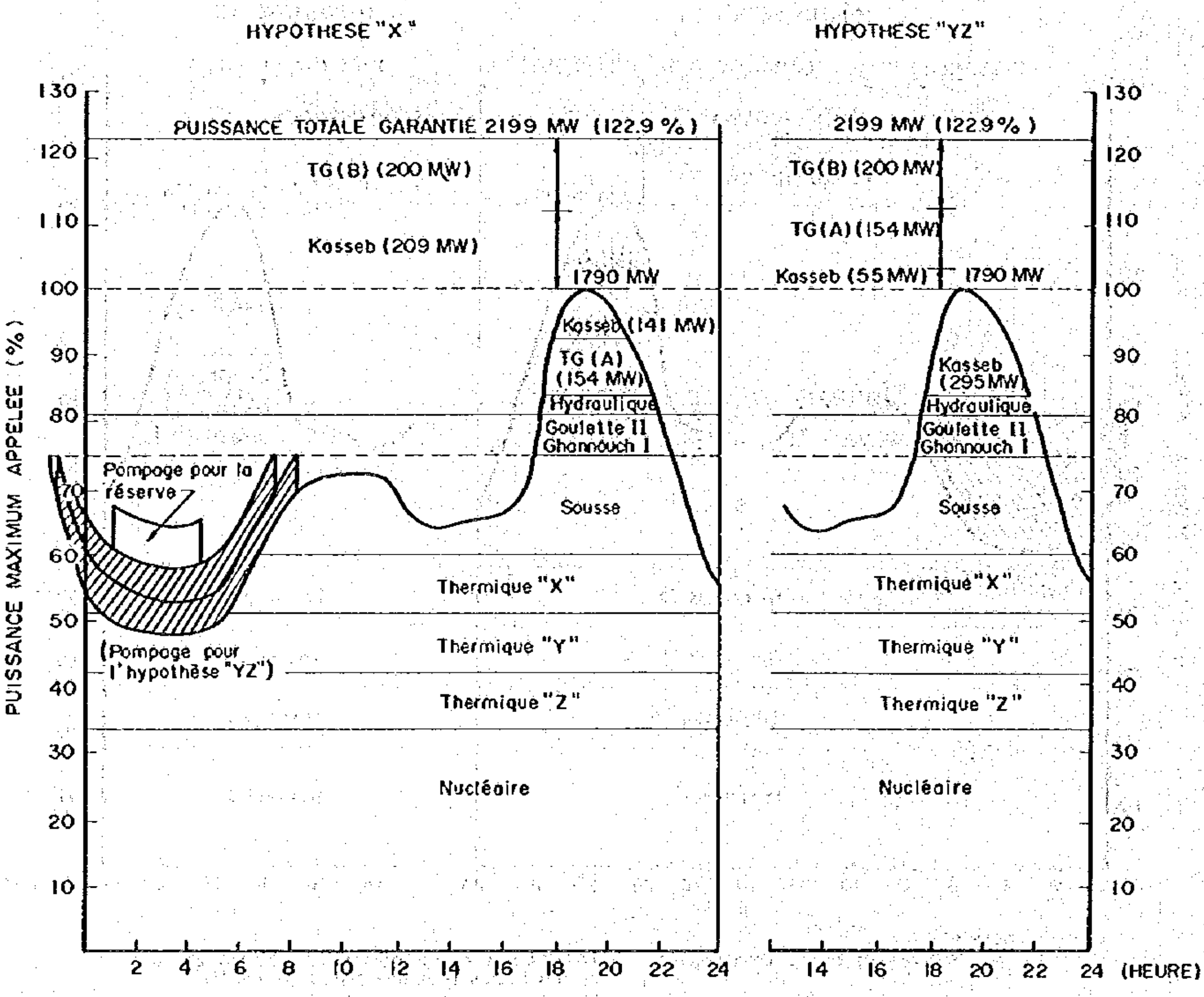
TG(B) ···· Turbines à gaz installées après la mise en service de la centrale de kosseb

Graphique 4-9 SCHEMA D'EXPLOITATION (DECEMBRE 1992)



Note :
- Durée de temps de pompage 14.5 heures pour 2 pompes de 90 MW chacune et 4.8 heures pour une pompe de 120 MW
- Energie pour le pompage 1882 MWh
- Production en régime de turbinage 1230 MWh

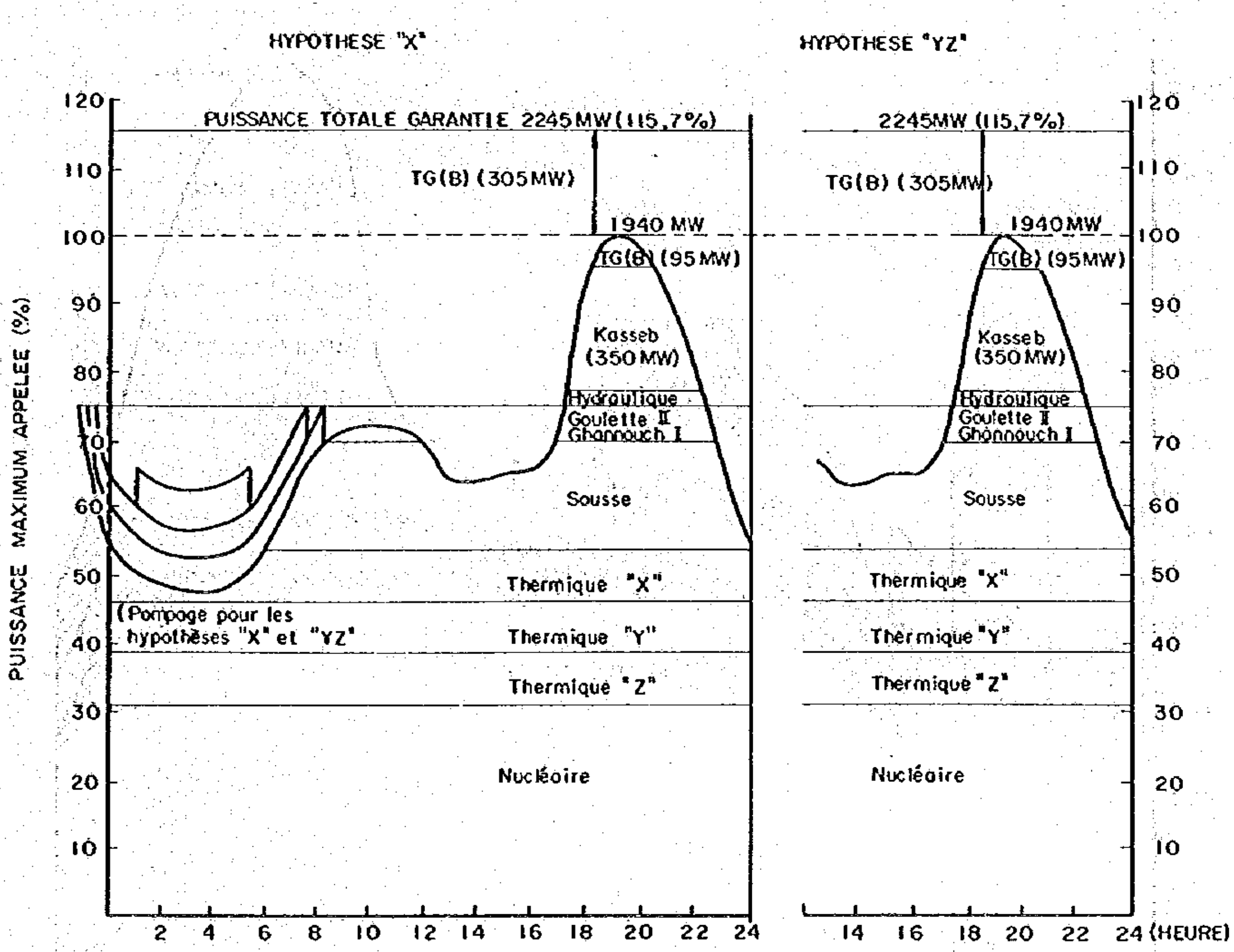
Graphique 4-10 SCHEMA D'EXPLOITATION (DECEMBRE 1993)



Note :

- Durée de temps de pompage — 15.2 heures pour 2 pompes de 90 MW chacune
- Energie pour le pompage — 1370 MWh
- Production en régime de turbinage — 890 MWh
- Pompage pour la réserve — 120 MW x 4.3 heures = 516 MWh

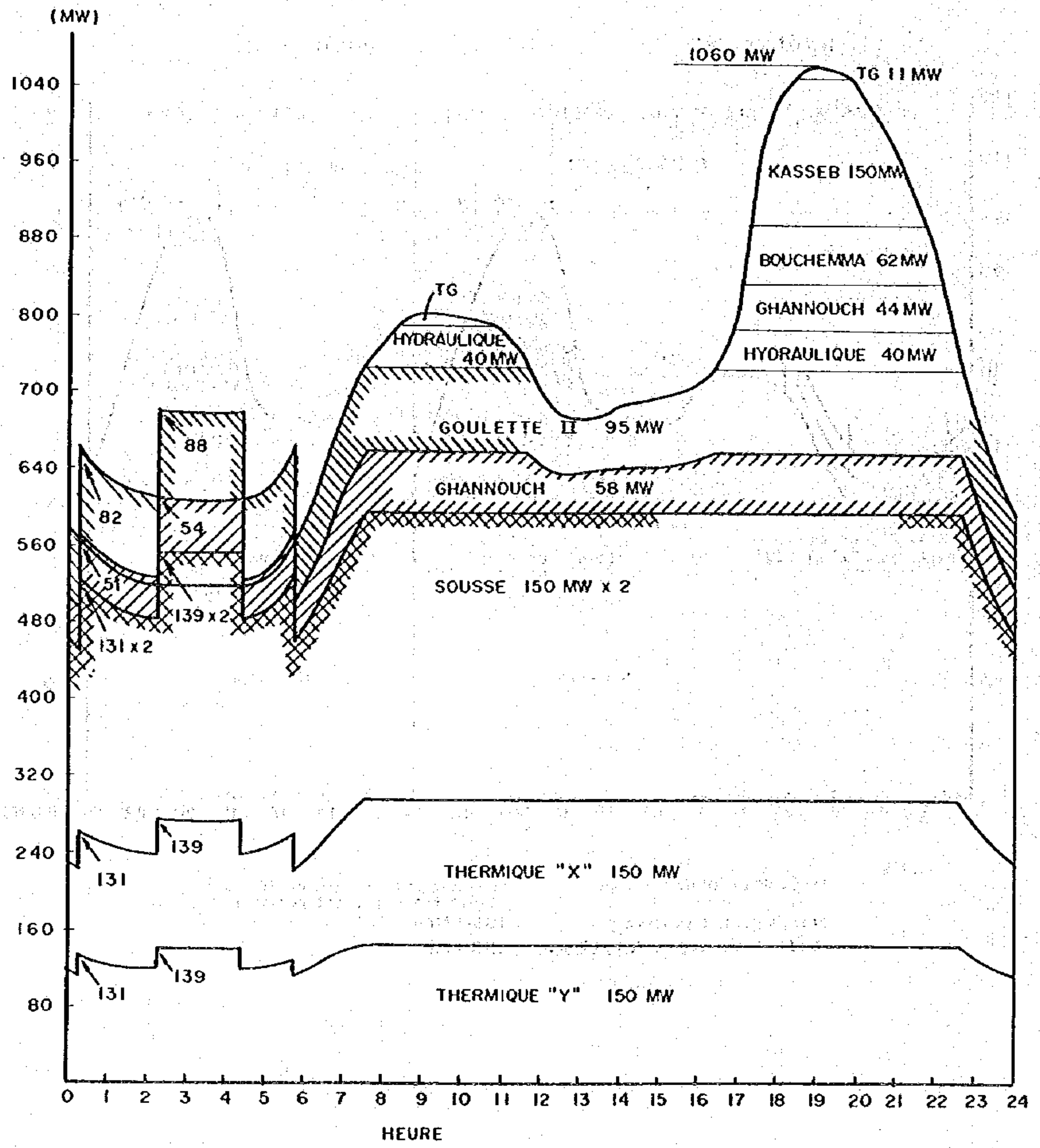
Graphique 4-11 SCHEMA D'EXPLOITATION (DECEMBRE 1994)



Note :

- Durée de temps de pompage — 15 heures pour 2 pompes de 90 MW chacune et 4,5 heures pour une pompe de 120 MW
- Energie pour le pompage — 1890 MWh
- Production en régime de turbinage — 1230 MWh

Graphique 4.12 SCHEMA D'EXPLOITATION PRATIQUE (FIN 1987)



CHAPTER 5. GEOLOGY

CHAPTER 5 GEOLOGY

5.1 GENERAL

The geological investigations for this pumped-storage power generation project was commenced form 1972, and up to the present, subsurface investigation works by such means as geological mapping, photogeological interpretation, boring, test pits, etc., have been carried out. of these subsurface investigation works, the principal ones were 13 core boring holes of total length of 748.4 m (with permeability tests conducted for all of the holes), and 51 test pits of total depth of approximately 170 m. The results of these investigations have already been compiled as reports and geological data indicated at the end of this chapter.

The Japanese Survey Team stayed in the vicinity of the Project Area for about 2 weeks from January 28, 1978 and carried out field investigations, but during this period the Team was plagued by cold rain and strong winds, and it was not possible to carry out adequate reconnaissances of the Project Area. Consequently, the result was that aerial photographs were greatly relied on to obtain an understanding of the topogra phical and geological conditions of the Project Area.

This chapter describes the geological conditions of this site, the problematic points from an engineering geological standpoint and the geological investigations required hereafter, based on the results of examinationg of exsiting geological data, and the field reconnaissances and photogeological interpretations mentioned above.

5.2 REGIONAL GEOLOGY (See Fig. 5-1)

The Project Area is approximately 15 km further west-northwest of the town of Beja about 90 km west of Tunis, and is located at the upstream part of the kasseb River, a tributary of the largest river in Tunisia, the Medjerda. The upper reservoir dam of this Project is planned at a highland of elevation higher than 400 m approximately 3 km west-northwest of the Kasseb Arch Dam constructed on the Kasseb River in 1969, while the power station is planned inside the mountain mass of Djebel El Fahama (peak elevation 540 m) running in the NEW-SWS direction along the left bank of Kasseb Reservoir which will serve as the lower reservoir.

The bedrock of this project area is comprised of strata from the Companien of Upper Cretaceous to the Oligocene of Palaeogene, these strata principally being limestone, marl, and marly limestone, with glauconitic sandstone and others at places. These rocks are generally distributed as alternations.

Regarding topography and geology in this area, formations consisting mainly of limestone are distributed at the mountain mass with outcrops numerous as a whole, whereas at flat areas strata which are chiefly marl are distributed with the feature being that there are practically no outcrops.

The geological structure of a wide region including this Project Area, while indicating broadly a NE-SW structural trend, locally presents a complex geological structure due to folding and faulting. Looking only at the Project Area, although the strata are cut by numerous faults while locally there are indications of small folded structures, the general strikes and dips are N0°-40°E, 15°-70°NW, and stratigraphically, it may be said that a relatively simple geological structure is presented.

5.3 ROCK TYPES

Various types of rock are distributed in the form of althernations in this Project Area as previously described. The Nithologic characters of these rocks are as described below.

Limestone

The limestone is gray to dark gray, mainly mediumgrained, but fine-grained and dense in parts, and is very hard. In general, bedding planes are distinct,and there are intercalations of marly limestone layers which are 2 to 3 cm in thickness at intervals of 20 to 30 cm where congested, and 2 to 3 m where not congested.

Marly Limestone

The marly limestone is dark gray to black and is mostly fine-grained, dense and hard. The hardness is slightly less than that of limestone. Bedding planes are distinct and there are frequent intercalations of hard limestone 20 to 30 cm in thickness, while at some places there are intercalations of slightly soft marl 2 to 3 cm in thickness. This rock is somewhat exfoliative along bedding planes.

Marl

The marl is grayish black to black in color and is mostly very fine-grained. The marl is generally soft with the hardness varying slightly depending on the calcium carbonate ($CaCO_3$) content. The harder the rock, the richer $CaCO_3$ content in rock is. The resistance to weathering is very small. At places, the marl has intercalations of more calcareous and slightly hard layers 2 to 3 cm in thickness, marly limestone and glauconitic portions, and bedding planes are observable.

Glauconitic Sandstone

The glauconitic sandstone is dark greenish gray in color, medium-grained, dense and hard. At certain places there are marly portions which are very fine-grained. It is generally non-calcareous, while marly portions are slightly calcareous. Bedding planes are generally destinct.

These rocks have been broadly divided into the four formations below in the descriptions and attached drawings of the text from an engineering geological standpoint based on the results of field reconnaissances and photogeological interpretations.

Limestone Formation (LS)

This is a formation where limestone is overwhelmingly dominant, outcrops of which are greatest in number in this area, and often comprises the nucleous of the mountain mass. This formation has locally intercalations of marly limestone, the thickness of which exceeds 10 m at places. Consequently, it is necessary to keep in mind that this formation is not comprised of massive limestone only.

Marl Formation (ML)

This is a formation where marl is overwhelmingly dominant. There are intercalations at places of more calcareous layers, marly limestone, limestone and glauconitic sandstone of thickness not more than 1 m.

In this Project Area, there are almost no outcrops further than local exposures near the vicinity of the river bed of the small stream (nameless) flowing down from the upper reservoir damsite to the lower reservoir, and the greater part comprises a flat plane with its surface covered with overburden consisting of topsoil or alluvium deposits. The existence of this formation in this area has been confirmed mainly through test pits and drill holes.

Compared with the other three formations, this formation is the weakest and most brittle, and also poses various problems in execution of civil work.

Marly Limestone-Limestone Alternation (MLS/LS)

This formation consists mainly of alternations of marly limestone and limestone, but has intercalations of marl at places. As a whole, these are alternations where marly limestone is slightly more dominant over limestone. The layer thickness of marly limestone is a maximum of approximately 10 m while that of limestone is approximately 5 m at most. At the ground surface this alternation is distributed accessory to the limestone formation and comprises a part of the mountain mass, but whereas limestone layer portions are outcrops, the marly limestone layer portions are covered by thin topsoil at places. In aerial photograghs, the limestone layers are the parts seen to be bands light in color and the marly limestone layers dark in color.

Glauconitic Sandstone Formation (GS)

This formation consists shiefly of glauconitic sandstone, but has intercalations of thin marl or marly limestone at places. There is little distribution of this formation in the vicinity of the Project Area, but there is a distribution at a hill approximately 1 km south of the upper reservoir.

Hereinafter, when referring to the above four broadly-divided formations in the text, they will be termed Formation LS, Formation ML, Alternation MLS/ML and Forma tion GS, respectively.

5.4 SITE GEOLOGY

5.4.1 Upper Dem (See Figs. 5 - 2, 5 - 3)

(1) Topography

The damsite is on the north side of the mountain mass extending in the NEN-SWS direction, that is Djebel* El Fahama with its peak at El. 540 m, on a small stream which flows down in the southwest direction to enter Kasseb Reservoir. (* means "mountain" in Arabic.)

At the damsite the left-bank side (Djebel El Fahama side) is a slope of approximately 30°, while the rightbank side is a slope of 5° to 15°. The riverbed part of the stream at the damsite is at El. 390 m to 395 m, while the proposed high water level is El. 430 m, with the valley width approximately 300 m. The river gradient of the stream is 1/35 on the upstream side of the damsite vicinity and approximately 1/6 on the downstream side. The stream has small meanders at some parts.

(2) Foundation Rocks

The foundation rocks of the damsite at the leftbank side consist of Formation ML, Alternation MLS/LS and Formation LS from upstream to downstream white Formation ML is distributed at the right-bank side.

Marl is predominantly distributed in the vicinity where the dam axis is presently being contemplated (Profile D-D in Fig. 5-3).

The strikes of these formations at the left-bank side are roughly parallel to the direction of the axis, while dips are 40°-60° in the downstream direction. There are no outcrops of foundation rock at the right-bank side, and although the strike and dip of the formation cannot be confirmed, it is estimated that they are roughly similar to the left-bank side judging by the conditions of the surrounding area.

According to the results of boring investigations made up to the present at the damsite, the marl has become weakened at the surface by weathering to a depth of 2 m to 10 m, while there are weak portions deeper down due to partial fracturing, and the rock character as a whole is flaky or friable.

(3) Overburden

There is practically no overburden at the left-bank Formation LS and Alternation ML MLS/LS, but the part of Formation ML is covered by 0.1 m to 1 m of overburden.

Colluvial deposits including various-size blocks of limestone breccia consolidated by calcareous matter are distributed relatively widely at the right-bank side of the damsite, the thickness being approximately 15 m at Drill Hole SB-4.

At Profile D-D, there is overburden of thickness 2 to 6 m consisting of topsoil and clay, and at parts, containing sand and graval.

(4) Faults

The existence of a fault crossing roughly perpendicularly with the dam axis direction is anticipated in the vicinity of the damsite due to the discontinuity in distributions of formations at the left and right banks.

Information on the accurate location and character of the fault has not been gained in surface investigations up to the present.

(5) Hydrogeology

There are springs at several locations in the surrounding area of the damsite. These showed discharges of 1-2 ℓ/min per spring to a maximum of 30-40 ℓ/min (the spring from the colluvial deposits at the left-bank side of the damsite) during the rainy season (January-February) of 1978.

According to the results of boring investigations, the water table at the damsite is at a depth of approximately 23 m (Drill Hole SB-1) at the left bank, while at the right-bank side the depth is 2 to 3 m.

To add a note, the little stream flowing at the damsite showed a relatively large volume of surface flow (1-2 t/min) since it was in the middle of the rainy season when the field reconnaissance was carried out.

(6) Permeability

Regarding the drill holes provided at the damsite and its surrounding area, permeability tests were performed by the water supply method for all drill holes during drilling work. On careful examination of the relation between injection pressure P (kg/cm^2) and permeation quantity Q (ℓ/min/m) in these permeability tests, it is possible to surmise the behavior of marl when coming in contact with pressurized water. The permeability of Formation ML comprising the foundation rock of the damsite indicates a value between 10^{-5} cm/sec - 10^{-3} cm/sec. Further, that permeability tests near the surface of marl are relatively small in number indicates that tests could not be performed since hole walls did not possess adequate strength to fix packers.

Further, it is thought that the colluvial deposits distributed at the right-bank side of the damsite is highly permeable as a whole.

The permeability coefficient of the tested portion of the drill hole is computed with the formulae of the United States Bureau of Reclamation,*

$$K = \frac{Q}{2 \pi LH} \cdot \log_e \frac{L}{r} \qquad (1)$$

$$L \geqq 10 r \qquad (2)$$

where

K = coefficient of permeability
Q = constant rate of flow into hole
L = length of portion of hole tested
H = differential head of water
r = radius of hole tested
loge = natural logarithm
* Earth Manual (2nd Edition, 1974), p. 576, U.S. Bureau of Reclamation

5.4.2 Intake

(1) Topography

The intake is planned on the gentle slope of 10°-15° at the northeast foot of Djebel El Fahame at the left-bank side of the upper reservoir damsite in either of the cases of waterway alignments in the two alternatives described later. These gentle slopes change over to the steep east slope of Djebel El Fahama of approximately 30°-35° at higher points.

(2) Foundation Rocks

The foundation rock at the intake site consists of Formation ML, which changes over to Alternation MLS/LS as the powerhouse is approached. An accurate boundary location between the two has not been confirmed from results of investigations made up to this time.

Judging from the investigation results of test pits and Drill Hole SB-7 up to now, it is thought that the surface of Formation ML has been made weak or brittle for a thickness of 5 to 6 m due to weathering.

(3) Overburden

The overburden consists of topsoil and an underlying layer mainly of clay of thickness from 2 m to about 7 m. The clay contains limestone rubble at the upper part with gradual transition from plastic clay to weathered clayey marl or marly limestone.

(4) Hydrogeology

The only data on the ground water table of this site available were those for Drill Hole SB-7, and in the field reconnaissances just made, confirmation was not possible as all of the test pits had been backfilled. According to the recollections of those in charge of the S.T.E.G. investigation works regarding the conditions at the time of test-pit excavation, water tables could be seen at some of the test pits, and there is a possibility that during the rainy season the water table will rise higher than a depth of 6 m.

5.4.3 Waterway, Powerhouse and Outlet

Basically, in this Project, an underground powerhouse will be provided, and regarding the alignment of the connecting waterway, comparison studies have been made of two proposals of Profile A-A = Downstream Alternative and Profile B-B = Upstream Alternative (hereafter tentatively called Alignment A and Alignment B in this text) as indicated in Figs. 5 - 2 and 5 - 3.

(1) Topography

Both of the underground powerhouses of Alignment A and Alignment B are considered for the center of the mountain mass of Djebel El Fahama of peak elevation of 540 m extending narrowly in the SWS direction at the left-bank side of the upper reservoir damsite.

With Alignment A, the outlet tunnel would extend along the west-side foot of the above-mentioned Djebel El Fahama to the vicinity approximately 750 m upstream of the left bank of Kasseb Arch Dam, and the outlet to be provided at the end will be installed at the slope contacting Kasseb Reservoir at a gradient of approximately 45°.

On the other hand, in case of Alignment B, the outlet tunnel from the underground powerhouse would pass under the gentle slope at the west side of Djebel El Fahama and reach the outlet site approximately 800 m upstream along Kasseb Reservoir. The slope in the vicinity of this outlet is gentle at 10° to 15°.

(2) Foundation Rocks

The underground powerhouse sites for both Alignments A and B have been selected so that the greater part would be in Formation LS and a part in Alternation MLS/LS, while the outlet tunnel for Alignment A would not pass through Formation ML.

Regarding the outlet tunnel of Alignment B and the outlet site, Formation ML of not very hard nature is distributed over the whole according to results of investigations at Drill Holes SU-4, 5, 6 and 7.

(3) Overburden

There is practically no overburden at the surface of the mountain mass of Djebel El Famaha. There is distribution of overburden of thickness 3 m to approximately 10 m in the vicinity of the outlet tunnel and the outlet site.

(4) Faults

According to the results of field reconnaissances and photogeological interpretation, numerous faults may be recognized or estimated to exist in the mountain mass of Djebel El Fahama Four or five of these faults were confirmed in the field, and the fractured zones were all less than 1 m in width. The surface of formation ML thought to be distributed at the greater part of the outlet tunnel route of Alignment B is covered by overburden and there is no information regarding the existence of faults.

(5) Hydrogeology

There are no data available concerning the water table of Djebel El Fahama. There is a water table at 65 m underground at Drill Hole SU-7 at the western foot of the mountain (albeit the water level in the hole during the drilling operation), and from the fact that the vicinity consists of Alternation MLS/LS with intercalations of limestone stone, and in consideration of the fact that open cracks (width 20-30 m) thought to be of a type of solution cavity were observed in Alternation MLS/LS at mid-slope on the eastern side of the mountain mass, it is surmised that the water table of this El Fahama mountain mass is relatively low, and moreover, presents a complex distribution state.

Further, it is known that artesian aquifers exist (Drill Hole SU-4 and-5) in Formation ML distributed at the outlet tunnel route of Alignment B.

5.4.4 Upper Reservoir

(1) Topography

The upper reservoir is situated at a small basin formed at high land of El. 400 m - 450 m with the reservoir rim contacting a gentle slope of 5° to 15° A small stream runs down this area from northeast to southwest, and as described in the section on the damsite, this stream has surface water flow during the rainy season and there is some erosion seen at the riverbed portion downstream from the vicinity of the damsite, but within the reservoir area there is very little downcutting due to erosion caused by this stream.

The size of the reservoir, in case high water level is set at El. 430 m will be approximately 850 m in the N-S direction and 600-650 m in the E-W direction for a catchment area of approximately 1.15 km^2.

Further, to the northeast and south of the reservoir there are saddles at El. 452-453 m and El. 433 m, respectively, and the minimum distances at El. 430 m, or pass lengths, of the saddles are approximately 300 m for the former and approximately 400 m for the latter.

(2) Foundation Rocks

There are practically no outcrops of foundation rocks at the upper reservoir or the peripheral slopes contactig the rim with the exception of a portion of the left-bank part of the damsite. Judging from the investigation results of test pits and Drill Hole SB-7, there is a strong possibility that mostly Formation ML is distributed at the reservoir area and its peripheral parts.

(3) Overburden

The greater part of the reservoir area is now farming land, and topsoil and rubble-or gravelbearing plastic-clay are distributed at a thickness of 0.5 to 3.5m. Generalizing from the results of test pits the bottom part of the overburden gradually changes over in most cases to underlying weathered clayed marl.

(4) Faults

It is thought that an assumed fault crossing roughly perpendicularly with the direction of the dam axis extends in the direction of the reservoir also. Other than the above, according to photogeological interpretations, there may be a fault or two extending into the reservoir.

(5) Hydrogeology

There are comparatively abundant data concerning water tables in the rainy season for the reservoir and its peripheral area, but little data for the dry season. For example, water tables have been confirmed at a number of test pits, and considering that there are a number of springs in the surrounding area of the reservoir (mostly higher than high water level of El. 430 m), while there is surface water flow even though slight at every small gully at the ground surface in the area during the rainy season, it is thought the water table of this area rises to a shallow part of underground (shallower than 2 to 3 m) during the rainy season.

(6) Permeability

There is no information on permeability in the reservoir area other than the results of permeability tests at Drill Hole SB-7.

5.5 ENGINEERING GEOLOGICAL ASSESSMENT

5.5.1 Upper Dam Axis

The thick colluvial deposits distributed at the right-bank side of the damsite are unsuitable for a dam foundation, and it will be optimum for the dam axis to avoid these deposits and be placed upstream (near the line connecting the vicinity of Drill Hole SB-1 and Drill Holes SB-2, SB-8 and SB-3).

In such case, it will be reason able to bend the dam axis a little in arch form so long as the geological conditions at the right-bank abutment are good.

5.5.2 Foundation Treatment of Upper Dam

Taking into consideration the results of permeability tests conducted up to the present and the special nature of the upper reservoir that it is a pure pumped-storage reservoir with little discharge of its own, in addition to which its storage capacity is small, thorough cutoff treatment will be required for the dam foundation.

Selection of the foundation treatment method naturally has a close relationship with design of the dam, and hereafter, it will be necessary to aim for collection of further data required, and at the same time, to carry out even more careful studies.

For example, considering the dam foundation in case the dam type is to be rockfill, the estimated excavation line at the impervious core section will be at a depth of about 2 m to 8 m from the present ground surface, and in regard to grouting, curtain grouting of depth of 25 to 30 m from the dam foundation surface at the middle part of the dam will be necessary.

5.5.3 Stability of Upper Dam Foundation

Marl is distributed at the entire upper dam foundation and considering together the property of the rock character to become deteriorated by swelling of marl itself and the condition of rock characters of boring cores, it may be pointed out that there will be a necessity to pay attention hereafter to stability of the foundation rock itself.

5.5.4 Intake Site

The intake site has distributions of overburden consisting mainly of plastic clay 2 to 7 m in thickness, and underlying it wholly weathered soft marl, so that it will be necessary to secure stablity of slopes of cuts near the intake. As for the upper reservoir, since the available drawdown is large and the daily waterlevel variation is great, special care will be required to prevent erosion of cut slopes.

5.5.5 Powerhouse Site

As previously described, an underground powerhouse is being considered for this Project, and in selection of the site, from an engineering geological viewpoint and if permissible from the standpoint of civil engineering design, it will be desirable for the underground powerhouse to be provided in Formation LS which comprises the nucleous of the El Fahama nountain mass. Alternation MLS/LS rocks are distributed on both sides of this Formation LS, and although it may be possible to excavate a powerhouse cavern at these parts also, the geological conditions are inferior to those of Formation LS. As for providing an underground powerhouse in Formation ML, it can never be recommended from an engineering geological standpoint.

5.5.6 Outlet Tunnel and Outlet

The outlet tunnel of Alignment A will pass through the weatern foot of the El Fahama mountain mass shortening the length as much as possible while taking care not to approack Formation ML, and it will be necessary to watch for misalignment at places due to faulting. At the outlet site of Alignment A the ground strata indecate a strike and dep roughly parallel to the slope, and it will be necessary to be careful about exfoliative sliding along bedding planes in excavation work.

The outlet tunnel of Alignment B will pass through Formation ML. According to the results of boring investigations, there are artesian aquifers at places (See Fig. 5 - 4), while the rock charecter is soft, and it will be necessary for maximum prevention measures to be taken against loosening of tunnel walls during excavation work. It may be that a pilot tunnel method of excavation will be required.

There are distributions of overburden consisting mainly of plastic clay 3 to 4 m in thickness and underlying weathered clayey marl at the outlet site of Alignment B. In excavation of the outlet, securing of stability of cut slopes will be the most important problem when the existence of these soft layers and marl of swelling nature, and work at parts lower than the existing reservir water surface are considered. Regarding the considerations to be given to this problem from the standpoint of civil engineering design, there are no divergent views from the various opinions given in previous reports.

5.5.7 Watertightness of Upper Reservoir

Seen from the distribution state of surface water flow at the reservoir area during the rainy season and other factors, it cannot be thought that watertightness of the bedrock of the reservoir is extremely poor, but as previously stated, this reservoir has almost no inflow of itself and is a pure pumped-storage reservoir of small storage capacity, so that leakage from the reservoir area must be prevented to the utmost degree.

In particular, thorough care will need to be exercised regarding permeability of the bedrocks of the two saddles in the topohraphy and the part from the left-bank dam abutment to the intake site.

Further, the entire reservoir area is covered by overburden consisting mainly of plastic clay underlying which there is distribution of weathered clayey marl. These may be looked forward to as serving as a blanket against infiltration flow from the reservoir to foundation rock, and it will be necessary for considerations to be given so as not to disturb these insofar as possible.

5.5.8 Stabilities of Slopes Surrounding Reservoir

Because of the large daily water-level variations of the reservoir, the slopes sround the reservoir which are subjected to water-level fluctuations will show occurrence of gully erosion judging from the geological conditions. It will be necessary for protecteve measures to be considered for these parts.

5.5.9 Construction Materials

Regarding rockfill materials, all materials other than filter materials will be available from the surrounding area of the upper reservoir.

As rockfill materials, the limestone and marly limestone distributed in this area are suitable.

Weathered marl and parts of talus deposits will be usable as core materials.

Natural materials suitable as concrete aggregates do not exist in this project area and its vicinity.

Regarding filter materials, it will be unavoidable for artificial cruching to be considered. Limestone distributed in the area will be suitable as material for crushing.

5.5.10 Further Investigations

In case it is decided that the difinite study of the Kasseb Project is to be implemented, there will be a necessity for the following geological investigation work to be carried out:

(a) Preparation of detailed geological maps of the entire Project Area.

(b) Detailed investigation of the dam foundation bed through test pits and boring.

(c) Excavation of a test adit from the vicinity of the intakes in the headrace direction to the boundary between the marl formation and marl and limestone alternations to ascertain the conditions of ground layers.

(d) Confirmation of the geological structure at the underground powerhouse location through boring.

(e) Investigation of the geological conditions in the outlet vicinity in accordance with the outlet lacation decided.

(f) Geological investigation of upper reservoir foundation bed through boring and investigation of time-dependent change in ground water distribution.

(g) Excavation of test pits southeast of the upper reservoir (the area 200 to 800 m from the reservoir rim) and south of the reservoir (the area 1,000 to 1,500 m from the reservoir rim) with the purpose of investigating impervious core material, preparation of geological profiles, and laboratory tests of materials.

5.6 EARTHQUAKE

As the design seismic coefficient, the value of 0.1 is to be adopted based on past records and the estimated probable maximum earthquake intensity, and in consideration of degrees of importance of structures and economic efficiency.

According to a communication regarding earthquake records of Tunisia from the Institut National de la Météorologie, Division de la Géophysique to the Department of Construction of STEG in December 1975, the following is stated regarding the area within a radius of 30 km with Beja at the center.

The number of earthquakes recorded from the beginning of the century up to the present is relatively large with a total of 61 having been felt at communities here and there. At a number of communities such asThibar and La Merja, there have been earthquakes of intensities of 8 to 9 on the M.S.K. scale. At the city of Beja itself 6 earthquakes have been felt during this century, the intensities having been 3 to 5 according to M.S.K. standards.

It may be concluded in consideration of statistical data that the probable maximum earthquake intensity will be 8 to 9 on the M.S.K. scale.

Tableau 5-1 LISTE DES TROUS DE FORAGE

Hole No.	Location	Co-ordination X Y	Top Elevation (m)	Length of Hole (m)	Direction of Hole	Thickness of Overburden (m)	El. of Bed-rock Surface (m)	Length of Casing Pipe (m)	Core Recovery (%)	Diameter of Hole (mm)	Rock Type of Bed Rock	Commenced / Completed	Remarks
SB-1	Dam, left bank.	X 87,028.91 Y 78,642.17	420.77	35.0	Vertical					NX(0m~350m)		24-Jan.-1975 27-Feb.- 〃	
SB-2	Dam, river bed.	X 87,108.06 Y 78,642.00	393.00	35.0	do.					NX(0m~350m)		6-Feb.- 〃 1-Mar.- 〃	
SB-3	Dam, right bank.	X 87,268.78 Y 78,643.46	422.82	35.0	do.					NX(0m~350m)		12-Apr.- 〃 8-May- 〃	
SB-4	Dam, right bank.	X 87,184.80 Y 78,759.93	419.61	35.0	do.					NX(0m~350m)		2-Mar.- 〃 4-Apr.- 〃	
SB-5	Dam, river bed.	X 87,103.80 Y 78,736.94	393.99	35.0	do.					NX(0m~350m)		16-Mar.- 〃 6-Apr.- 〃	
SB-7	Dam, left bank.	X 87,015.88 Y 78,543.96	407.54	30.0	do.					NX(0m~30.0m)		23-Nov.- 1974 19-Dec.- 〃	
SB-8	Intake. (left bank)	X 87,184.70 Y 78,642.94	404.94	35.0	do.					NX(0m~350m)		29-Dec.- 〃 27-Jan- 1975	
			Sub-total	240.0									
SU-1	Lower reservoir side(outlet)	X 86,022.03 Y 79,641.05	298.57	70.0	Vertical					NX(0m-48.65m) BX(48.65m~70.0m)		27-Nov.- 1974 13-Feb.- 1975	
SU-2	Lower reservoir side(outlet)	X 85,985.16 Y 79,429.74	339.79	38.5	do.					NX(0m~38.5m)		25-Jan.- 〃 11-Mar.- 〃	
SU-4	Lower reservoir side(outlet)	X 86,242.47 Y 79,516.81	310.22	100.0	do.					(mm) ø 107.95(0m~1.6m) ø 147.00(1.6m-8.5m) ø 110.0 (8.5m~28m) NX(28m~100m)		15-Mar.- 1975 16-Jun.- 〃	
SU-5	Lower reservoir side(water way)	X 86,319.87 Y 79,400.52	330.00	100.0	do.					NX(0m~100.0m)			
SU-6	Lower reservoir side(water way)	X 86,413.12 Y 79,260.43	350.14	125.0	do.					NX(0m~122.2m) BX(122.2m~125m)		6-Jun.- 〃 19-Aug- 〃	
SU-7	Lower reservoir side(power house)	X 86,502.77 Y 79,125.87	380.24	75.0	do.					NX(0m~75.0m)		1-Jul.- 〃 14-Aug.- 〃	
			Total	748.5									

Tableau 5-2 LISTE DES PUITS DE SONDAGE (1-2)

Pit No.	Top Elevation(m)	Depth (m)	Location	Geological Log (0m – 6m → Depth)	Remarks
P 2	421.47	5.20	Upper dam site, right bank	Small gravel in MLY. soil / 1.0 Small gravel in gry. yel. or wht. MLY. soil / 3.0 Boulders (ø 0.5m) with clay and flat LS. frgs. / 5.2m	
P 3	433.11	4.30	do.	LS. gravel in clay / 0.5 Blk. or brn. MLY. soil with gravel (ø 5~20cm) / 2.6 Small LS. gravel in brn. clay / 4.3m	
P 4	426.25	5.60	do.	Compacted clay with LS. fragments / 4.2m Compacted fine-grained clay / 5.6m	
P 5	420.10	6.00	do.	Black compacted clay with small gravel / 2.5 Compacted clay with LS. frgs. / 3.8 LS. boulders (ø 0.5~1m) with brn. clay mortar / 6.0m	
P 6	412.50	5.50	do.	Farming soil with small LS. gravel / 2.0 Weathered, disturbed and schistosed / 5.0 ML. — pale blue, schistosed / 5.5m	
P 7			do.		No data
P 8	426.25	5.60	do.	Farming soil / 0.5 Weathered, schistosed ML. / 2.2 Pale blue ML. with cemented calc-part / 5.6m	Water table at 4.2m
A 1/2	468.95	3.65	South-east of upper reservoir	Ts. and slope wash / 1.0 Yel. fine grained sand and brn clay with soft LS. gravel / 3.2 ML. — weathered and friable yel-brn. / 3.65m	Ts.=0.2m Water table at 3.3m
A 3/2	449.81	3.50	do.	Ts and brn. organic plastic clay / 1.4 Yel.-brn. ML. with some joints / 3.5m	Ts.=0.2m Water table at 3.2m
A 5/2	431.50	2.00	do.	Ts and brn. organic plastic clay / 1.2 Gry. ML. — with speckles / 2.0m	Ts.=0.2m Wet at bottom
A 1/4	459.93	4.30	do.	Ts and gry organic plastic clay / 1.2 Yel. speckled, plastic clay with calc-part / 3.5 Stratified gry. ML. / 4.3m	Ts.=0.2m
A 3/4	444.94	4.00	do.	Ts. and brn. organic, plastic clay / 2.0 Gry. speckled ML. / 4.0m	Ts.=0.2m Wet at bottom
B 1	426.74	3.80	Upper dam site and reservoir, right bank	Ts. and yel. sandy clay with gravel and breccia / 2.35 Co. deposits, clayey in lower part / 3.8m	Ts.=0.25m
B 2	426.71	2.25	do.	Ts. and yel. clay with calc-part, and weathered yel ML. / 2.25m	Ts.=0.2m
B 3	432.57	2.00	do.	Ts. and organic plastic clay with gravel / 1.4 Yel. ML. — calcite veins along joints / 2.0m	Ts.=0.2m Wet at bottom
B 4	454.16	2.00	do.	Ts. and yel.-brn. organic, plastic clay / 1.4 Brn. ML. — weathered but not clayey / 2.0m	Ts.=0.2m
C 1	459.68	2.00	North-east of upper reservoir	Ts. and organic plastic clay / 1.0 Gry ML. with calcite veins / 2.0m	Ts.=0.2m
C 2	437.48	2.30	do.	Ts. and organic, plastic clay / 1.4 Brn. ML. not clayey / 2.3m	Ts.=0.2m
C 3	442.11	1.70	do.	Ts. and compact ML. / 1.0 Calc-ML. — with many calcite veins / 1.7m	Ts.=0.2m
C 4	444.81	2.70	do.	Ts. and organic plastic clay / 1.2 Yel.-brn. MLY. clay / 2.0 Brn. ML. — with calcite veins along joints / 2.7m	Ts.=0.2m
C 5	450.60	1.40	do.	Ts. and plastic clay / 0.8 Yel.-brn. ML. / 1.4m	Ts.=0.2m
D 1	442.42	2.00	South of upper reservoir, saddle part	Ts. and organic clay, wet / 1.0 Weathered gry. ML. / 2.0m	Ts.=0.2m Wet at bottom
D 2	436.74	1.60	do.	Ts. and organic plastic clay / 1.2 ML. — yel.-brn. speckled and with calcite veins / 1.6m	Ts.=0.2m
D 3	431.57	2.00	do.	Ts. and organic clay / 0.8 Yel.-gry. clayey ML. — with calc-nodules, LS. gravel and boulders / 2.0m	Ts.=0.2m
D 4	436.36	3.20	do.	Ts. and organic, wet, plastic clay / 1.8 Weathered, blk. ML. — with calcite veins / 3.2m	Ts.=0.2m

Abbreviations	
Ts.	Topsoil
LS.	Limestone
ML.	Marl
MLS.	Marly limestone
SLS.	Sandy limestone
MLY.	marly
frgs.	fragments
Yel.,yel	yellow
Brn.,brn.	brown
Gry.,gry	grey
Blk.,blk	black
Wht.,wht.	white
Co.	colluvial

Tableau 5-2 LISTE DES PUITS DE SONDAGE (2-2)

Pit No.	Top Elevation (m)	Depth (m)	Location	Geological Log (Depth 0m 1 2 3 4 5 6m)	Remarks
D 5	435.38	2.50	South of upper reservoir, saddle part	Ts. and wet MLY. clay | Clayey ML. 1.5 wet | with calc-nodules 2.5m	Ts.= 0.2m
D 6	433.72	1.45	do.	Ts. and Clay 0.65 | Wet ML. 1.45m | with calcite veins along joints	Ts.= 0.2m
H 1		5.00	Upper reservoir, intake site	Ts. and blk. clay with rubble | Light brn. clay 1.2 with calc-part | MLS. rubble 2.6 | Hard gry. MLS. rubble 3.4 | 5.0m	Ts.= 0.3m
H 2		5.00	do.	Ts. and blk clay with LS. rubble 1.2 | "Slide" clay with Hellix | Yel-brn. 3.3 clay | Weathered 4.1 MLS. | 5.0m	Ts.=0.3m
H 3		5.70	do.	Ts. and "Slide" clay | Weathered 4.2 clay | Weathered 4.9 MLS. | 5.7m	Ts.=0.3m
H 4		5.90	do.	Ts. and blk. clay with rubble 1.5 | Compacted "slide" clay soft clay at 5.2m | MLY. 5.3 clay | weathered 5.8m	Ts.=0.3m
H 5		6.30	do.	Ts and organic clay with LS. rubble 3.0 | Weathered "slide" clay with brn. LS gravel. | 6.3m	Ts.=0.3m
H 6		6.80	do.	Ts. and blk. clay with LS. rubble 2.4 | "Slide" clay with potholes 3.4 | "Slide" clay | 6.8m	Ts.=0.3m
H 7		4.80	do.	Ts and slightly or moderately weathered LS. Alternation of hard layer and soft layer | 4.8m	Ts.=0.1m
H 8		4.00	do.	Ts. and gry. clay with Hellix, "slide" clay 2.2 | Weathered MLS. clayey with hard SLS. beds | 4.0m	Ts.=0.1m
H 9		3.80	do.	Ts. and blk. clay with rubble 2.6 | Weather MLS. with slip plane | 3.8m	Ts.=0.1m
H 10		2.00	do.	Ts. and light brn. "slide" clay | 2.0m	Ts.=0.2m
H 11		2.80	do.	Ts. and clay with rubble 0.9 | Light brn clay with lots of with shell frgs. (Hellix) | 2.8m	Ts.=0.4m
H 12		3.50	do.	Ts. and clay with rubble 1.1 | Clay with calc-nodules 2.0 | Gry-brn. clay 2.7 | Weathered MLS. | 3.5m	Ts.=0.4m
H 13		2.80	do.	Ts. and "slide" clayey ML. 1.1 | Yel-gry "slide" clay | 2.8m	Ts.= 0.3m
H 14		3.00	do.	Ts. and "slide" clay with rubble 1.3 | Plastic weathered substratum | 3.0m	Ts.= 0.3m
H 15		4.00	do.	Ts. and "slide" Clayey ML. 1.4 | Yel.-gry. clay 2.1 | Weathered substratum 3.2 | Gry. substratum | slightly weathered | 4.0m	Ts.= 0.4m
K 1		0.70	Upper dam site, right bank	Ts. and substratum | weathered 0.7m	Ts.=0.2m
K 2	420.20	2.70	do.	Ts. and organic clay with rubble 1.7 | Gry.-brn. substratum | weathered 2.7m	Ts.=0.3m
K 3	420.18	1.50	do.	Ts. and organic clay with rubble 1.4 | Weathered substratum, clayey 1.5m	Ts.=0.3m
K 4	428.63	0.60	do.	Ts. and blk. rubble | 0.6m	Ts.=0.3m
K 5	429.08	1.90	do.	Ts. and blk. clay with rubble 1.2 | Clay 1.5 | Gry-brn MLS | 1.9m	Ts.=0.3m
K 6	429.81	2.30	do.	Ts. and clay with rubble. Clay rich in lower part 1.9 | Weathered substratum | 2.3m	Ts.=0.3m
L 1		4.90	Lower reservoir outlet site	Ts. and "slide", weathered substratum generally compact | Weathered MLS. with glauconitic sandstone | 4.9m	Ts.= 0.3m
L 2		5.00	do.	Ts. and MLY. soil 1.1 | Weathered substratum (MLS.) 2.6 | Slightly weathered substratum | 5.0m	
L 3		2.00	do.	Ts and MLY. soil 0.9 | Yel.-brn. clay | 2.0m	

Abbreviations

- Ts. Topsoil
- LS. Limestone
- ML. Marl
- MLS. Marly limestone
- SLS. Sandy limestone
- MLY. marly
- frgs. fragments
- Yel., yel. yellow
- Brn., brn. brown
- Gry., gry. grey
- Blk., blk. black
- Wht., wht. white
- Co. colluvial

CHAPTER 6. HYDROLOGY

CHAPTER 6 HYDROLOGY

6.1 GENERAL

The report, "Barrage sur l'oued Kasseb-Etudes de l'éfficacité économique de la construction de la centrale hydroélectrique," prepared by USSR in 1963 for construction of the existing Kasseb Dam, contains a detailed hydrological analysis carried out based on observation data from several meteorological observation stations and B-11 Runoff Gauging Station in the area in which the project site is situated.

Observation data on runoff are available for a period of 13 years from 1948 to 1961. However, from 1961, adequate runoff data on the Kasseb River do not exist.

In the present Kasseb Pumped Storage Power Generation Project Feasibility Study, the basic data concerning hydrology have mainly been taken from the abovementioned USSR Report.

For reference, to indicate the locations and the periods of observation of the above mentioned meteorological observation stations and the runoff gauging station, they are as given in Fig.6.1.

6.2 PRECIPITATION

The precipitation in the Kasseb River catchment area consists essentially of rainfall only, with the rainy period starting in September and ending in April of the following year.

According to the USSR Report, the average annual precipitation is 1,534 mm at Ain-Draham Observation Station, and 626 mm at Baja Observation Station. The average annual precipitation of the Kasseb Basin to the damsite (catchment area 101 km^2) is estimated to be approximately 800 mm.

Maximum diurnal precipitations have been recorded to be 128 mm at Tunis Gauging Station (September 14, 1935), 124 mm at Beja Observation Station (June 4, 1915), and 170 mm at Ain-Draham Observation Station.

The monthly precipitations of the Beja and Ain-Draham observation stations according to the U.R.S.S. Report are indicated in Fig.6.2.

6.3 INFLOW

6.3.1 Inflow at Lower Reservoir

Water impoundment was started at the existing Kasseb Dam from February 1969.

As records prior to construction of the dam, B-11 Runoff Gauging Station was installed immediately downstream of the dam by the Tunisian Ministry of Agriculture, and diurnal inflow was observed for a period of 12 years, 1948/49, 1950/51 -1960/61.

Further, in the Feasibility Study Report of U.R.S.S., the average monthly runoff of the Kasseb River was estimated utilizing the close correlation between the inflow records of the B-11 site on the Kasseb River for the 6 years of 1943/44 - 1947/48 and 1949/50, and the inflow records of El Lile River Runoff Gauging Station installed in the vicinity of Ben Metir.

On the other hand, regarding the inflow after construction of the dam, there are data on average monthly runoffs prepared by the Ministry of Agriculture during the period from February 1968 to August 1977.

Since the results of study of the discharge rate from the amount of precipitation showed figures high in scatter, the previously-mentioned runoff data after construction of the dam out of the above data were iliminated from the examination data for this Report.

Consequently, as the inflow data of the Kasseb River, the inflow data for the 18-year period of September 1943 to August 1961 as shown in Table 6.1 (average annual inflow 1.66 m^3/sec) will be utilized for the hydrological analysis in this Report.

6.3.2 Inflow of Upper Reservoir

Regarding the inflow at the upper reservoir, in view of the fact that the catchment area is extremely small at 1.3 km^2, and the surface is covered with relatively impermeabile soil, it may be considered that practically all of the rainfall will flow into the reservoir.

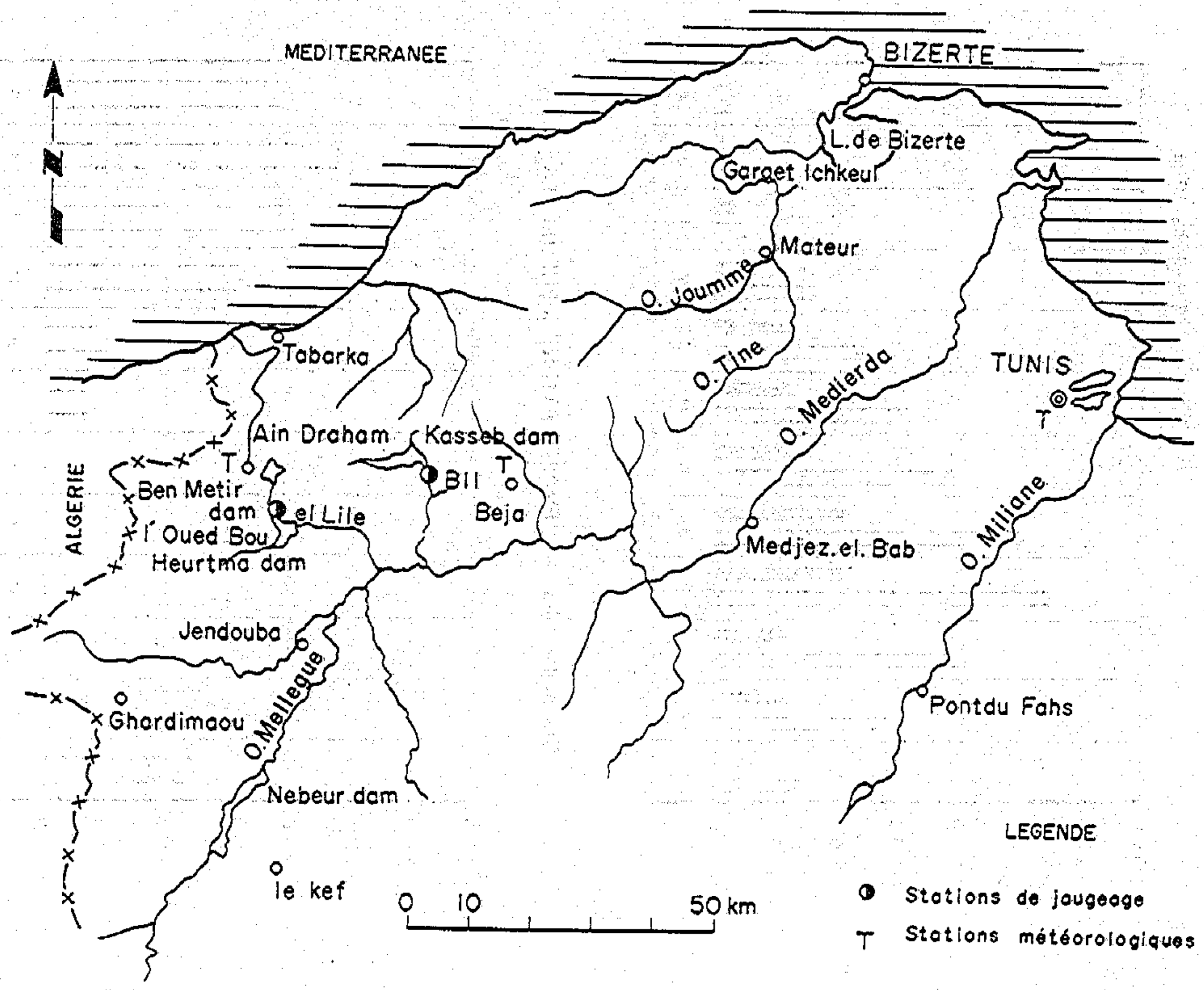
Therefore, since the annual rainfall of this catchment area is estimated to be 800 mm, the annual discharge will be 1.04×10^6 m^3 (0.033 m^3/sec).

6.4 FLOOD DISCHARGE

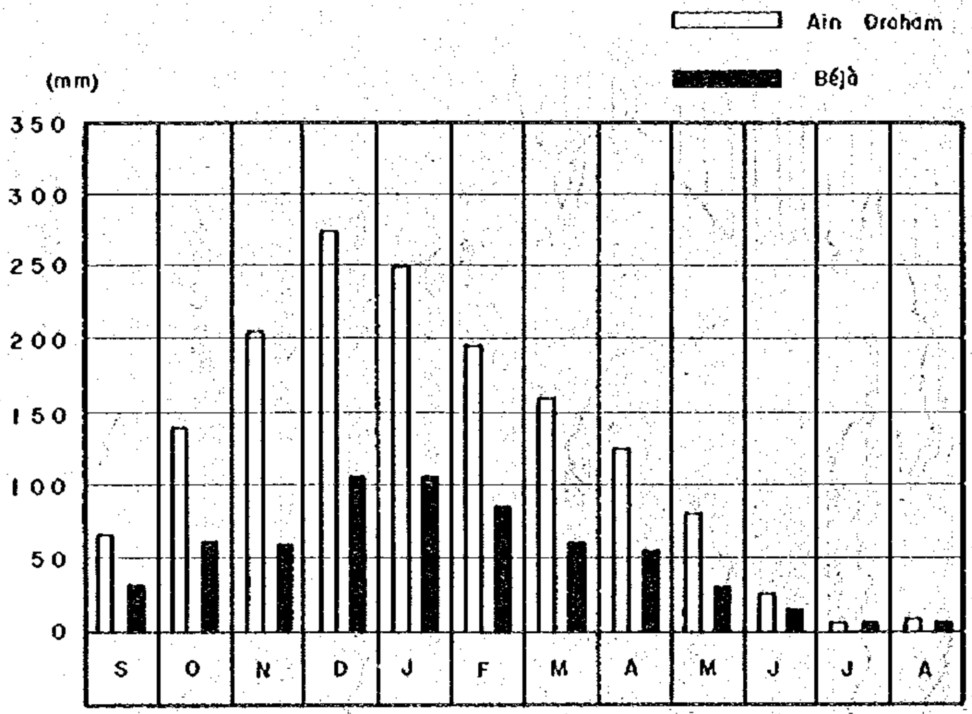
6.4.1 Flood Discharge of Lower Reservoir

Regarding the past flood discharge of the lower reservoir, there are the observed values of maximum annual discharges for 1948 to 1961 at the B-11 site, and these are as tabulated in Table 6.2.

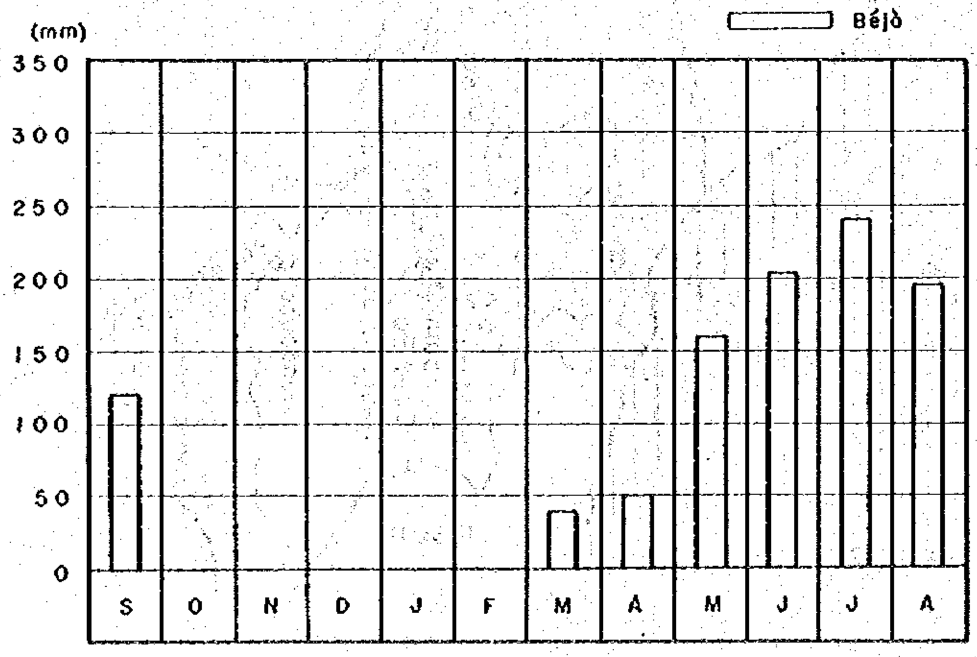
Graphique 6-1 STATIONS METEOROLOGIQUES ET STATIONS DE JAUGEAGE



Graphique 6-2 PRECIPITATIONS MENSUELLES



Graphique 6-3 EVAPORATIONS MENSUELLES



Source : Barrage-voûte sur l'oued Kasseb - Plans U.R.S.S. Moscou-1964 Design N° 112197

Tableau 6-1 DEBITS D'APPORT MENSUELS A LA STATION B-11 DE L'OUED KASSEB

Mois	Année	1943-44	1944-45	1945-46	1946-47	1947-48	1948-49	1949-50	1950-51	1951-52	1952-53	1953-54	1954-55	1955-56	1956-57	1957-58	1958-59	1959-60	1960-61
Septembre	1	-	-	-	-	-	0,05	-	-	0,022	0,069	0,098	0,062	0,29	0,043	0,066	0,043	0,083	0,072
	2	-	-	-	-	-	0,23	-	-	0,051	0,070	0,089	0,055	0,38	0,075	0,069	0,041	0,085	0,091
	3	-	-	-	-	-	0,12	-	-	0,050	0,072	0,075	0,051	2.79	0,17	0,065	0,041	0,056	0,11
	Moyenne	0,034	0,18	0,10	0,012	0,017	0,14	0,015	0,058	0,041	0,070	0,087	0,056	1,15	0,096	0,067	0,042	0.075	0,091
Octobre	1	-	-	-	-	-	0,10	-	-	0,19	0,069	0,38	0,048	5,40	0,12	0,25	0,043	0,35	0,12
	2	-	-	-	-	-	0,10	-	-	2.95	0,059	0,12	0,064	1,32	0,074	0,19	0,048	0,26	0,12
	3	-	-	-	-	-	0,21	-	-	2,45	0,060	4,71	0,055	3,97	0.098	1,31	1,77	0,52	0,10
	Moyenne	0,20	0,40	0,038	0,058	0,12	0,14	0,048	0,056	1,88	0,063	1,83	0,056	3,58	0,099	0,61	0,66	0,38	0,11
Novembre	1	-	-	-	-	-	2,46	-	0,58	0,53	12,1	5,11	0,045	1,28	0,26	0,63	3,20	2,52	0,083
	2	-	-	-	-	-	7,50	-	0,81	0,89	1,78	2,06	1,94	1,61	0,45	2,76	2,63	0,65	0,079
	3	-	-	-	-	-	1,02	-	0.37	8,37	1,29	2,71	0,59	2,33	0,34	4,11	4,24	0,54	0,085
	Moyenne	0,41	0,78	0,090	0,54	0,18	3,66	4,41	0,59	3,26	5,06	3,29	0,86	1,74	0,35	2,50	3,36	1,24	0,082
Décembre	1		-	-	-	-	1,15	-	0,35	1,08	12,3	0,94	0,24	0,79	13,4	2,68	2.23	6,07	0,086
	2	-	-	-	-	-	4,56	-	2,15	0,66	3,19	0,46	11,1	0,43	1,25	8,50	1,63	3,42	1,12
	3	-	-	-	-	-	4,10	-	2,26	7,07	2,39	5,93	2,57	8,66	2,13	9,91	2,63	3,55	2,44
	Moyenne	0,51	4,22	3,08	20,2	2,30	3,30	1,86	1,61	7,07	5,85	2,56	4,57	3,47	5,48	7,12	2,18	4,32	1,25
Janvier	1	-	-	-	-	-	4,05	-	5,11	3,25	3,52	13,7	2,50	9,06	7,96	5,17	8,09	0,99	1,24
	2	-	-	-	-	-	11,4	-	3,72	7,71	6,30	10,3	2,63	2,24	3,28	14,4	6,87	11,1	2,62
	3	-	-	-	-	-	2,64	-	3,25	6,45	8,64	3,22	2,91	2,52	12,6	2,55	1,52	2,53	7,38
	Moyenne	1,71	8,20	2,53	4,47	2,22	5,93	5,60	4,00	5,82	6,23	8,88	2,69	4,54	8,10	7,39	5,36	4,80	3,86
Février	1	-	-	-	-	-	6,01	-	6,60	7,61	2,01	4,31	5,00	24,8	1,99	1,34	0,87	0,92	3,14
	2	-	-	-	-	-	2,72	-	2,40	3,87	2,88	13,8	1,97	10,95	0,75	0,82	0,72	0,68	0,92
	3	-	-	-	-	-	1,46	-	1,67	2,71	1,73	9,79	1,38	2,18	0,52	0,73	2,80	0,44	0,57
	Moyenne	9.42	2,89	1,82	2,70	1,74	3,54	2,62	3,69	4,80	2,24	9,26	2,88	13,0	1,13	0,98	1,37	0,69	1,61
Mars	1	-	-	-	-	-	7,04	-	2,94	1,35	1,56	1,86	3,05	1,09	0,38	7,44	1,00	0,33	0,62
	2	-	-	-	-	-	3,90	-	0,94	2,12	5,44	2,27	3,46	0,81	0,32	2,26	14,7	1,21	0,44
	3	-	-	-	-	-	2,53	-	0,54	0,96	1,04	1,69	0,50	3,86	0,34	0,77	1,86	0,53	0,30
	Moyenne	3,52	3,05	4,27	0,26	1,04	4,43	5,19	1,44	1,46	2,63	1,03	2,28	1,98	0,35	3,40	5,72	0,68	0,45
Avril	1	-	-	-	-	-	3,10	-	0,38	0,66	0,55	2,62	0,95	0.79	1,32	0,61	2,27	0,50	0,23
	2	-	-	-	-	-	1,15	-	0,26	1,70	0,53	0,82	0,35	0,46	1,87	0,50	3,43	1,14	0,22
	3	-	-	-	-	-	0,60	-	0,22	2,34	0,36	1,49	0,67	0,34	0,64	0,46	5,59	0,53	0,21
	Moyenne	1,13	0,16	0,45	1,02	1,51	1,61	4,27	0,29	1,23	0,48	0,48	0,66	0,53	1,28	0,54	3,76	0,72	0,22
Mai		0,22	0,059	0,24	0,058	0,57	0,25	0,68	0,16	0,42	0,73	0,13	0,24	0,47	0,70	0,25	0,57	2,91	0,17
Juin		0,034	0,16	0,026	0,008	0,15	0,15	0,22	0,086	0,16	0,22	0,087	0,096	0,12	0,27	0,088	0,38	0,31	0,088
Juillet		0,033	0,020	0,012	0,007	0,010	0,094	0,10	0,045	0,095	0,12	0,063	0,065	0,084	0,14	0,060	0,13	0,16	0,054
Août		0,033	0,007	0,012	0,028	0,010	0,11	0,024	0,400	0,048	0,17	0,063	0,062	0,051	0,085	0,046	0,10	0,095	0,038
Moyenne annuelle		1,41	1,68	1,06	2,47	0,82	1,94	2,08	0,99	1,85	1,99	2,45	1,20	2,50	1,52	1,94	1,98	1,38	0.668

Source : Barrage-voûte sur l'oued Kasseb
U.R.S.S. Moscou - 1963

Tableau 6-2 **DEBITS MAXIMA DE L'OUED KASSEB**

Année	Débit maxima (m^3/sec)	Date
1948	65	11 novembre
1949	90	13 janvier
1951	109	29 janvier
1952	55	6 décembre
1953	167	15 mars
1954	90	14 décembre
1955	78	25 décembre
1956	100	2 décembre
1957	115	30 décembre
1958	66	1 mars
1959	175	avril
1960	35	7 mai
1961	54	27 janvier
Moyenne	92	

The maximum annual discharge during the 13 years indicated in the above table is 175 m^3/sec.

However, the above observation period is an extremely short one and inadequate for estimating maximum discharge of low probability. Because of this, analysis of the maximum discharge of the Kasseb River for the cases of probability P = 1. Y and P = 5. Y was made by an indirect technique utilizing the heavy rainfall records of Tunis Meteorological Observation Station.

These probability maximum discharges were estimated using the formula adopted at Bureau de Cadastre des Ressources Hydrauliques of Tunisia and the figures are the following:

Probability	Maximum Discharge
0.1%	1,050 m^3/sec
1 "	700 "
5 "	500 "

The existing Kasseb Dam was designed and constructed taking the above maximum discharges into consideration.

6.4.2 Flood Discharge of Upper Reservoir

The flood discharge of the upper reservoir, determined using Creager's Equation and based on the same coefficients as for the flood discharge of the lower reservoir will be the following:

Creager's Equation

$$Q = 46.C.A^{(0.894A^{-0.048})}$$

Where

Q : Flood discharge, ft^3/sec
A : drainage basin area, mil^2 Upper reservoir 1.24 km^2 = 0.478 mil^2
Lower reservoir 101 km^2 = 38.98 mil^2
C : coefficient depending upon characteristic of drainage basin

Probability	Maximum Discharge
0.1%	34 m^3/sec
1 "	23 "
5 "	16 "
(Past Maximum)*	5.7 "

* Note: By past maximum is indicated the maximum annual discharge of the lower reservoir, 175 m^3/sec, during the 13-year period.

6.5 EVAPORATION

The evaporation from the ground durface is measured by the rainfall observation network of the Kasseb Basin. From the average annual precipitation and the average long-time discharge during the 5-year period from 1946 to 1950, the average annual evaporation is estimated to be 740 mm.

Meanwhile, with regard to evaporation from the surface of water, the USSR Report calculates it to be 1,790 mm based on Tunis-Aouina observation data and using its own special formula.

Therefore, it is estimated that the additional evaporation at the project area is 1,790 - 740 ≒ 1,000 mm.

The monthly losses due to evaporation are as indicated in Table6.3,and the evatorations of neighboring sites in Fig. 6. 3.

Tableau 6-3 EVAPORATIONS MENSUELLES

Mois	Evaporation (mm)
Septembre	120
Octobre	0
Novembre	0
Décembre	0
Janvier	0
Février	0
Mars	40
Avril	50
Mai	160
Juin	200
Juillet	240
Août	190
Annuelle	1.000

Consequently, the loss from evaporation from the existing Kasseb Reservoir was calculated to be approximately 2.9×10^6 m^3, and this value was taken into consideration in determining the plan for the reservoir.

CHAPTER 7. EXAMINATION OF DEVELOPMENT PLAN

CHAPTER 7 EXAMINATION OF DEVELOPMENT PLAN

7.1 BASIC CONDITIONS

The development plan will be studied under the following conditions.

(1) This Project is to be for a pure pumped-storage power station with the reservoir formed by a dam constructed at a gorge between Djebel Bou Sttar and Djebel El Fahama as the upper reservoir, and the existing Kasseb Reservoir as the lower reservoir. As inflow to the upper reservoir is extremely small, it is to be ignored in energy production calculations.

(2) The high water level of the upper reservoir is to be considered at El. 430 m. This is because geological investigations of the existing saddles (El. 433 m and El. 452-453 m) have not yet been carried out so that it is not possible at this time to confirm the economic soundness of further raising the water level by constructuing dikes at these parts.

(3) The effective storage capacity of the upper reservoir is to be of a size sufficient to cope with daily peak load duration time of 4 hours.

(4) Discharge from the lower reservoir, according to the water supply plans of SONED is to be maintained constant at 1.25 m^3/sec and 1.40 m^3/sec (from the summer of 1978), and the available drawdown of the lower reservoir is to be determined based on these plans.

(5)* The high water level of the existing Kasseb Dam (lower reservoir) is at El. 288.6 m. This elevation, according to the USSR Kasseb Feasibility Report was 291.0 m, but in a subsequent investigation ., it was confirmed that 288.6 m is the correct elevation.

(6) The available drawdown is to be the two depths of 20 m (the elevation difference between the dam spillway crest and the bed of the lower intake) adopted in the existing plans, and 15 m providing an allowance over the long-term maximum fluctuating water depth obtained as a result of examining water-level fluctuations based on the lower reservoir water supply plan of Item (4) above.

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* High Water Level of Existing Kasseb Dam

(a) The figures regarding high water level and storage capacity are inexact as shown below.

Item	USSR	Ministry of Agriculture	SONED	TECSULT
High Water Level (m)	291.00	292.00	282.20	288.53
Storage Capacity ($10^6 m^3$)	82.00	81.70	82.57	Read from water levelstorage capacity curve

(b) According to the figure, "Barrage Kasseb Auscultation Echelle 1/2000 Reseau des Reperes, Reference A.M. 1-1-140," there are the datum points RN3 and RN1 of N.G.T. near the left and right-bank abutments of the dam, and the station elevation on the road bridge of the dam is 284.13 m.

(c) The Survey Team proposes the Kasseb lower reservoir high water level at El. 288.6 m corresponding to the topographical maps of 1/5,000 and 1/500 scale based on Item (b) and Barrage Kasseb Auscultation. The reasons for this are the following:

	U.S.S.R.	N.G.T.
Dam Crest Elevation	296.5	294.13
High Water Level	291.0	Z

$Z = 294.13 - (296.5 - 291.0) = 288.63 \fallingdotseq 288.6$

(d) STEG concurred in the proposal of Item (c) by telex dated April , 1978.

7.2 EXAMINATION OF LOWER RESERVOIR AVAILABLE DRAWDOWN

7.2.1 Preconditions

The reservoir preration of the lower reservoir is to be calculated according to the conditions below.

(1)* Examination of reservoir water level fluctuation based on monthly runoffs during the 18-year period from September 1943 to August 1961 indicated in Table 6.2.

(2) Examinations of the two cases of discharge for water supply throughout the year at a constant 1.25 m^3/sec and of discharge at 1.40 m^3/sec.

(3) High water level is to be at El. 288.6 m, which corresponds to the high water level of 291.0 m indicated in the USSR Kasseb Feasibility Study Report.

(4) Assuming the annual average water level of the existing Kasseb Reservoir for the end of July to be the water level for September 1943, the water-level variation of the reservoir under Item (1) is calculated.

* USSR Kasseb Feasibility Study Report, Table 7.

Figure 7-1 WATER LEVEL OF THE LOWER RESERVOIR
— Discharge : 1.25 m³/sec —

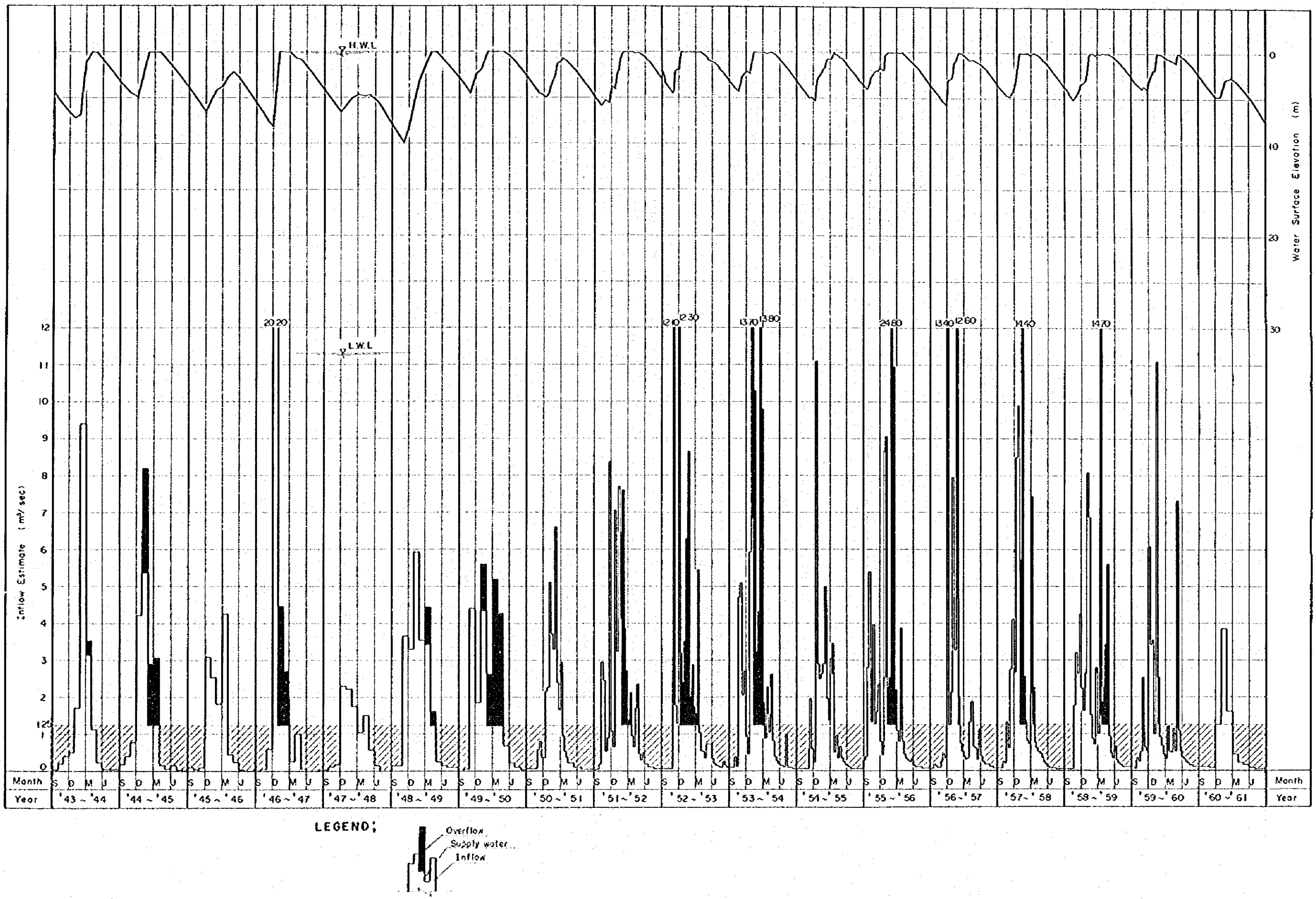
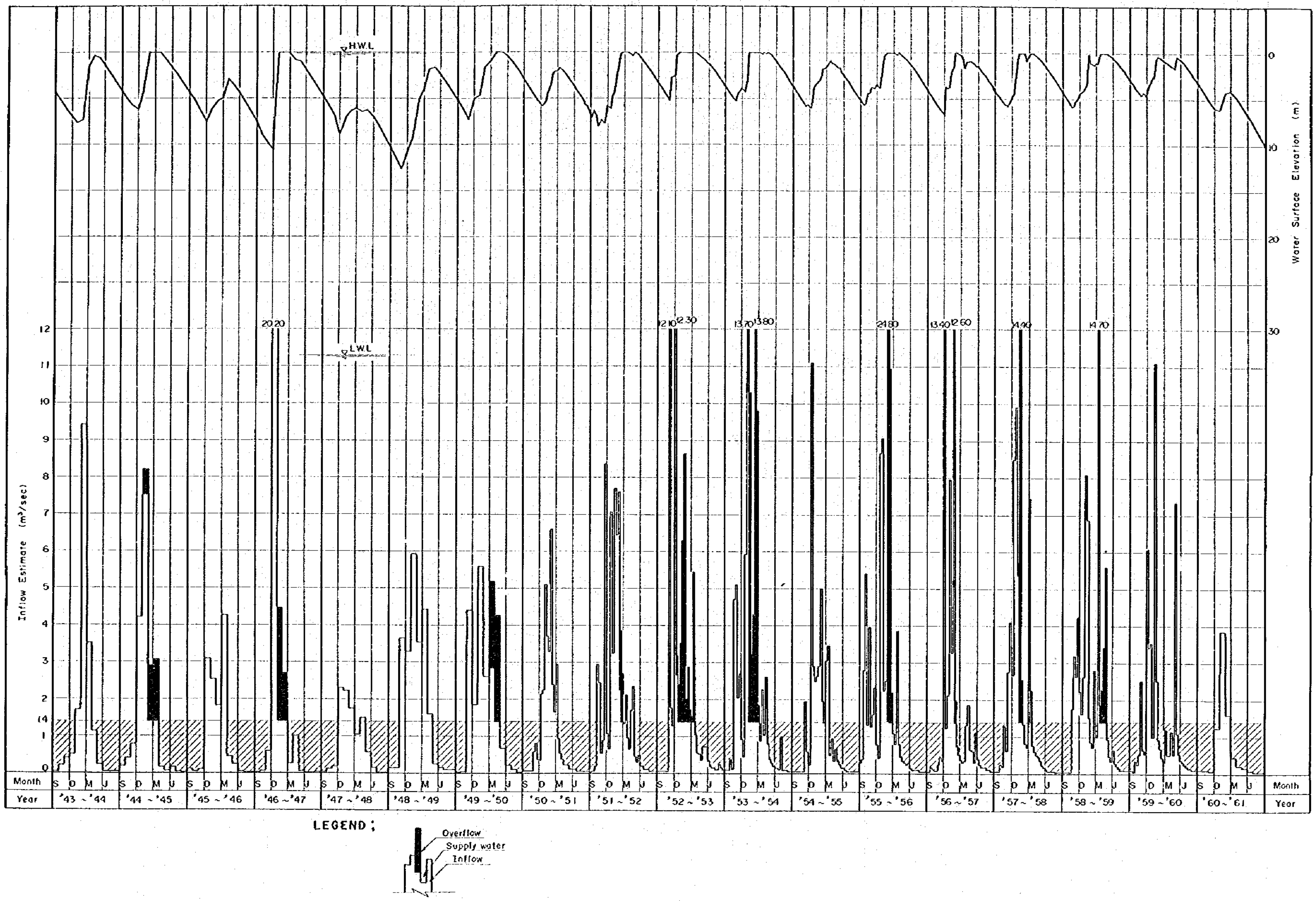


Figure 7-2 WATER LEVEL OF THE LOWER RESERVOIR

— Discharge : 1.40 m³/sec —



7.2.2 Conclusions

The reservoir operating water level and inflow for the 18 years calculated based on the above conditions are as shown in Figs. 7.1 and 7.2.

The long-term minimum water level for the above is as indicated below.

Reservoir Operation Minimum Water Level

Case	Discharge (m^3/sec)	Maximum Drop from High Water Level (m)
1	1.25	10.0
2	1.40	12.7

7.2.3 Determination of Available Drawdown

The available drawdown of the reservoir is determined to be 15.0 m adding an allowance of 2.3 m to the maximum drop in water level of 12.7 m during discharge of 1.40 m^3/sec.

7.3 SEDIMENTATION STUDY

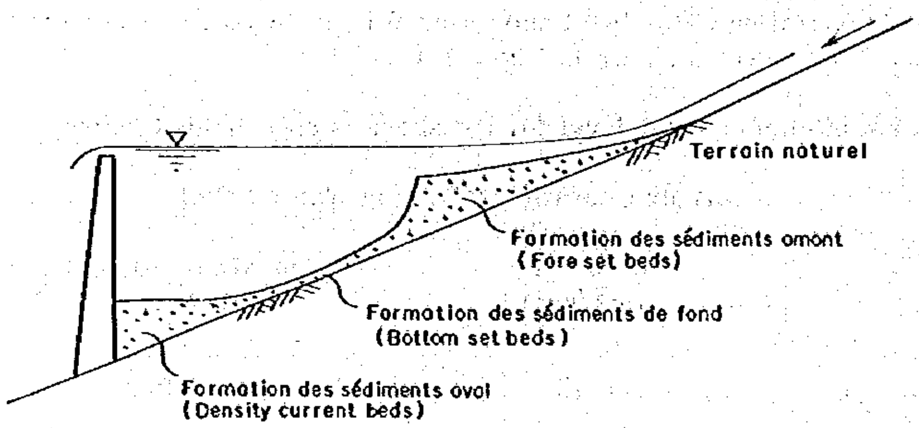
7.3.1 Outline

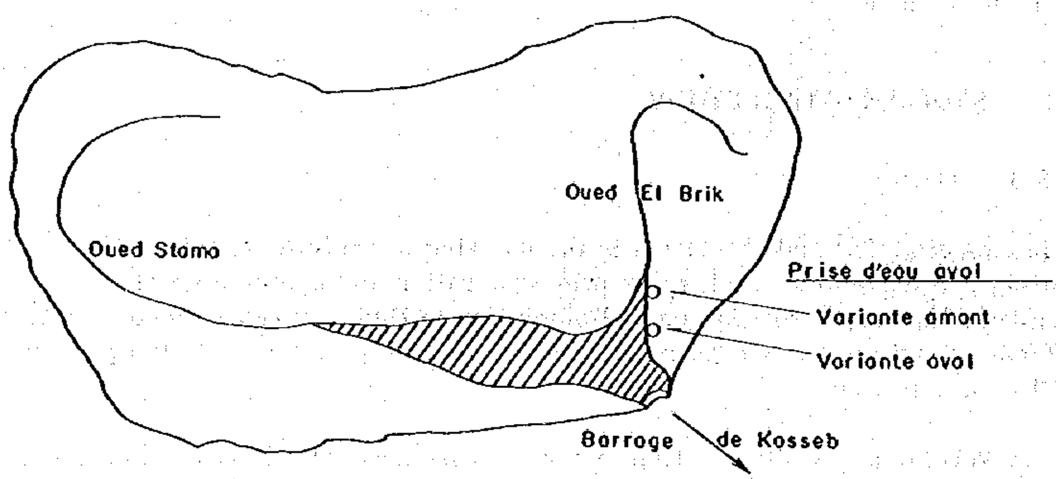
It is expected that sedimentation in the existing Kasseb Reservoir will affect the outlets and become obstacles to power generation and pumping up of water. Regretfully, measurements on sedimentation in this reservoir do not exist, and the effects cannot be studied quantitatively in concrete terms so that the present study will be qualitative.

As is well known, sedimentation in a reservoir generally begins from the back end of the reservoir due to the reason that the tractive force of a river gradually decreases from the back end of a reservoir to finally become zero in the vicinity of dam. For example, the form shown in Fig.-7.3 may be envisioned.

The effect on the Kasseb Power Station outlets can be considered to be the composite effects of fore set beds formed at the course of the El Brik River and damsite current beds from the entire catchment area including the course of the Stam River. However, since it is difficult to estimate the damsite current beds, when only the fore set beds of the course of the El Brik River are taken into consideration, the results will be as described below.

Figure 7-3 FORMATION DES SEDIMENTS DANS LE RESERVOIR

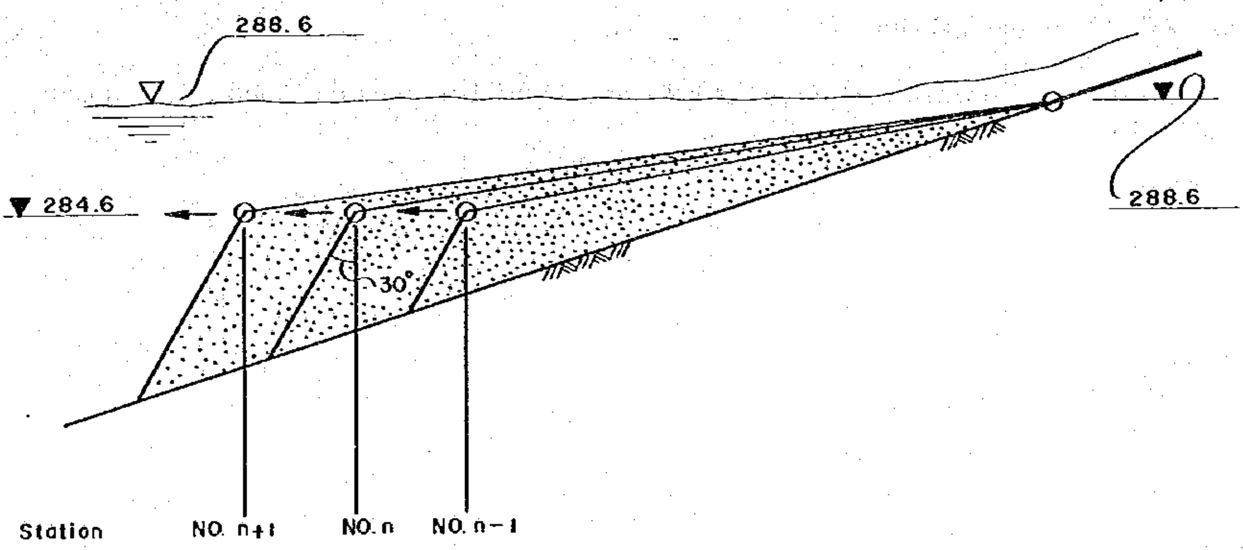




7.3.2 Sedimentation at El Brik River Course

The development of fore set beds is assumed simplified as shown in the drawing below, and sedimentation is thereupon calculated.

Fig-7.4 Process of Fore Set beds



Related drawings are given as Figs.-7.5 and 7.6 .

The amount of sedimentation thought not to affect the outlets at the lower reservoir of the lower reservoir of Kasseb are calculated to be the following:

Harmless Sedimentation at Outlet Location

		Upstream Alternative	Downstream Alternative
Station	No.	12	7
Sedimentation	m^3	1,240,000	8,000,000
Unit Sedimentation	$m^3/km^2/yr$	1,080	6,960

Harmless sedimentation: Quantity of fore set beds upstream of outlets.

Unit sedimentation: Corresponds to 50-year period.

On examining the only sedimentation dada* obtained on whether or not these sedimentation quantities would be permissible, Table 7.1* and Fig. 7.7 regarding the influence of sedimentation from the El Brik River on the outlets, the unit sedimentation of 6,690 $m^3/km^2/yr$ in case of the Downstream Alternative is a design sedimentation quantity which is quite reassuring, whereas the unit sedimentation of 1,080 $m^3/km^2/yr$ in case of the Upstream Alternative may be juged as being too small for a design sedimentation quantity.

However, since sedimentation in a reservoir is an extremely complex natural phenomenon which is determined by various conditions such as the soil quality of overburden in the catchment area, the form of the river, frequency of floods, etc. , it is absolutely necessary for early measurements to be made of sedimentation in the existing Kasseb Reservoir.

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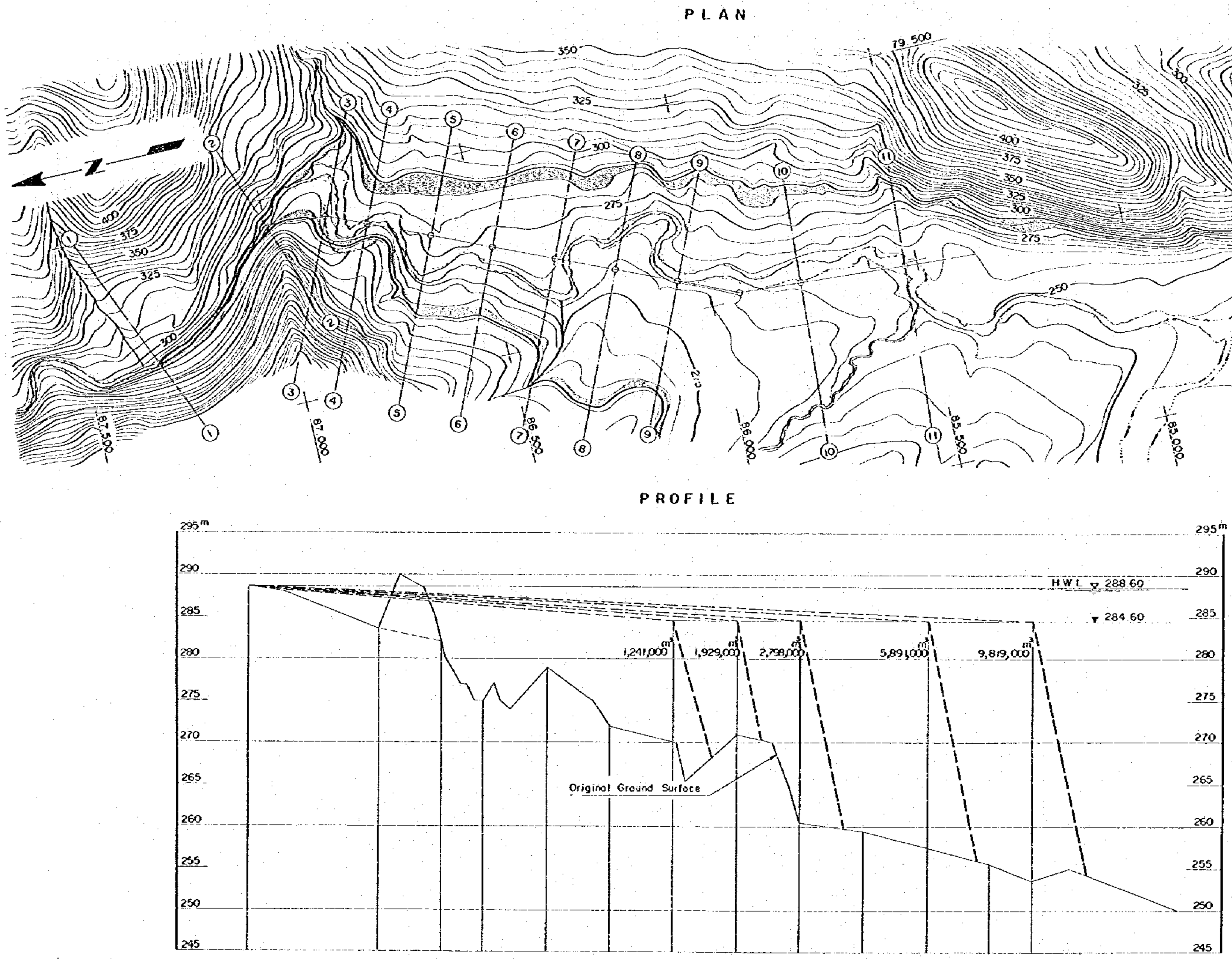
* Mesure de l'envasement dans les retenues de six barrages en Tunisie - Campagne de 1975



Sta-tion	Section Distance	Accumu-lative Distance	Original Ground Le.
1	0	0	288.60
2	308.00	308.00	283.50
3	150.00	458.00	281.50
4	100.00	558.00	275.00
5	150.00	708.00	279.00
6	150.00	858.00	272.00
7	150.00	1008.00	270.10
8	150.00	1158.00	271.00
9	150.00	1308.00	260.40
	150.00	1458.00	259.50
10	150.00	1608.00	257.50
	150.00	1758.00	255.50
11	100.00	1858.00	253.50

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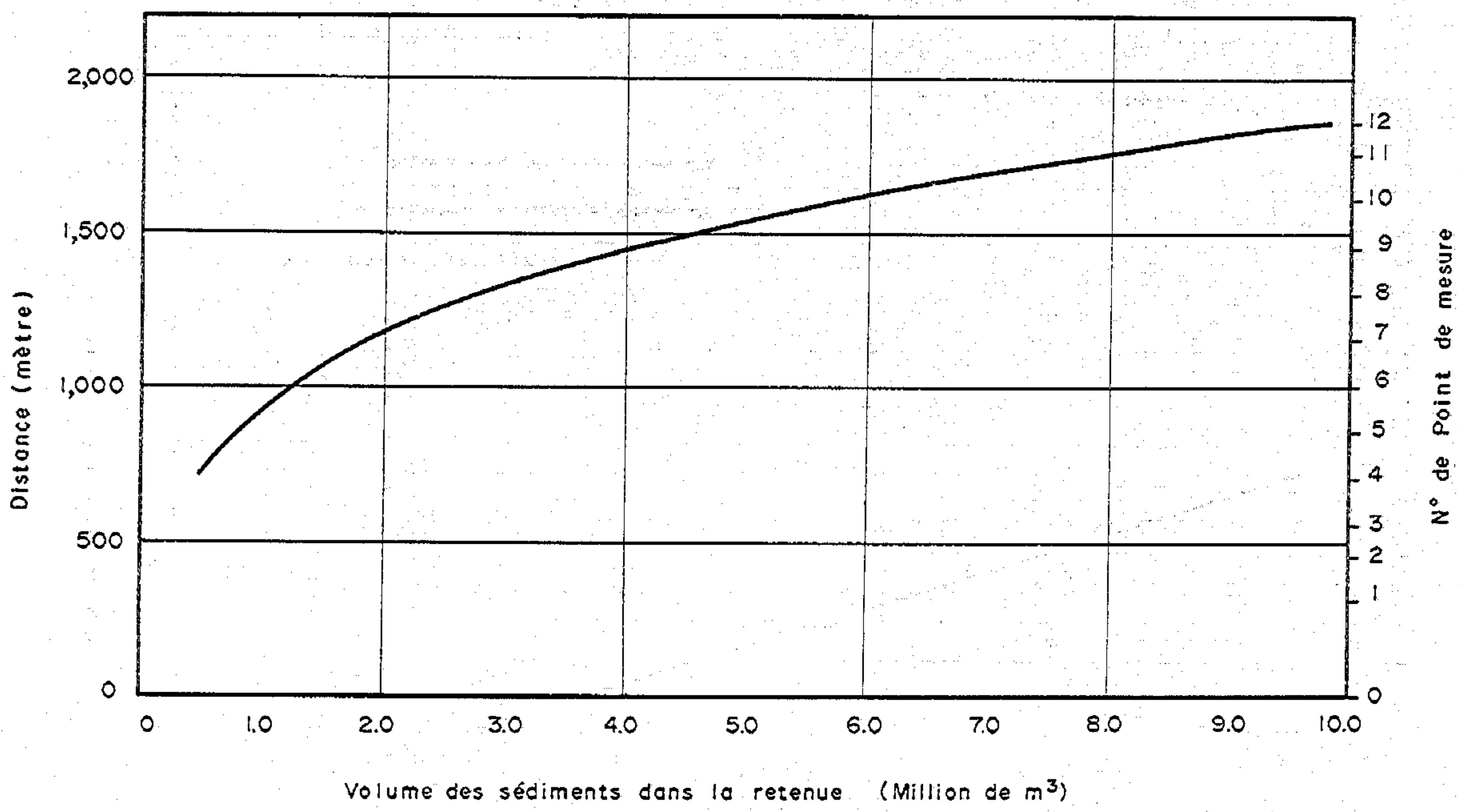
KASSEB PUMPED STORAGE PROJECT

STUDY OF SEDIMENTATION (1-2)

— Lower Kasseb Reservoir —

Figure.- 7.5 | August 1978

GRAPHIQUE 7-6 ETUDE SUR L'ENVASEMENT
—RESERVOIR DE KASSEB—



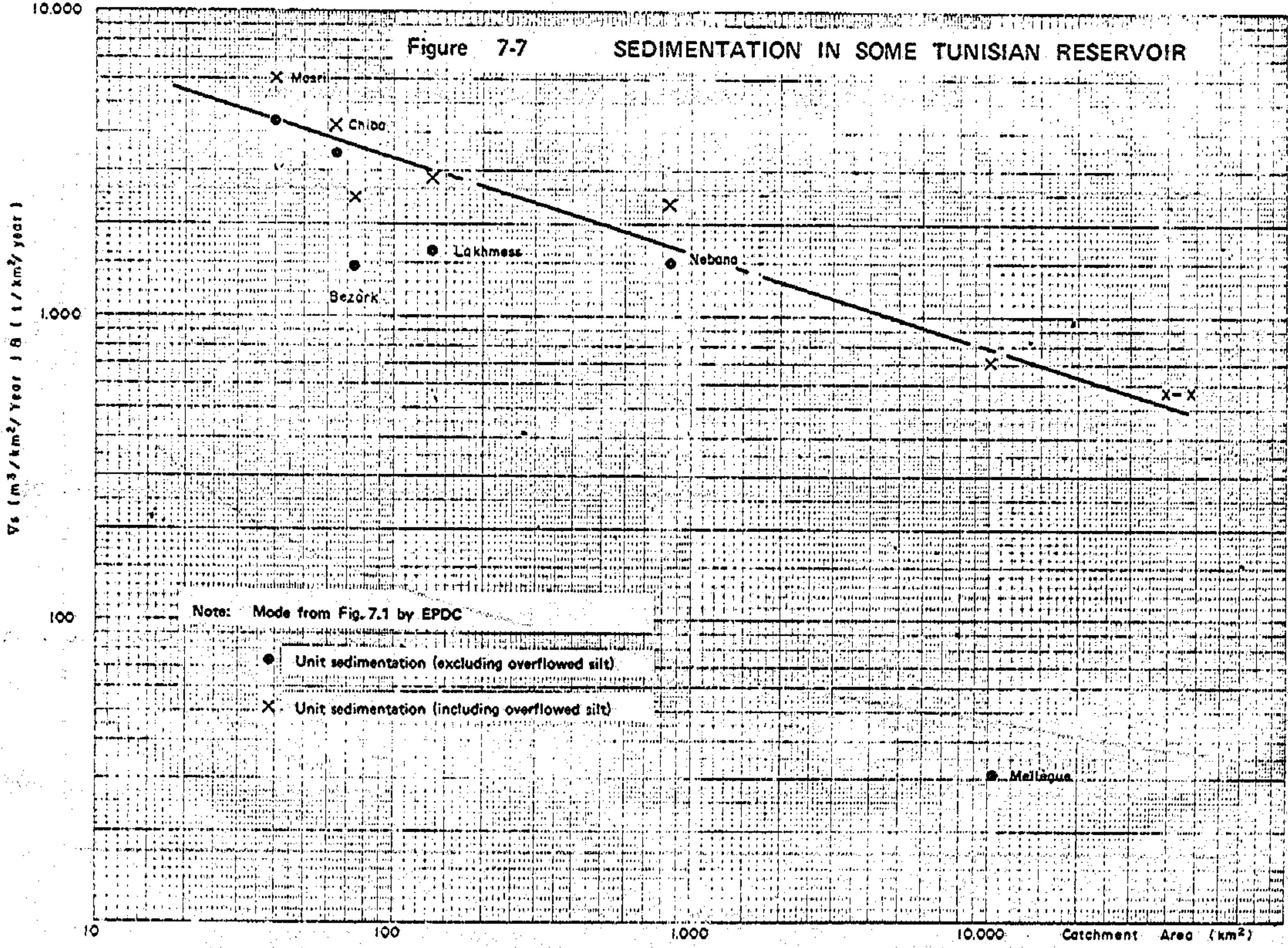
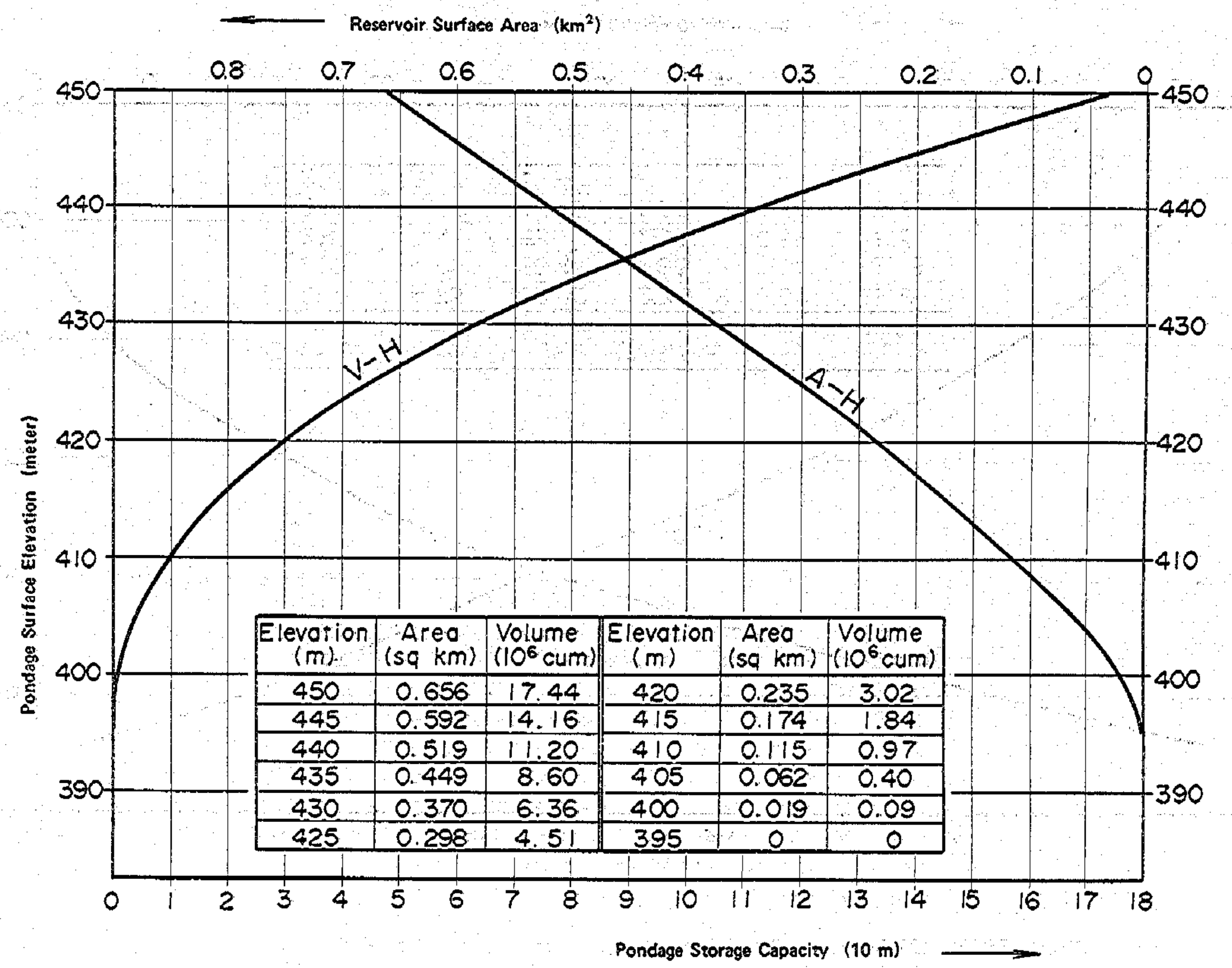


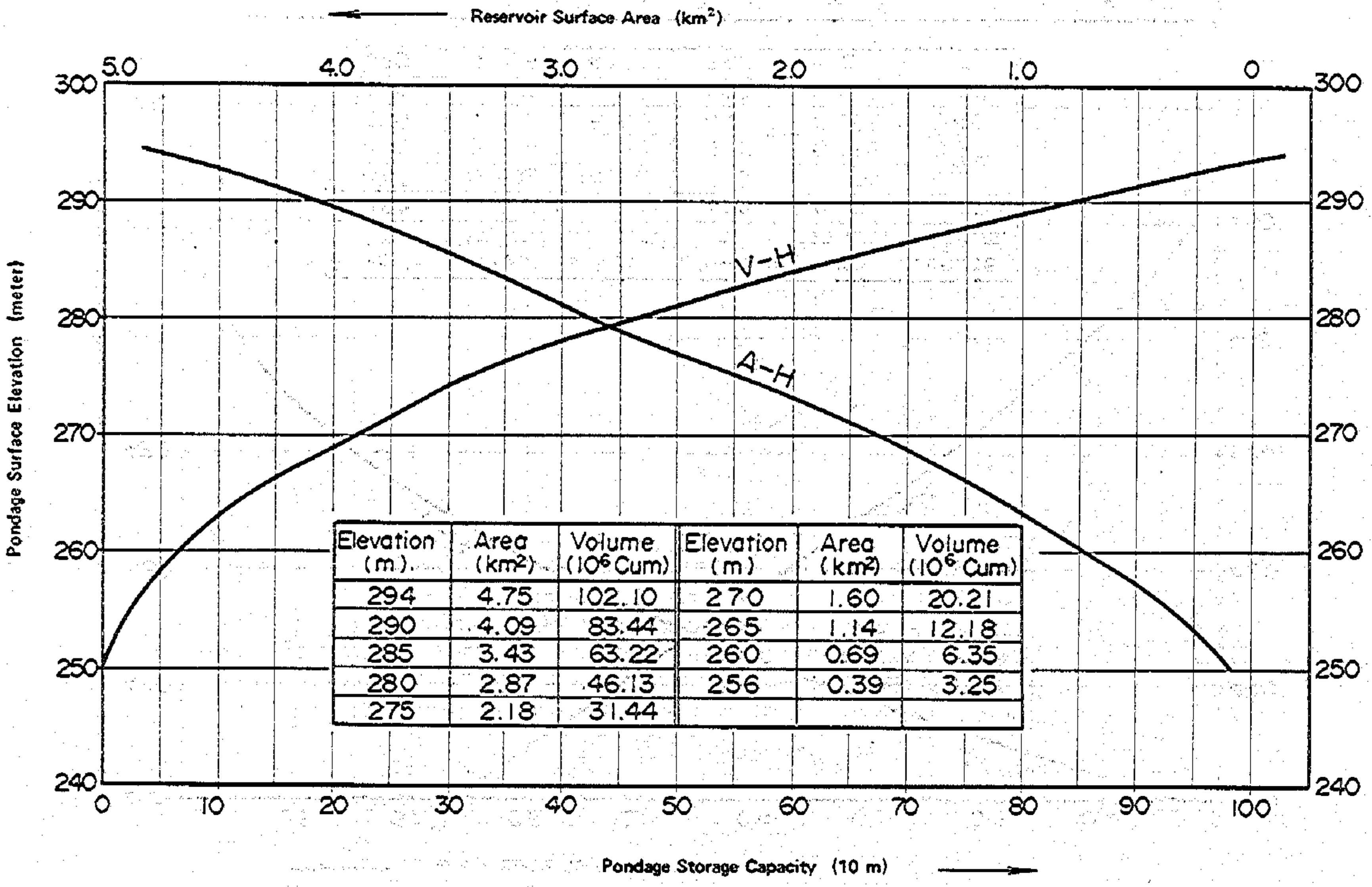
Figure 7-7 SEDIMENTATION IN SOME TUNISIAN RESERVOIR

Figure 7-8 SURFACE AREA AND STORAGE CAPACITY (UPPER RESERVOIR)



Elevation (m)	Area (sq km)	Volume (10^6 cum)	Elevation (m)	Area (sq km)	Volume (10^6 cum)
450	0.656	17.44	420	0.235	3.02
445	0.592	14.16	415	0.174	1.84
440	0.519	11.20	410	0.115	0.97
435	0.449	8.60	405	0.062	0.40
430	0.370	6.36	400	0.019	0.09
425	0.298	4.51	395	0	0

Figure 7-9 SURFACE AREA AND STORAGE CAPACITY (LOWER RESERVOIR)



Elevation (m)	Area (km²)	Volume (10^6 Cum)	Elevation (m)	Area (km²)	Volume (10^6 Cum)
294	4.75	102.10	270	1.60	20.21
290	4.09	83.44	265	1.14	12.18
285	3.43	63.22	260	0.69	6.35
280	2.87	46.13	256	0.39	3.25
275	2.18	31.44			

Tableau 7-1 TABLEAU RECAPITULATIF DES MESURES D'ENVASEMENT SUR LES SIX BARRAGES

NOM DU BARRAGE		Mellègue	Nebaann	Bezirk	Chiba	Masri	Lakhmess
Surface du bassin versant	km^2	10300	855	73–84	64	40	131
Volume total de la retenue	$10^6 m^3$	268	86,4	6,46	7,86	6,82	8,00
Date de la mise en service		Avril 54	Avril 65	Déc. 1960	1963	Fév. 1968	Avril 66
Période de fonctionnement jusqu'en Août 1975	ans	21,33	10	14,75	12	7,5	9,33
Volume de sédiments déposés	$10^6 m^3$	47,6	12,9	1,68	2,65	1,32	2,01
Taux d'envasement	%	17,8	14,9	26,0	33,7	19,3	25,1
Rythme annuel de comblement	%	0,83	1,5	1,76	2,81	2,58	2,69
Apports liquides totaux	$10^6 m^3$	4010	414,4	63,65	44,65	21,47	65,99
Apport moyen annuel	$10^6 m^3$	188	41,4	4,32	3,72	2,86	4,15
Apports solides totaux	$10^6 m^3$	126,7	16,4	2,42	2,70	1,52	2,92
Apports solides moyens annuels	$10^6 m^3$	5,94	1,64	0,164	0,225	0,202	0,313
Tonnage moyen annuel	$10^6 t$	7,13	1,97	0,197	0,270	0,242	0,375
Erosion spécifique	$t/km^2/an$	695	2300	2430	4220	6050	2865
Charge moyenne des apports liquides	g/lt ou kg/m^3	38	47,5	45,6	72,5	85	53

Source: Mesure de l'envasement dans les retenues de six barrages en Tunisie – Campagne de 1975

7.4 STANDARD WATER SURFACE LEVEL DURING MAXIMUM OUTPUT

In case of maximum output of 350 MW, the intake water level of the upper reservoir and the tailwater surface level of the lower reservoir, the following will be planned:

Upper Reservoir:

High water surface level at 430.0 m, low water surface level at 412.0 m. In this case, the standard surface level which is the centroidal water level of the effective water storage is El. 424.0 m.

Lower Reservoir:

High water surface level is at El. 288.6 m, but low water surface level can be considered at the two levels of either El. 273.6 m or El. 268.6 m. The operation of the lower reservoir will be done for water supply between this water level difference.

Therefore, the standard water surface level of the lower reservoir for design of turbines and generators is taken to be El. 281.0 m, intermediate between El. 288.6 m and El. 273.6 m.

Further, regarding the standard water surface levels in the cases of the 250 MW and 300 MW alternatives and corresponding dam height described later, the upper reservoir water level are to follow the thinking of the 350 MW alternative, while in case of the lower reservoir, since the dam already exists, it is to be the same as for the case of the abovementioned 350 MW alternative.

The impounded areas and water storage capacity curves of the upper and lower reservoirs are as indicated in Fig. 7.8 and 7.9.

7.5 EXAMINATION OF ALTERNATIVES

With this pumped - storage power station project, since the existing Kasseb Reservoir is to be utilized as the lower reservoir, the upper reservoir will be at a site within a limited area from a geographical standpoint.

Therefore, in the examination of the Project, 6 comparison proposals were taken up under the conditions below regarding outlet location, available drawdown of lower reservoir, and scale of power generation, and based on the respective unit construction costs per kilowatt, the economics are compared.

(1) Outlet Location:

The existing planned location is at approximately 1.5 km upstream on the left-bank side of Kasseb Dam, but in consideration of hindrances produced by sedimentation, a location approximately 0.8 km upstream and at the left bank of Kasseb Dam was selected for the comparison proposal. The former is considered as the upstream alternative, and the latter as the downstream alternative.

(2) Lower Reservoir Available Drawdown

The existing plan is for available drawdown of 20 m, but in this report, besides the 20 m proposal, a study will be mado for the previously-mentioned 15 m proposal also.

However, the 20 m available drawdown proposal for the upstream alternative was omitted from the comparisons since the bed height of the outlet structure would be lower than the original river bed of the reservoir and undesirable from the standpoint of operation.

(3) Installed Capacity

Under the conditions (1) and (2), three alternatives will be considered for the case of power generation scale of 350 MW, besides which, the two alternatives of outputs of 30 MW and 250 MW will be considered for the case of available drawdown of 15 m for the downstream outlet alternative unaffected by sedimentation.

Further, for the 300 MW and 250 MW alternatives, the high water surgace levels will be changed according to power generation scale to maintain peak duration time at approximately 4 hours similarly to the case of 350 MW.

Output of 350 MW is the maximum that can be obtained in case of upper reservoir high water surface level at 430 m and of peak duration time of approximately 4 hours.

The particulars and the construction costs of the 5 alternatives mentioned above are as indicated in Table 7.2.

Tableau 7-2 COMPARAISON DES CARACTERISTIQUES GENERALES ET DES COUTS DIRECTS DES TRAVAUX DES CINQ VARIANTES

Description	Unité	Variante "amont"	Variante "aval"			
Marnage de la retenue inférieure	m	15,00	15,00	15,00	15,00	20,00
Puissance totale installée	MW	350	350 (A)	300 (B)	250 (C)	350 (D)
Puissance unitaire x nombre de groupe	MW	75 x 2	75 x 2	75 x 4	50 x 2	75 x 2
	MW	100 x 2	100 x 2		75 x 2	100 x 2
Réservoir supérieur						
Retenue normale	cote	430	430	428,5	426,5	430
Volume brut	$10^6 m^3$	6,36	6,36	5,68	5,0	6,36
Volume utile	$10^6 m^3$	5,04	5,04	4,36	3,68	5,04
Marnage disponible	m	18	18	16,5	14,5	18
Barrage supérieur						
Type : Enrochements						
Hauteur x longueur en crête	m	50 x 400	50 x 400	48,5 x 390	46,5 x 360	50 x 400
Volume du barrage	$10^3 m^3$	960	960			960
Réservoir inférieur						
Retenue normale	cote	288,60	288,6	288,6	288,6	288,6
Volume utile	$10^6 m^3$	49,4	49,4	49,4	49,4	59,7
Galerie						
– Galerie d'amenée						
Longueur	m	954,5	727,5	727,5	727,5	727,5
Diamètre	m	6,7~3,9	6,7 ~ 3,9			6,7 ~ 3,9
– Galerie de fuite						
Longueur	m	490	1.861			1.861
Diamètre	m	7,5~4,5	6,7~4,2			6,7~4,2
Production de la puissance						
Niveau d'eau pondéré de la retenue amont	cote	424,0	424,0	423,0	421,7	424,0
Niveau d'eau pondéré de la retenue aval	cote	281,0	281,0	281,0	281,0	281,0
Chute utile	m	137,4	134,5	133,5	132,2	134,5
Débit turbiné équipé	m^3/sec	304,0	310,0	268,0	226,0	310,0
Coût direct des travaux de génie civil	1000 Dinars	39.153	48.620	44.269	39.385	49.924
Coût unitaire par kW installée	DT/KW	112,0	138,9	147,6	157,5	142,6

Note: Les coûts d'aménagement de la variante "amont" comprend les coûts de dragage du réservoir inférieur.