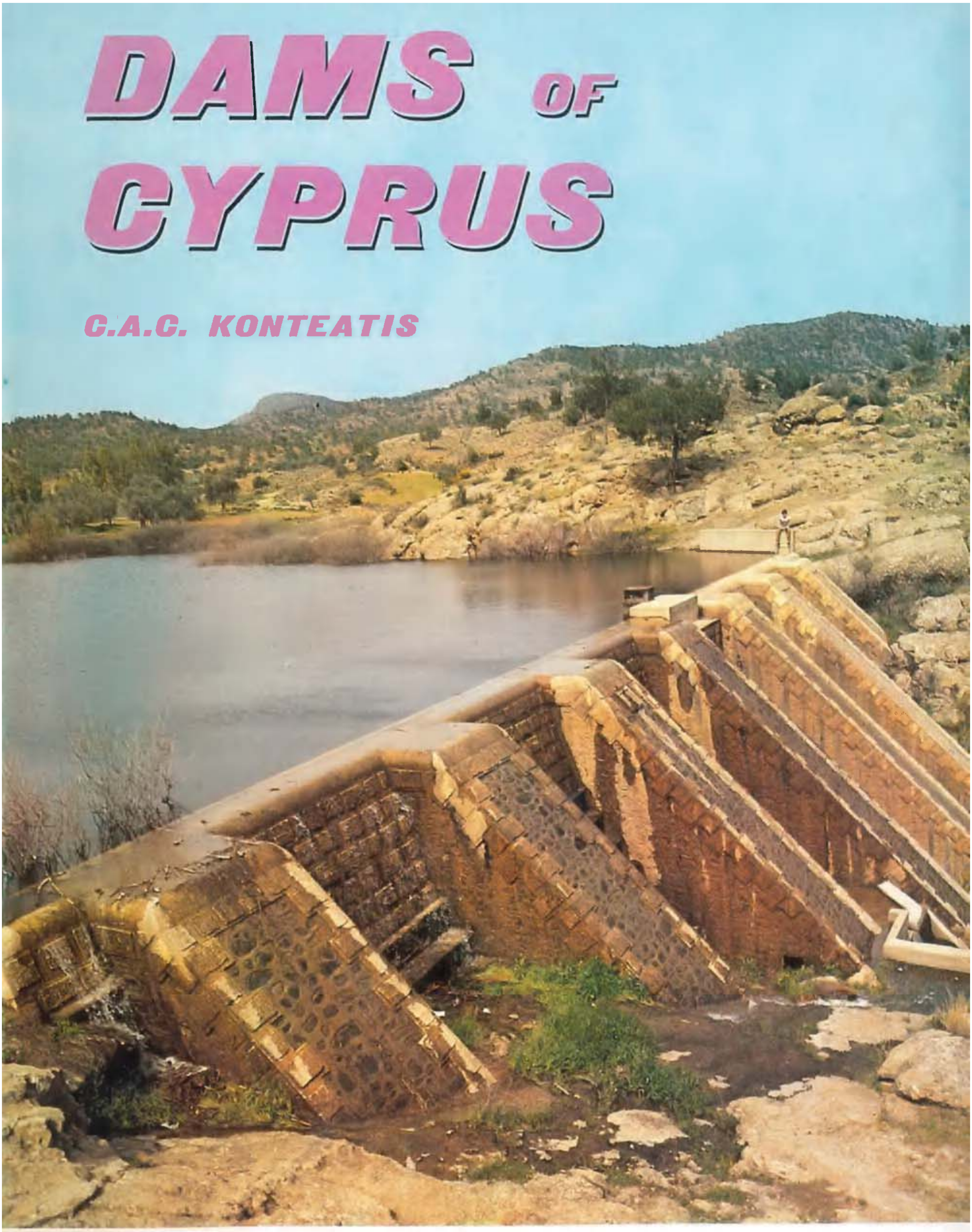
DAMS OF CYPRUS

C.A.C. KONTEATIS



Republic of Cyprus

Ministry of Agriculture and Natural Resources

Water Development Department

DAMS OF CYPRUS

C.A.C. KONTEATIS

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Finally, the contribution of a number of other members of the Department in various fields has also to be acknowledged.

ABBREVIATIONS - NOTATIONS

mm	=	Millimetre.
cm	=	Centimetre.
m	=	Metre.
km	=	Kilometre.
cm^2	=	Square centimetre.
m^2	=	Square metre.
km^2	=	Square kilometre.
ha	=	Hectare = 10,000 square metres.
m^3	=	Cubic metre.
l	=	Litre.
kg	=	Kilogram.
t	=	Ton.
s	=	Second.
a	=	Annum.
dia	=	Diameter.
asl	=	Above sea level.
D/S	=	Downstream.
U/S	=	Upstream.
A.C.	=	Asbestos Cement Pipes.
R.C. or R.C.C.	=	Reinforced Concrete or Reinforced Cement Concrete.
B.S.	=	British Standard.
W.D.D.	=	Water Development Department.
Donum	=	120 feet x 120 feet.
Lugeon	=	Litres per metre per minute for a pressure of 10 kilograms per square centimetre.
£	=	Pound sterling up to July 1972. After this date it should be read as C£ (Cyprus pound).
C£1	=	£ stg 1.2

FOREWORD

The existence of a detailed study on one of the most difficult problems which the Cyprus Government faces, like the one on the scarcity of water, was undoubtedly an urgent necessity. This became even more touchy and delicate after the unprecedented drought that appeared in Cyprus during the last years, and especially during 1973.

It is therefore with great pleasure that the Ministry of Agriculture and Natural Resources has encouraged the publication of this very interesting study on DAMS OF CYPRUS, *for which the author, Mr. Konteatis, Director of the Department of Water Development, and all his associates are to be congratulated.*

The Department of Water Development has been dealing with the water problem and, since independence in 1960, special attention was given by it in investigating and studying the thorny problem of water and how to increase its availability both for irrigation purposes and for domestic use.

This study DAMS OF CYPRUS *touches in a very clarified way all sides of this problem, regarding the importance of saving the spate waters by all possible and practicable means and helps the reader to feel and understand the pressing necessity in saving every drop of water to the interest of the country as a whole.*

This publication gives a picture of the great efforts made and the large sums of money spent recently in the building of dams in Cyprus, and tries to convey a message to the reader that the water in Cyprus is of paramount importance to the development of the Island, and that since independence a lot of work has been done to promote and to construct water saving works including the building of dams.

ROGIROS CHR. MICHAELIDES
Director-General, Ministry of Agriculture & Natural Resources.

In preparing this book the Author has done a valuable service to the history and development of dam construction in Cyprus. It is the first time that such information has been collected and presented in a single and authoritative work of reference.

As I have had some contact with water problems in Cyprus since 1926 and, more closely since 1947, I am glad that I was given the opportunity to write this Foreward. Mr. Konteatis was the first Cypriot to become Director of Water Development and during his tenure of office since 1967 water production from dams has become of major importance, culminating in the recent completion of the Lefkara Dam which is the first to be built in Cyprus for the domestic needs of any city. This site was suggested by Mr. Konteatis about 14 years ago after arduous explorations on foot.

I encouraged Mr. Konteatis to form a National Committee of Large Dams of which he is now the Chairman and to apply to the International Commission on Large Dams for membership. This application was approved at the Executive Meeting of ICOLD held in Warsaw, Poland in 1969 and as a member nation Cyprus has benefited by the exchange of information and valuable personal associations which ICOLD provides. Much of the inspiration to write DAMS OF CYPRUS *stems from the example of such publications by other member countries and the outcome is a worthy addition to international literature on the dams of the World.*

HUGH H. DIXON *CBE, MA, FICE, FASCE, FIWE, FIPHE, MInstHE*
Vice-President for Europe of the International Commission on Large Dams.

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INTRODUCTION

Cyprus is an Island with a long history dating back several thousands of years before Christ. Its civilization, examples of which are abundantly manifest throughout the Island, has been one of the world's most dominant for hundreds of years before and after Christ. Water has ever since been utilized both for domestic and irrigation purposes and there are cases of lengthy conveyors made up of canals and aqueducts conveying water from distant springs to cities, such as the case of the ancient city of Salamis which was supplied with water from the Kythrea Spring, lying at a distance of about 35 km.

Unfortunately, Cyprus has no rivers of perennial flow. Most rivers flow only during the winter and spring months, so that no dependable supplies can be obtained from them without storage. In the case of groundwater, it is found at relatively big depths in most parts of the Island which made it difficult to be developed in the older days when mechanical means were not available. With these problems in the ground and surface water development in the old days, it was natural that water coming from springs was the one primarily used. The possibility for surface water development through the construction of storage reservoirs presents many technical and economic problems such as poor topography and geology. Consequently, the construction of dams in days before the development of modern machinery and dam techniques was a tough proposition. Although in other countries of similar or even less civilization, favourable topographic and geologic conditions made possible the construction of dams very early in history, in Cyprus, for the reasons mentioned, this had not become possible until recent times.

The first important works constructed for impounding water on a large scale, were carried out at a cost of £60,000 in 1896-1912. These works were the Eastern Mesaoria reservoirs at Kouklia, Akhyritou and Syngrasi formed of long low earth embankments up to 8 m high, of a total storage of about 5 million m^3.

Until recently there has been a controversy on the economic advantages of building dams in Cyprus.

In the early 1900, the opinion was that further impoundment of water in Cyprus other than that already carried out in the Eastern Mesaoria, should not be undertaken, mainly due to the unfavourable configuration of the country for the construction of cheap dams for irrigation, which in most cases is such that the cost of construction would be out of all proportion to the revenue leviable for the use of the water, the only part of the country giving favourable sites for reservoirs being the plain of the Eastern Mesaoria.

Other disadvantages pointed out were the unreliable runoff, the heavy sediment transport and adverse effects on downstream water rights and on natural groundwater recharge through river beds.

Thus the opinion at the time was that no important irrigation works involving the construction of dams could be economically carried out and that it was only possible to construct small irrigation works on a village scale by building small river diversion intakes operating seasonally when water was available in the rivers.

This remained the policy of the Government until about 1950. During this period, emphasis was given on the development of groundwater, including springs and small river diversion works.

Subsequent investigations carried out established, that although dams are costly per unit volume of water stored, yet there still were many suitable sites where sizeable dams could be economically built for the production of high return crops. In addition, domestic and industrial demands would afford even more costly projects. Finally, our aquifers, which produced water at least three times cheaper than dams, have been badly overpumped with a consequential depletion and sea intrusion in many parts. As a result, with the beginning of 1950, a cautious dam programme was initiated, which by the end of the decade became a very important component of water development. With the next decade of 1960, the growing demand for irrigation, domestic, industries and tourism, led the Government to embark on a major programme of dam construction, so that within the last 13 years the storage capacity of our dams has increased from 6 to 64 million m^3 as shown in Fig. 2. Table 1 records the main data of the 26 dams completed which can be classified as large according to the International Commission's standards, 22 of which have been constructed during the last decade. Fig. 1 indicates the location of these dams. In addition well over 100 smaller dams mainly in the form of earth fill have been constructed for direct irrigation or groundwater recharge purposes.

Admittedly, due to the configuration of the country, the steep terrain and adverse geologic conditions, dam projects are amongst the most costly in the world regarding the cost of water per unit volume stored. In an extreme case a dam has been built producing water at 45 mils/m^3 at the dam. However, the weighted average cost for all dams constructed to date is 17.5 mils/m^3. Even considering the extreme case, the cost is much cheaper than that of producing water by seawater desalination which has been decided to be introduced by the end of the 1970's as a pilot project of a 5,000 m^3/day capacity connected to the proposed Electricity Generating Station at Dhekelia.

With only 25% of our surface water resources being utilized at the moment, our main dam programme still lies ahead. Feasibility Studies for utilizing the balance of our surface flows have already been completed and important dam construction is scheduled to start soon in the regions of Paphos, Morphou, Limassol and Larnaca involving larger dams than what we have built up to date, the largest of which may be up to 100 million m^3 capacity.

Our dam development programme for storing and utilizing the surface water, still lost to the sea, is envisaged to be carried out over the next 15 years.

C. A. C. KONTEATIS *B.Sc. (Eng.) FICE, FIWE, FGS*
Director, Water Development Department

CHAPTER I

PLANNING REQUIREMENTS

PRINCIPLES OF POLICY

1. AIMS AND OBJECTIVES

In building dams for storing the surplus water of the rivers, we have the following important objectives in mind.

i. To substitute the water extracted from the depleted groundwater resources for the irrigation of extensive plantations in many parts of the Island. This can be done either by direct water supply to these plantations or by groundwater recharge of the aquifers.

ii. To supply enough water to meet the present and expanded domestic water requirements of towns, villages, tourist and industrial units.

iii. To expand irrigation and thereby increase the farmers' income to the benefit of the national economy.

iv. To protect inhabited and agricultural areas from floods.

In all cases an economic justification has to be made for each project, the benefit cost ratio of which should in general be at least 1, in order that the Government would consider it for implementation.

2. PLANNING POLICIES

In a country like Cyprus where the water resources are limited and the demand for water far exceeds the water available, we have to consider that the water resources are integrated and belong to the Island as a whole and that their utilization should take into account the requirements of all the economic sectors such as agriculture, domestic water supply, industries and tourism. Such conditions call for certain basic guidelines of policy which have to be observed during dam planning.

a. Multi Purpose Projects

Dams are quite expensive structures and it is necessary to consider their use for as many purposes as possible. Such purposes can be irrigation, groundwater recharge, domestic water supply, flood protection and hydropower.

Dams have been built in Cyprus combining all the above purposes except the production of power. For example, the Morphou dam on the Serakhis river combines irrigation and groundwater recharge, the Athalassa dam combines flood protection and irrigation and the Lefkara dam combines domestic water supply and irrigation.

Regarding the production of power from dams, it has been found that due to the infrequency and limited flow of our rivers, such possibilities are of limited application.

Hydropower in the form of kinetic energy has been for many centuries developed in Cyprus by conveying water from rivers over a vertical shaft, the falling energy of which would move grinding wheels for crushing wheat into flour. This form of power has now been replaced by modern petrol or electrically driven machinery.

b. Integrated River Basin Studies

The river basins of Cyprus, however small, are inhabited by as many as 30 villages throughout their catchment of an average population of 500 people each. Every village relies for its livelihood mainly on agriculture and the provision of adequate water for irrigation is of fundamental importance. The importance of irrigation for every community calls for an integrated use of the whole water resources, with a study made of all potential requirements of the whole basin and the necessary water allocated through projects which have to be identified as economic. On the upstream villages, where the river valleys are steep and prohibit the construction of dams, small irrigation works in the form of river intakes with piped or channel distribution systems, are constructed. Where possible, small impounding dams have been built and in other cases cut and fill reservoirs have been erected. The latter system of open reservoirs is now being widely introduced for the upstream villages, as it has the additional advantage of an off-channel resevoir which reduces the rate of silting up by the high sediment transportation along the river beds. Downstream, larger irrigation projects are built and better land is available.

c. Multi River Basin Studies

As the costal strips of land in many parts of the Island form the best agricultural soils and embrace more than one river inter-fluvially, with the water resources of some of the rivers being in excess of the demand within their particular catchments whilst the existing domestic and irrigation demands in other regions exceed the available water supplies in those regions, it becomes necessary to plan for multi-river basin studies involving the interconnection of the water and land resources systems.

Such multi-river basin irrigation projects have been planned for Paphos, Akrotiri, Larnaca, Famagusta and Morphou-Tylliria involving the interconnection of several rivers.

An interesting completed example is the major domestic water supply system of Famagusta conveying water from several watersheds in Larnaca lying as far as 80 km away from the town.

DAMS CONSTRUCTED UP TO 1960

No	DAM	TYPE	HT	1000m³	YEAR
1	Kouklia	Earth	6	4,545	1900
2	Lymbia	Gravity	5	18	1945
3	Lythrodhonda	Gravity	11	32	1945
4	Kalokhorio (KI)	Gravity	9	82	1947
5	Akrounda	Gravity	7	23	1947
6	Galini	Gravity	11	23	1947
7	Petra	Gravity	9	32	1948
8	Petra	Gravity	9	23	1951
9	Lythrodhonda	Gravity	10	32	1952
10	Kafizes	Gravity	23	113	1953
11	Ayios Loucas	Earth	3	455	1955
12	Gypsos	Earth	3	100	1955
13	Kandou	Gravity	15	34	1956
14	Perapedhi	Gravity	22	55	1956
15	Pyrgos	Gravity	22	285	1957
16	Trimiklini	Gravity	33	340	1958

Total Storage Capacity 6,192 $m^3 \times 10^6$

MINOR RECHARGE DAMS FROM 1960-70

No	DAM	TYPE	HT	1000m³	YEAR
47	Sotira	Earth	8	45	1962
48	Panayia (F)	Earth	7	45	1962
49	Paralimni (45)	Earth	5	115	1963
50	Ayia Napa (7)	Earth	8	55	1963
51	F'sta Recharge	Earth	5	50	1963
52	Phrenaros (6)	Earth	5	115	1964
53	Dherynia	Earth	6	23	1964
54	Phrenaros (3)	Earth	7	45	1966
55	Avgorou (7)	Earth	3	68	1966
56	Kandea (2)	Earth	5	82	1966
57	Xylophaghou (4)	Earth	7	86	1966
58	Sotira (4)	Earth	5	32	1966
59	Lysi	Earth	7	77	1967
60	Ay. Yearyios (9)	Earth	3	68	1967
61	Ay Epiktitos (6)	Earth	6	34	1968
62	Akanthou (6)	Earth	6	45	1968
63	Akhna (3)	Earth	4	40	1968
64	Xylotymbou (5)	Earth	5	50	1969

Total Storage Capacity 1,075 $m^3 \times 10^6$

MAJOR DAM PROJECTS FROM 1960-70

No	DAM	TYPE	HT	1000m³	YEAR
17	Pradhramos	Earth	10	122	1962
18	Morphou	Earth	13	1,879	1962
19	Lefka	Gravity	35	368	1962
20	Geunyeli	Earth	15	1,045	1962
21	Athalassa	Earth	18	791	1962
22	Kanli Keuy	Earth	19	1,113	1963
23	Argaka	Rockfill	41	1,150	1964
24	Mia Milia	Earth	22	355	1964
25	Ovgos	Earth	16	845	1964
26	Kiti	Earth	22	1,614	1964
27	Agros	Earth	26	99	1964
28	Liopetri	Earth	18	340	1964
29	Polemidhia	Earth	45	3,864	1965
30	Ayia Marina	Rockfill	33	311	1965
31	Kalopanayiotis	Earth	40	391	1966
32	Mavrokolymbos	Earth	45	2,180	1966
33	Pomos	Rockfill	38	859	1966
34	Yermasoyia	Earth	49	13,600	1968
35	Syngrasis	Earth	7	1 115	1968

Total Storage Capacity 32,041 $m^3 \times 10^6$

MAJOR RECHARGE DAMS FROM 1960-70

No	DAM	TYPE	HT	1000m³	YEAR
36	Ayios Yearyios	Earth	6	90	1962
37	F'sta Antiflood	Earth	8	165	1963
38	Ayios Nikolaos	Earth	2	1,365	1964
39	Paralimni Lake	Earth	1	1,365	1964
40	Fresh Water Lake	Earth	3	4,545	1964
41	Makrasyka	Earth	8	195	1966
42	Akhna (Mesania)	Earth	4	90	1967
43	Morphou spreading grounds	Earth	5	130	1969
44	Ormidhia	Earth	5	100	1968
45	Vrysoules	Earth	7	140	1969
46	Protopapas	Earth	6	90	1970

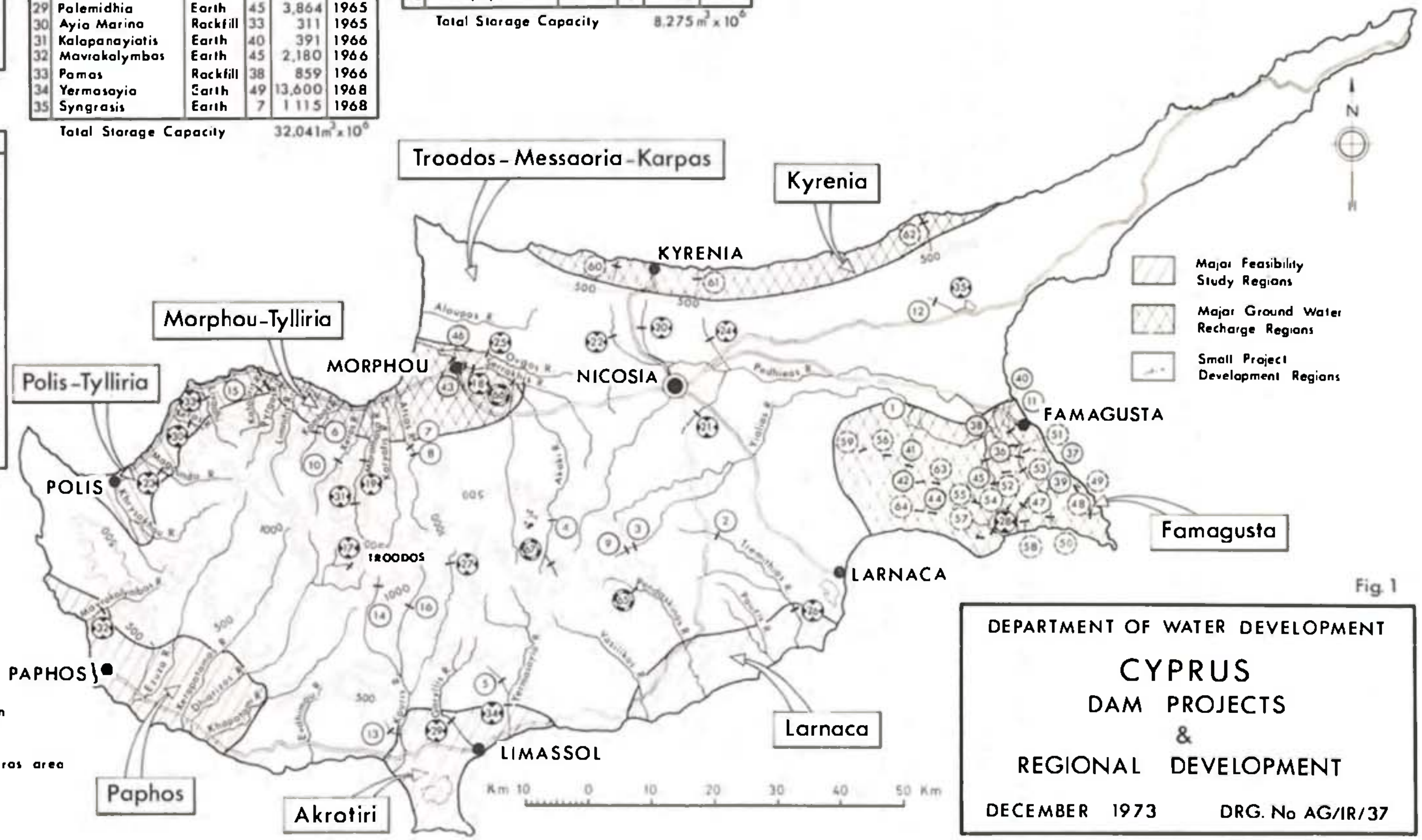
Total Storage Capacity 8,275 $m^3 \times 10^6$

MAJOR DAM PROJECTS FROM 1971-73

No	DAM	TYPE	HT	1000m³	YEAR
65	Lefkara	Earth Rockfill	74	13,850	1973
66	Massari Recharge dam	Earth	15	2,273	1973
67	Palechari-Kambi	Gravity	33	620	1973

Total Storage Capacity 16,743 $m^3 \times 10^6$

GRAND TOTAL UP TO END OF 1973 : 64.4 $m^3 \times 10^6$



(1) Dams constructed up to 1960
Major dam projects from 1960-70
Major dam projects from 1971-73
Major recharge dams from 1960-70
(47) Minor recharge dams from 1960-70

H T refers to height in meters from foundation

YEAR is the year of completion

Phrenaros (6) means six small dams in Phrenaros area

Fig. 1

DEPARTMENT OF WATER DEVELOPMENT

CYPRUS
DAM PROJECTS
&
REGIONAL DEVELOPMENT

DECEMBER 1973 DRG. No AG/IR/37

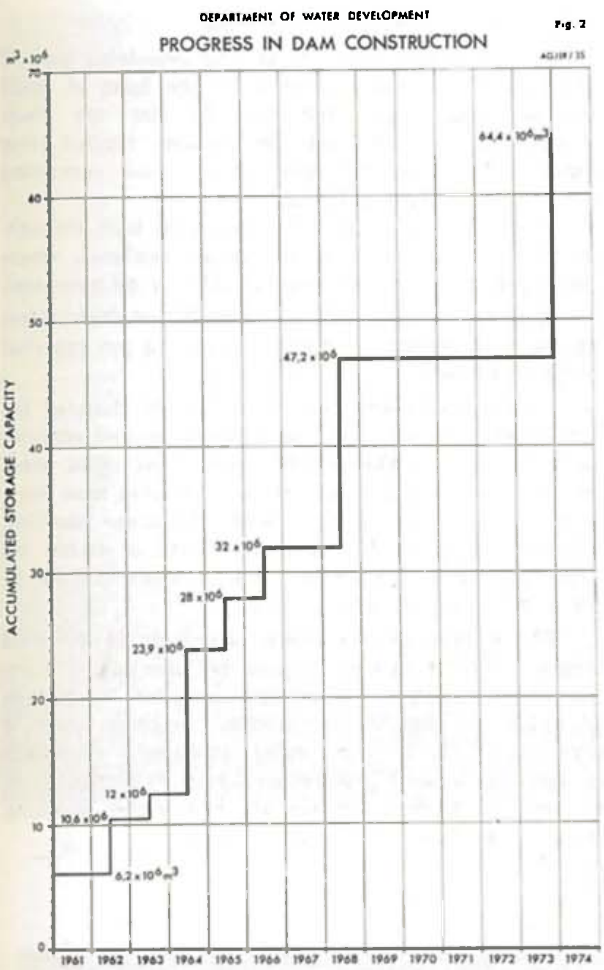
REGISTRE DES BARRAGES EN CHYPRE — *REGISTER OF DAMS IN CYPRUS*

TABLE 1

Ligne / Line No	Nom du barrage / *Name of dam*	Année d'achèvement / *Year of completion*	Cours d'eau / *River* (Situation / *Location*)	Ville la plus proche / *Nearest city* (Situation / *Location*)	Etat, province ou département / *State, province or county* (Situation / *Location*)	Type	Hauteur au-dessus de la plus basse fondation / *Height above lowest foundation* (m)	Longueur de crête / *Length of crest* (m)	Volume du barrage / *Volume content of dam* (10^3 m³)	Capacité totale du réservoir / *Gross capacity of reservoir* (10^3 m³)	Destination / *Purpose*	Capacité maximale des évacuateurs / *Maximum discharge capacity of spillways* (m³/s)	Type des évacuateurs / *Type of spillways*	Propriétaire / *Owner*	Bureau d'études / *Engineering by*	Constructeur / *Construction by*	Ligne / Line No
1	KAFIZES	1953	Xeros (Morphou)	Nicosia	Nicosia	PG	23	27	4	113	I	54	L	Lefka Irrigation Division	Department of Water Development	Department of Water Development	1
2	KANDOU	1956	Kouris	Limassol	Limassol	PG	15	53	2	34	I	59	L	Kandou Irrigation Division	Department of Water Development	Department of Water Development	2
3	PERAPEDHI	1956	Kouris	Limassol	Limassol	PG	22	62	4	55	I	107	L	Perapedhi Irrigation Division	Department of Water Development	Department of Water Development	3
4	PYRGOS	1957	Katouris	Nicosia	Nicosia	PG	22	66	5	285	I	125	L	Pyrgos Irrigation Division	Department of Water Development	Department of Water Development	4
5	TRIMIKLINI	1958	Kouris	Limassol	Limassol	PG	33	76	6	340	I	59	L	Trimiklini Irrigation Division	Department of Water Development	Department of Water Development	5
6	ATHALASSA	1962	Pedhieos	Nicosia	Nicosia	TE	18	447	103	791	I	48	L	Government	Department of Water Development	Department of Water Development	6
7	GEUNYELI	1962	Pedhieos	Nicosia	Nicosia	TE	15	254	50	1 045	I	173	L	Geunyeli Irrigation Division	Department of Water Development	Department of Water Development	7
8	LEFKA	1962	Marathasa	Nicosia	Nicosia	PG	35	149	11	368	I	246	L	Lefka Irrigation Division	Department of Water Development	Department of Water Development	8
9	MORPHOU	1962	Serakhis	Nicosia	Nicosia	TE	13	1 436	206	1 879	I	764	L	Morphou Irrigation Division	Department of Water Development	Department of Water Development	9
10	PRODHROMOS	1962	off stream	Limassol	Limassol	TE	10	756	73	122	I	-	L	Prodhromos Irrigation Division	Department of Water Development	Department of Water Development	10
11	KANLI KEUY	1963	Pedhieos	Nicosia	Nicosia	TE	19	311	47	1 113	I	116	L	Kanli Keuy Irrigation Division	Department of Water Development	Department of Water Development	11
12	AGROS	1964	Kouris	Limassol	Limassol	TE	26	180	61	99	I	6	L	Agros Irrigation Division	Department of Water Development	Department of Water Development	12
13	ARGAKA	1964	Magounda	Paphos	Paphos	ER	41	173	138	1 150	I	0.3	L	Government	Howard Humphreys & Sons of U.K.	Department of Water Development	13
14	KITI	1964	Tremithios	Larnaca	Larnaca	TE	22	990	183	1 614	I	602	L	Government	Il Nuovo Castoro of Italy	Department of Water Development	14
15	LIOPETRI	1964	Potamos	Famagusta	Famagusta	TE	18	579	50	340	R	150	L	Liopetri Irrigation Division	Department of Water Development	Department of Water Development	15
16	MIA MILEA	1964	Pedhieos	Nicosia	Nicosia	TE	22	140	54	355	I	24	L	Mia Milea Irrigation Division	Department of Water Development	Department of Water Development	16
17	OVGOS	1964	Serakhis	Nicosia	Nicosia	TE	16	745	130	845	I	786	L	Morphou Irrigation Division	Department of Water Development	Department of Water Development	17
18	AYIA MARINA	1965	Xeros (Tyilirias)	Paphos	Paphos	ER	33	142	61	311	I	161	L	Ayia Marina Irrigation Division	Energoproject of Yugoslavia	Mediterranean Constructors Greece.-G.P. Zachariades Cyprus	18
19	POLEMIDHIA	1965	Garyllis	Limassol	Limassol	TE	45	196	215	3 864	I	581	L	Government	Energoproject of Yugoslavia	Howlem & Ridgway of U.K.	19
20	KALOPANAYIOTIS	1966	Marathasa	Nicosia	Nicosia	TE	40	137	156	391	I	207	L	Government	Howard Humphreys & Sons of U.K.	Department of Water Development	20
21	MAVROKOLYMBOS	1966	Mavrokolymbos	Paphos	Paphos	TE	45	528	267	2 180	I	340	L	Government	Energoproject of Yugoslavia	Cybarco of Cyprus	21
22	POMOS	1966	Livadhi	Paphos	Paphos	ER	38	302	153	859	I	300	L	Pomos Irrigation Division	Energoproject of Yugoslavia	Mediterranean Constructors Greece.-G.P. Zachariades Cyprus	22
23	YERMASOYIA	1968	Yermasoyia	Limassol	Limassol	TE	49	409	539	13 600	I	850	V	Government	Energoproject of Yugoslavia	Cybarco of Cyprus	23
24	LEFKARA	C (1973)	Pendaskinos	Larnaca	Larnaca	TE/ER	74	240	800	13 850	S/I	316	L	Famagusta Water Board & Lefkara Irrigation Division	Howard Humphreys & Sons of U.K.	L. Fairclough & Medcon Construction Ltd.	24
25	MASARI	C (1973)	Serakhis	Nicosia	Nicosia	TE	15	929	245	2 273	I	622	V	Government	Department of Water Development	Department of Water Development	25
26	PALEKHORI-KAMBI	C (1973)	Akaki	Nicosia	Nicosia	PG	33	131	27	620	I	65	L	Government & Palekhori Irrigation Division	Department of Water Development	Ioannou & Paraskevaides Ltd.	26

Flour Mill using hydropower for the operation of grinding wheel.

The Kouris River showing the proposed damsite and reservoir of up to 100 million m³ capacity



d. Integrated Island Wide Studies

All major projects have to be part of the Master Plan of the water resources and utilization of the Island, within the framework of which each individual project should form a link. A dynamic plan is now being processed made up of a multi-disciplinary system of hydrological, engineering, geological, agricultural, economic and other inputs, analysing alternative studies for water utilization schemes, benefits, benefit:cost ratios and internal rates of return. Such a computerized programme of development applied to the various regional and inter-regional plans enables for comparison purposes to introduce any technical, social or political data and derive the economic feasibility of the various alternative projects.

e. Regional Plans

All dams constructed and those to be constructed come under one of the Regional Plans shown in Fig. 1 These regions have been chosen on criteria of hydrology. soils, topography and water demands.

A description of the regional plans follows:

i. Morphou — Tylliria

The Morphou lands overlying the largest aquifer of the Island constitute the most important citrus producing region. There are about 75,000 donums of land under irrigation over 65,000 of which is citrus. The safe yield of the aquifer is about 50 million m^3/a whereas the extraction for irrigation is about 74 million m^3 with an additional 2 million annually for domestic supplies including Nicosia town.

The result of this overpumpage has been a considerable drawdown of the water table with a limited sea intrusion at the coastal region.

To overcome this adverse situation proper control of the extraction and conservation measures are applied and at the same time a programme for the construction of storage dams for the winter flows now being wasted to the sea has been implemented. Also a study for the storage of the surplus flows of the rivers situated West of Morphou as far as Tylliria to be diverted to the Morphou region for direct use or for groundwater recharge purposes has nearly been completed and construction work is scheduled to start soon.

ii. Polis — Tylliria

This is an area of water resources in excess to the present demand, both as groundwater and as river flows.

A programme has already been initiated for the construction of several storage dams. Already three have been constructed, which have contributed to the extension of irrigation in the region. The possibility for diverting surplus water from this region to Morphou by interconnecting both regions forms part of the Master Plan.

iii. Paphos

The rivers and groundwater resources of Western Paphos give more surplus water than those of any other region. Therefore, the development of these water resources will add considerably to the national economy. Already one major dam has been built and a Feasibility Study has been completed which envisages the building of a major dam on the Xeros river. The possibility for diverting surplus water Eastwards, is part of the Master Plan.

iv. Akrotiri

Significant surplus water is also available in this region both from rivers and aquifers. Already two major dams have been built and a Feasibility Study has been completed which provides for the construction of a major dam on the Kouris river. Also in this case the possibility for diverting surplus water Eastwards is part of the Master Plan.

v. Larnaca

Larnaca is also a region of surplus water and there is a considerable margin for the expansion of irrigation and meeting the domestic water supply requirements.

Already two major dams have been built. The one for the domestic water supply of Famagusta and Larnaca and the other for irrigation. The Feasibility Study provides for more dams and for the possibility of interconnecting with the water resources to the East and West of this region.

vi. Famagusta

This is the second most important aquifer of Cyprus and it is as important as Morphou for irrigation demand. In this region the deficiency of the water resources is very serious. For the irrigation of about

80,000 donums, mostly of vegetables, about 50 million m^3/a of water is used and an additional 3 million m^3 is used for domestic purposes. With the safe yield of the aquifer being only 22 million m^3/a, this substantial deficiency has resulted in a complete drawdown in some parts of the aquifer and to sea intrusion as far as 2 km inland in the coastal parts.

A considerable number of groundwater recharge schemes have been carried out in this region including over one hundred small check dams, underground infiltration galleries, recharge canals, spreading grounds and recharge wells. The possibility for the conveyance of surplus water from the West which would help to relieve the pressure on the groundwater is studied under the Master Plan. In fact the Master Plan in the Southern part of the Island, at its maximum projection would provide for an integrated basin wide utilization of the water resources from Paphos to Famagusta.

vii. Kyrcnia

The water resources of this region are mostly in the form of groundwater in the limestone mountain range and in the coastal Pliocene aquifer. A programme for small recharge dams is being implemented mainly on the Pliocene aquifer which enables the infiltration of stream flows into the groundwater, which would otherwise have been wasted to the sea. Under the current programme about 30 such dams have been built and possibly another 30 have still to be built.

viii. Troodos — Mesaoria — Karpas

In this region which forms the remaining part of the Island, the development is in the form of small storage dams. The limitations in size are steep topography and rocky soils in Troodos, limited river flows in Mesaoria, and poor quality water containing salts in the Karpas peninsula.

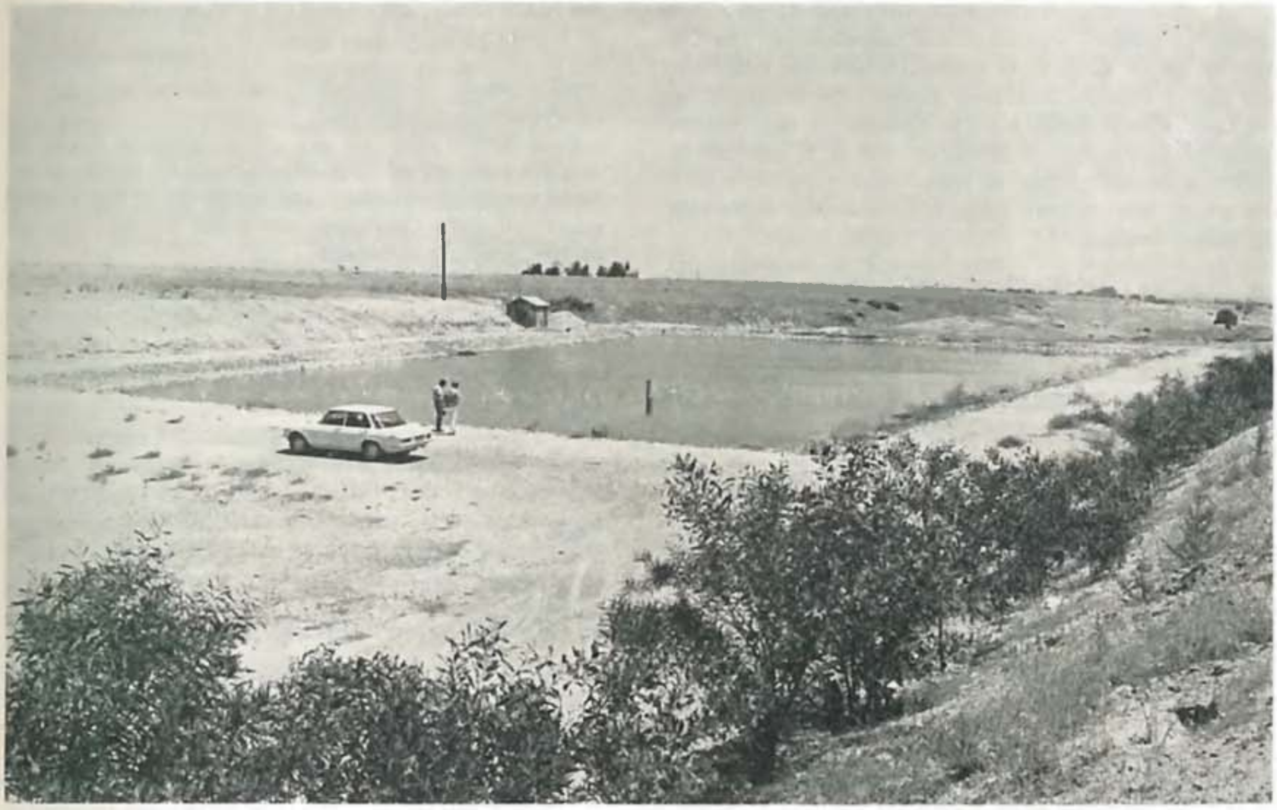
A number of small dams have been built throughout this region except in the Karpas peninsula where the brackish water problem has still to be overcome.

More dams have still to be built and their effect on the main regions previously outlined is not expected to be significant.

Also a programme has been recently initiated for the construction of earth reservoirs of limited capacity in hilly areas, to enable the irrigation of small areas of permanent crops. Such earth reservoirs have been recently built and covered with Polythene sheeting 0.25 mm thick, at Margo near Nicosia of 60,000 m^3 capacity, and at Kyperounda at an altitude of 1,400 m asl of 5 000 m^3 capacity.

The building of these storage reservoirs is preferred instead of dams in hilly regions because of the steep topography requiring costly structures for the storage of water and due to the sediment problem which is very great. On the other hand, off-channel reservoirs at suitable sites do not present such difficulties. A lot more of these reservoirs are now under planning for implementation in the near future.

Kyperounda Reservoir 1400 m asl. Excavated and earth fill. Polythene sheet cover 0.25 mm thick. Capacity 5,000 m^3.



Margo Reservoir: Excavated and earth fill. Polythene sheet cover 0.25 mm thick. Capacity 60,000 m³.

3. FINANCING

The Government of Cyprus for the past 13 years has been spending an average of about £0.75 million annually on dam projects which represents about 30% of the total annual funds spent on water development. This amount is expected to rise substantially during the next few years.

Various systems of financing water development projects in Cyprus are practiced. The following systems are used:-

a. Major Dams

Major Dams are financed at full cost to the Government, the costs including construction, investigation, design and planning. In evaluating the water charges levied on the beneficiaries, the investment costs are amortized over a period of 40 to 50 years at a rate of interest of 7%. The operation and maintenance costs are either paid by the Government and the expenditure included in the rate or, as preferred in some cases, the operation and maintenance of the headworks only, including the dam and main conveyor systems are paid by the Government and included in the rate, whereas the secondary and tertiary distribution and farm operation costs are paid by the beneficiaries.

The Government policy is to recover about 50% of the weighted average cost of all Government financed dams. However, the Government, taking into consideration the actual cost of each individual project, the economic and social conditions of the region concerned, the financial position of the people, the kind of crops which can be cultivated in the area and the benefits expected, may vary the Government contribution to between 33.33% and 80% of the overall Islandwide weighted average cost. The weighted average of the recently constructed Government financed major irrigation projects is 30 mils/m³ including the irrigation system which is very high according to international standards. A 50% contribution of this would give 15 mils/m³ which is what is now being accepted in Cyprus for high return crops such as citrus. Because there are limitations as to the maximum charges that can be paid by the beneficiaries for different types of crops, due to the repayment ability of the farmer, the Government has decided certain maximum charges which cannot be exceeded. These charges are as follows:-

Kind of crop	*Maximum charge mils/m³*
Cereals	5
Vegetables	10
Citrus and Bananas	20
Deciduous	20

The above described method of financing is preferred for major irrigation projects because it has many advantages such as:-

i. It encourages the efficient utilization of water because the proprietors will pay according to the amount of water which they consume.

ii. The headworks belong to the Government which results in efficient operation and maintenance. In the case of major dams it is necessary both for economic and safety reasons that such projects be managed by the Government which has the services of experienced technical personnel. It is known that it is difficult to convince private people to spend money on an urgent job which may require immediate attention especially for safety reasons.

iii. The establishment of water rights is eliminated. Government retains ownership of water which can be used for the enforcement of an all Island or basinwide water policy and promotes a general policy for the distribution of the available water resources according to priorities.

b. Minor Dams for Irrigation Projects

i. Government contributes towards works carried out for Irrigation Divisions (owners of land) according to the following principles:-

Winter crop irrigation schemes	80%
Spring crop irrigation schemes	75%
Perennial irrigation schemes	66.67%

ii. Government contributes towards works carried out for Irrigation Associations (owners af water) on a basis which depends upon the distribution of the shares of water amongst the proprietors as follows:-

Maximum water owned by 25% of share-holders %	*Government contribution* %	*Maximum water owned by 25% of shareholders* %	*Government contribution* %
25—30	66	66— 70	50
31—35	64	71— 75	48
36—40	62	76— 80	46
41—45	60	81— 85	44
46—50	58	86— 90	42
51—55	56	91— 95	40
56—60	54	96—100	38
61—65	52		

The Director of Water Development Department fixes the contribution acording to the principles described previously whilst the Government may accept a reduced contribution on the grounds of poverty, flood damages, failure of previous works and other similar circumstances and social reasons.

iii. The Government contribution is given as a grant whilst the beneficiaries' contribution is given by the Government as a loan the repayment period of which depends upon the financial position of the beneficiaries, their expected benefits from the scheme and on whether they have any other loans. If they, or any other authority in the village which has borrowed money from the Government have not been fulfilling their obligation and are in arrears with their loan instalments, no new loans can be granted to them until they pay their arrears. Arrears are paid at an increased interest of 7%. The normal rates of interest for small dams are:-

up to 5 years loan	4.5%
6—12 years loan	5%
13—15 years loan	5.25%
16—20 years loan	5.5%

No loan period normally exceeds 20 years. These loans are being examined and approved by Government appointed Loan Commissioners under the chairmanship of the Accountant-General.

The Loan Commissioners accept a relaxation period for paying off the loan until the full production from the project is reached. The Government also may accept reduced water rate charges during the developing periods.

In all the above cases except in the case of major dams, investment cost represents only the actual construction costs. Investigations, planning, design and supervision of construction are contributed by the Government whilst operation and maintenance are taken over by the beneficiaries concerned.

c. Town Domestic Water Supplies

In the case of town domestic water supply projects, the Government does not contribute. Town Water Boards are expected to pay the whole investment cost which is normally made available as a Government loan. The consumers then, through water charges applied by the Water Boards are expected to pay off investment, operation, maintenance and administration costs. The payment is charged per cubic meter and the rate is on an increased scale increasing as the consumption increases, from a minimum of 50 mils/m^3 to a maximum of 200.

d. Rural Domestic Water Supplies

In the case of rural domestic water supplies the Government constributes 50% of the total investment cost. The Government contribution is given as a grant and the village contribution as a loan to the village.

e. Groundwater Recharge Works

These are financed similarly as for minor irrigation projects.

f. Flood Protection Works

These are financed at full cost to the Government unless carried out for an Irrigation Division in which case the contributions previously stated apply.

g. Drainage Works

These are financed at full cost to the Government unless carried out for an Irrigation Division in which case the contributions previously stated apply.

h. Multi Purpose Projects

In the case of multi-purpose projects, the financing is based upon the principles previously described, that is to say, where a project is for irrigation and flood protection, the part of the project which is accounted for flood protection is paid at full cost by the Government whilst the part of the project which is accounted for irrigation is contributed according to the principles for irrigation project financing.

4. MANAGEMENT

a. Institutional

The collection of hydrological data, their interpretation, the planning, design and construction of all water development works on the Island is the responsibility of the Department of Water Development of the Ministry of Agriculture and Natural Resources.

The initiation for starting investigations and planning for a water development project may come either from the people concerned through their District Officer or from the Department and the Ministry. In deciding priorities for promoting the planning and construction of a project, the economic viability of the project is the main criterion. The interest of the people, social reasons and indirect benefits are also considered. Major projects should fit in a whole river basin regional study which itself should fit in the all Island-wide Master Plan.

The management of dams is carried out by a Project Committee under the chairmanship of the District Officer with an Engineer of the Department of Water Development and an Agriculturist of the Department of Agriculture as members. The Project Committee is responsible for the administration and management of the Project.

On a District level, District Water Boards may be appointed whose functions are:-

i. To make recommendations on the development, conservation, management and efficient use of water resources within the District;

ii. To have overall responsibility for the management, operation and administration of all projects within each District constructed entirely at Government expense;

iii. To maintain and operate the works within the District with a view to:-

Improving the standard of agricultural practices, improving the methods of irrigation, increasing the revenue from land and water to the full economic value, controlling the expenses of maintenance and management of the waterworks to the best possible degree and making recommendations for the optimum use of Government land within the area commanded by the waterworks.

iv. To be also responsible for the regulation, use and fair distribution of water resources and for undertaking the sale of water on behalf of the Government, by imposing the water rates decided by the Government.

v. To be responsible too for the preparation of the budget for every waterwork in the District, showing the revenue and expenditure in connection with the operation and maintenance of each project.

For every project an Irrigation Division is formed which is made up of the beneficiaries in the project. They have their own committee elected and are responsible for the management of the irrigation system on the farm level. They may also be required to collect water rates on behalf of the Project Water Committee or they may purchase in bulk the water from the Committee and make their own arrangements for selling the water to the beneficiaries.

The principle of this management procedure is that the Government retains control over the headworks and main conveyor system to enable more efficient operation and maintenance, the enforcement of an all Island or basin-wide agroeconomic policy and the better distribution of available water resources according to priorities, whilst on the other hand it recognizes that the farmers should be given a free hand to manage their own works as much as possible.

In case of minor projects which do not involve the construction of major dams, the irrigation systems are financed on a contributory basis so that the Irrigation Division signs a loan being a percentage of the investment cost. By this method, the Irrigation Division is responsible for collecting the loan instalments from every beneficiary who is required to pay according to his benefits derived from the project. Where more than one villages or independent areas are benefited by a single project, more than one Irrigation Divisions may be formed for the same project.

In Cyprus we have also Irrigation Associations on the same level as Irrigation Divisions which are formed of farmers who are owners of water and have a registered water title deed or ab-antiquo water right, The Government does not encourage the construction of major water development works for Irrigation Associations unless the water owners relinquish their water rights. However, works of a minor character, such as for repairs and improvements may be undertaken.

Domestic water supplies are managed by Municipal Water Boards in the case of the four main towns of Cyprus, namely Nicosia, Limassol, Famagusta and Larnaca whilst in the case of all the other rural communities of the Island, they are managed by Village Water Commissions. The Municipal Water Boards function under the chairmanship of the District Officer who heads a Committee, members of which are the representatives of the Department of Water Development, of the Accountant-General and elected representatives from the Municipality of the town concerned. The Water Boards have enough administrative and technical personnel to deal with the day to day routine administrative and technical management of their affairs. However, where major water development issues are concerned, it is normal for the Department of Water Development to undertake the necessary investigations and plans. The Water Boards are also responsible for the collection of the water rates which every consumer of the town has to pay. The water consumed is always measured by automatic water meters.

In the case of rural water supplies, these are administered by the Village Water Commissions, made up of the District Officer as Chairman with elected representatives from the villagers as members. This Commission is responsible for the management of the domestic water supply works of the village and is also responsible for collecting the loan instalments and paying off the loan signed in connection with the execution of the water supply works.

b. Legal

There is no special legislation regarding dam building in Cyprus, but there are many laws dealing with water which have been enacted from time to time in the past as the need had arisen. An effort is now being made to modernize and codify this legislation.

The most important laws which are more concerned with dam building and management are:

i. The Government Waterworks Law

This law vests all groundwater and all water running to waste from rivers to be the absolute property of the Government. It gives power to the Government to store, divert or otherwise use and control all Government water resources and to construct any water works as required. Under this law Project or District Water Boards can be established for the regulation of the Government projects. It empowers the application of water charges to be levied on the consumers. Also under this law any water rights or other property can be evaluated and acquired if required in connection with the proposed works.

ii. Water Development and Distribution Law

According to this law the Council of Ministers can declare an area as a development area and appoint a Development Committee for the purpose of taking over and acquiring all water rights and all waterworks as required for carrying out schemes for the proper water distribution and water development of that area.

iii. Public Rivers Protection Law

This law gives power to the Government for the protection of rivers against dumping of any materials, excavation in the river bed and pollution of the rivers.

Other important water related laws but not directly dealing with dams are:

i. The wells law and connected legislation for issuing of drilling permits and controlling the use of water.

ii. The Irrigation Divisions and Irrigation Associations laws in connection with the establishment of Irrigation Divisions and Associations and the regulation of the use of water for irrigation.

iii. The Village Water Supply law for the regulation of rural water supplies.

iv. The Municipal Water Supply law for the regulation of town water supplies.

c. Maintenance and Safety Considerations

The importance of the safety of dams cannot be overlooked and the necessary safeguards should be ensured so that the life of people, animals, property downstream and the dam structure itself, do not suffer any damages.

Routine inspection by the operating and Head Quarters staff has to be made under the supervision of the responsible Engineer. The main features which should be given close examination are settlement of the embankment, structural deformations, rip-rap condition, downstream erosion, drains, leakage, concrete deterioration, sedimentation of the reservoir, grading of the river bed downsteam, gates and other mechanical parts.

As a matter of fact, the first impounding of the reservoir is the first actual test to prove the safety of the structure. A successful first impoundment however, does not prove that the dam will be safe in the future, unless the time comes when it will be subjected to the severe conditions envisaged in the design such as maximum floods, sudden settlement conditions, earthquakes etc. For the accurate determination of any defects the availability of suitable instrumentation on the dam is of great importance. For the large dams, piezometers have been installed both upstream and downstream of the dam to measure the water level. In the case of Yermasoyia, Mavrokolymbos and Lefkara dams pore water and total pressure cells have been installed in the foundations and the embankment.

Good record keeping is also very important for proper safety control measures to be effective. It is essential that whenever possible complete details should be kept about the dam, including design and completion reports and of all routine supervisions and inspections that are undertaken during the process of construction. In Cyprus a standard has been issued for routine and special inspection requirements which are being followed by the responsible Engineers and Technicians for this purpose.

To help the various Countries lay down effective measures for the safety and supervision of dams, the International Commission on Large Dams has issued its bulletin GR-Q38 on the "Supervision of dams and reservoirs in operation". Similarly UNESCO has issued a report entitled "Recommendations concerning reservoirs in operation". Finally UNESCO has regulatory powers that must exist in order to implement effective measures and safety. For the time being, there are enough of such regulatory and institutional facilities in Cyprus but as our construction programme increases and larger dams are built, there is no doubt that further institutional and legal provisions have to be made in order to safeguard the safety and proper maintenance of dams.

BASIC STUDIES AND DATA

In planning for dams the aspects that have to be considered and studied by the Engineer, are many and complex and involve the knowledge of a number of technical sciences such as hydraulics, hydrology, structural engineering, soil mechanics, concrete technology, engineering geology, foundation engineering, mechanical engineering, engineering, economics etc. Dams are costly structures which have to be properly planned so that they would serve fully the purpose for which they have been constructed, otherwise not only considerable funds of money may be wasted but in addition these structures may be turned from beneficial to a potential threat to the property and life of people living downstream.

TENTATIVE WATER BALANCE (For an average Year) TABLE 2

Ser No.	Utilization	Consumption and Extent of Land																			
		Surface Runoff Excluding River Bed Infiltration				Ground Water Including River Bed Infiltation								Evaporation and Evapotranspiration plus Consumption by Direct Rainfall				Totals			
						Pumped and Subsurface				Springs											
		Million m^3	%	Extent of Land thousand donums	%	Million m^3	%	Extent of Land thousand donums	%	Million m^3	%	Extent of Land thousand donums	%	Million m^3	%	Extent of Land thousand donums	%	Million m^3	%	Extent of Land thousand donums	%
1.	Irrigated crops including irrigated cereals	150	3.3	435	6.3	332	7.2	270	4	27	0.6	30	0.4	130	3	690	10	639	14.1	690	10
2.	Dry farming crops (cereals, carobs, olives, tobacco, vines)													900	20	2,000	29	900	20	2,000	29
3.	Forests, shrubs													900	20	1,400	20	900	20	1,400	20
4.	Pastures, grass													450	10	1,200	17.5	450	10.0	1,200	17.5
5.	Evaporation from bare land, fallow land, built up areas, water surfaces													1,270	26 4	1·610	23.5	1,270	26.4	1,610	23.5
6.	Domestic, animal husbandry and industrial water					35	0 8			6	0.2							41	1.0		
7.	Ground water over extraction					—120	—3.											—120	— 3		
8.	Subsurface losses					70	1.5											70	1.5		
9.	Surface losses to the sea	450	10															450	10		
	Totals	600	13.3	435	6 3	317	6 5	270	4	33	0 8	30	0 4	3,650	79.4	6,900	100	4,600	100	6,900	100

(1 hectare = 7.5 donums)

Note:- Total Rainfall 4,600 Million m^3
Total Land 6,900,000 donums

Important technical studies in dam planning are:-

1. HYDROLOGY

a. Water Resources — Selection of Dam Reservoir Capacity

In selecting the storage capacity of a dam, the limiting conditions may be topography, geology, hydrology, water requirements and economics all of which have to be carefully assessed. A topographic survey is naturally the first work to be done. In Cyprus the existing topographic conditions as can be seen from Fig. 3 are unfavourable. In the highlands where water is available, river beds are very steep and render storage very expensive while in the plains where reasonably good dam sites exist only spate flows are occasionally available.

Regarding hydrology, the most accurate assessment of river flows has to be done through a long period of actual measurements of flow. Unfortunately in Cyprus accurate direct measurements through automatic flow recorders or current meters started to be taken for the past 20 years only in a limited number of cases which gradually increased. However, rainfall data are widely available for the past 60 years which is considered a good period for hydrologic evaluations. Therefore, for dam planning, hydrological studies rely heavily on the rainfall data and indirect methods for the estimation of runoff.

Table 2 shows an estimate of the water balance for an average year of rainfall. It can be seen from the table that in an average year it is estimated that the rainfall is 4,600 million m^3. The distribution of rainfall (Fig 4.) shows a close relationship of the isohyetal lines with the topographic contours given in Fig 3. Out of the total rainfall, 950 million m^3 are estimated to be useable water resources, of which, 600 in the form of surface runoff and 350 in the form of groundwater including overflow through springs. The balance of rainfall being about 80% is accounted for by evaporation and evapotranspiration (Ref. Table 2).

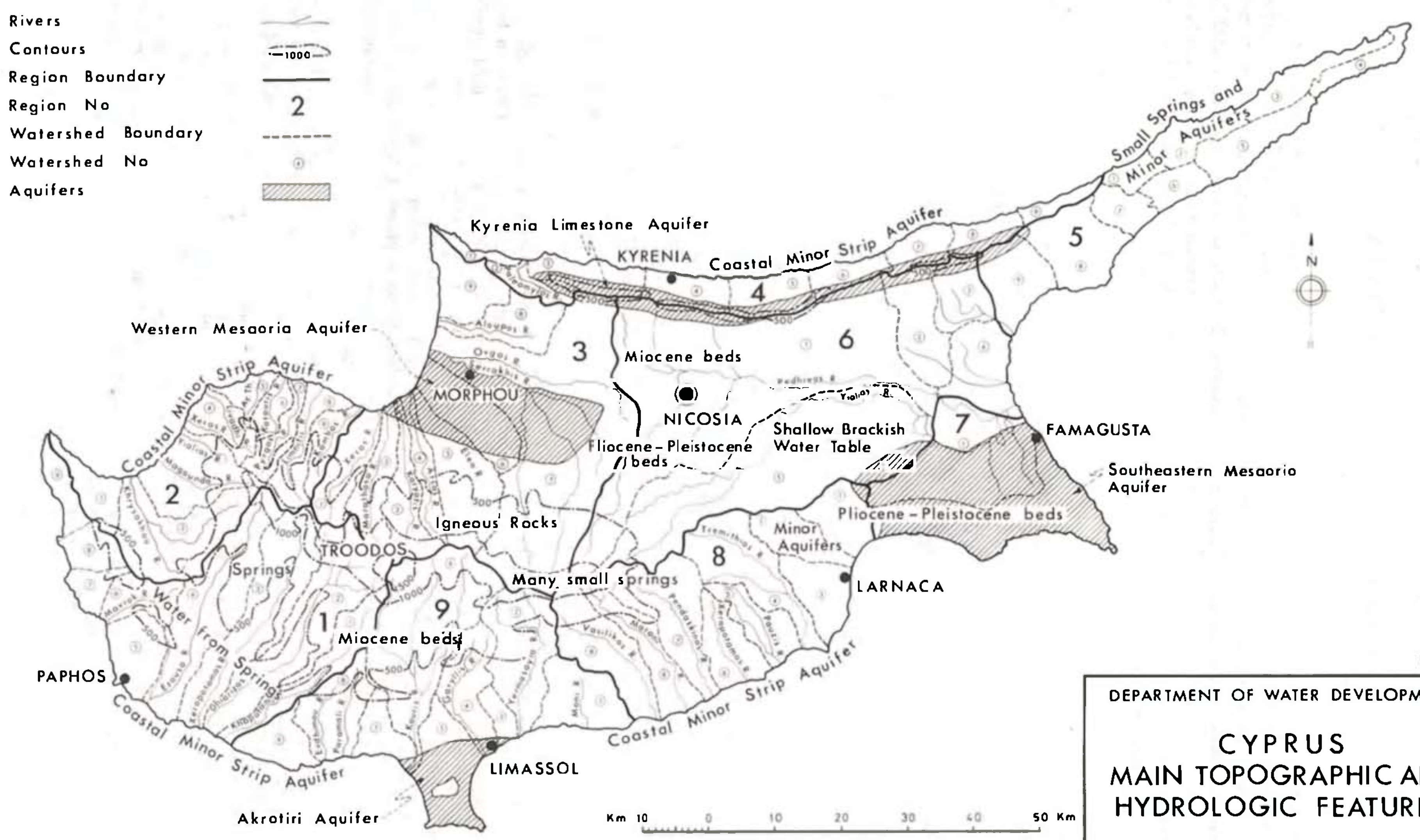


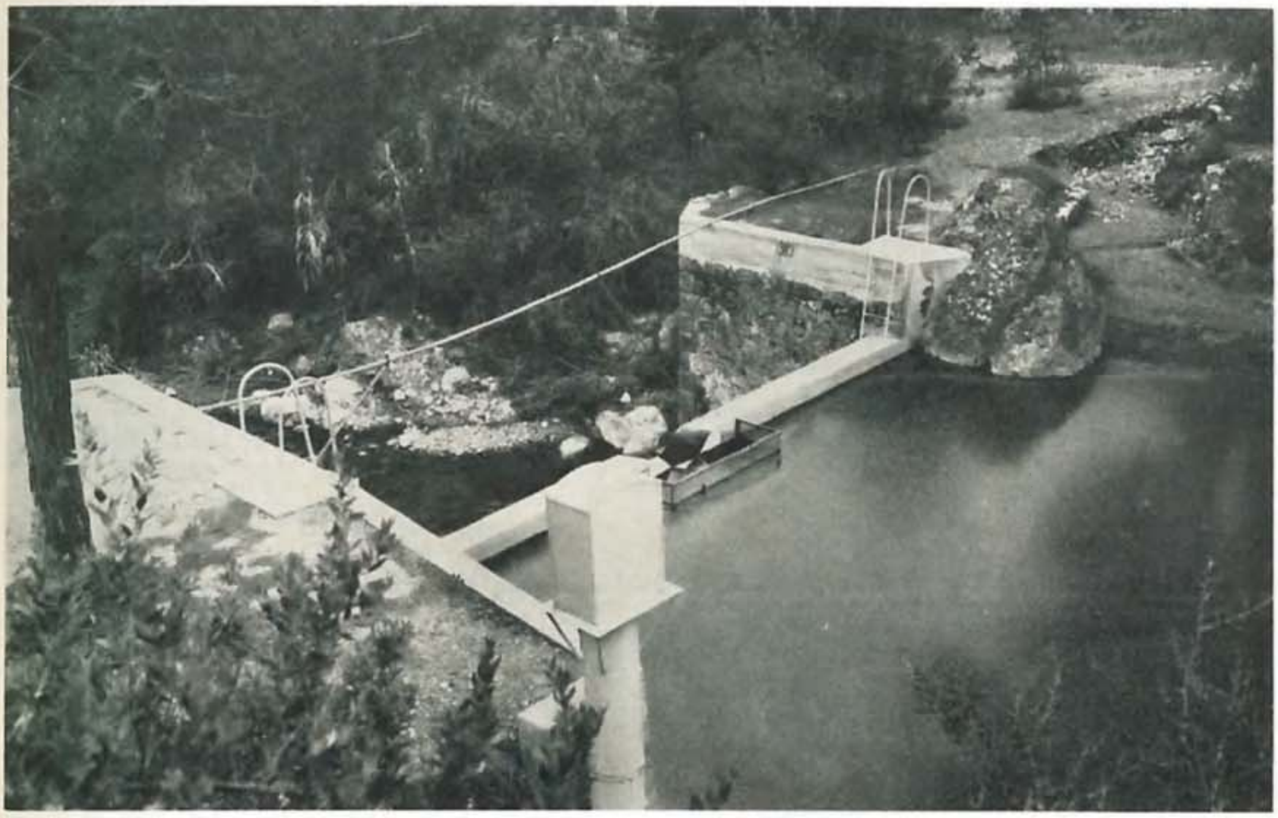
Fig. 3

DEPARTMENT OF WATER DEVELOPMENT

CYPRUS
MAIN TOPOGRAPHIC AND HYDROLOGIC FEATURES

DECEMBER 1966 DRG. No. BL/G/1

Current meter measurements of flow at Kythrea Kephalovrysos spring.



Measuring weir at Thermia River showing V - notch, screen baffle, stilling basin and automatic flow recorder.

The runoff distribution given in Fig. 3 occurs through about forty important watersheds radiating from the Troodos range and traversing the plains downhill, finally reaching the coast, mainly in the Western half of the Island. The largest river as regards flow is estimated to be the Kouris river which has a gross mean annual flow of 58 million m3. There are big interannual fluctuations of the runoff and there are long periods of drought recorded. For example, in the case of the river Kouris where it is possible to built the largest dam in Cyprus up to a capacity of 100 million m3, the probability is 10% for the flow to exceed 50 million m3 and to be less than 24 million m3 as compared with the 58 million m3 average.

From up to date hydrological records, Fig. 5 gives the relationship of rainfall to runoff for four slope categories, under normal geologic and vegetative conditions of Cyprus, which can be used for preliminary runoff estimates.

However, for detail design work a rainfall-runoff mathematical model is being used which is programmed on the computer so that answers can be rapidly produced. This model incorporates parameters such as precipitation, interception, evaporation, soil moisture and deep percolation being calibrated with actual river flow meansurements. This computerised system, enables the simulation of stream flows from daily rainfall data over 55 years and facilitates the study of multiannual operation of dams within minutes, which would otherwise require several days of work to achieve.

Flow duration curves can also rapidly be drawn, for evaluating diversion conveyor capacities and power potentialities.

The mass curves are also readily produced indicating cummulative flows. The demand curve combined with flow on the mass curve gives storage required.

In the final capacity evaluation, the necessary adjustment for evaporation and sedimentation have of course to be applied. In addition allowances have to be made for settlements and wave action, more particularly where spillway freeboards are considered.

For a given demand, computerized reservoir operation programmes, rapidly indicate the extent of daily utilization of the river flow and hence the surplus or deficiency of water and the dependability of the storage for the purpose of the demand. Depending on the desired dependability of the water supply, the optimum dam capacity can be evaluated, due regard being made for evaporation and sedimentation.

b. Water Use

Water in dams may be used for irrigation, domestic water supply, flood protection, groundwater recharge, power and recreation.

In Cyprus dams have been mainly used for irrigation purposes. The second important use has been their role as check dams for purposes of groundwater recharge of aquifers either in downstream spreading grounds or through the reservoir bed itself when properly maintained. Recently, dams are being built for domestic water supply purposes, as for example the major dam at Lefkara.

It can be seen from Table 2 that out of the 510 million m3 presently exploited, only 41 million m3, that is to say about 8% is used for domestic and industrial purposes. Out of this 41 million m3, only the 6 million m3 of the Lefkara dam is used from surface storage schemes. However, as the groundwater resources have been overexploited, it is natural that a lot more water for domestic purposes will be coming in the future from surface storage schemes. Furthermore, the increased demand to meet urban development, the expan-

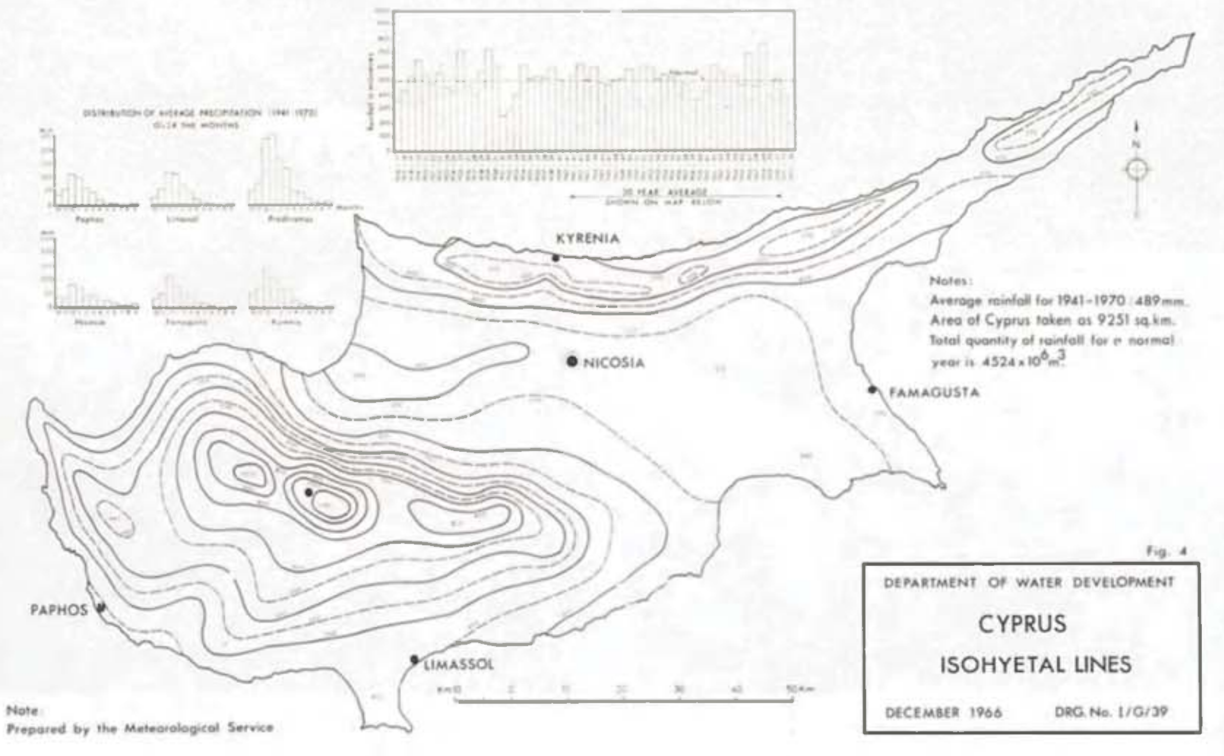
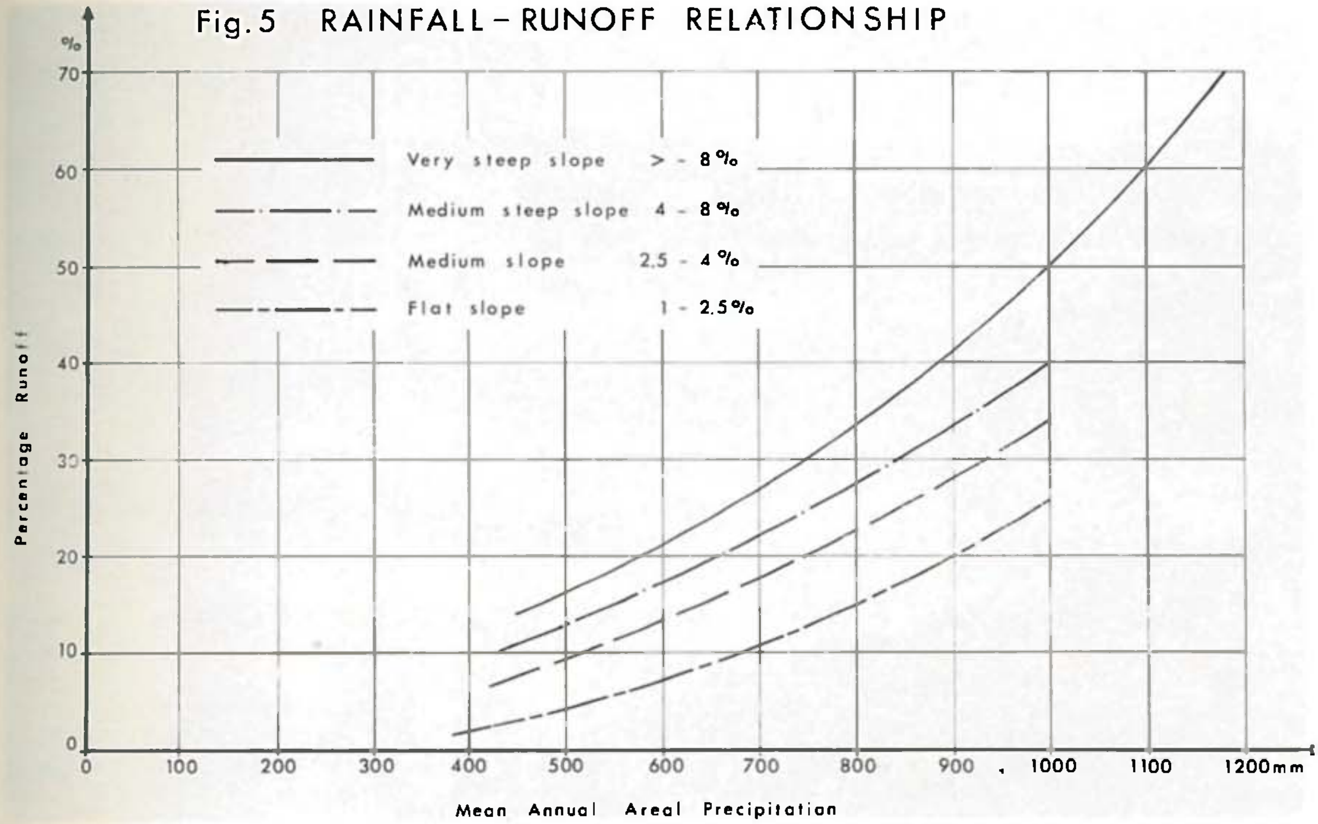


Fig. 4



sion of industries and flourishing tourism, will mean that the percentage of surface water utilization for domestic purposes will be considerably increased in the years to come.

From Table 2, again, it can be seen that the quantity of surface flow presently utilized is only 150 million m^3. Out of the 450 million m^3 balance it is estimated that an amount of 300 to 350 million m^3 can be utilized by constructing storage reservoirs. The remaining should be accounted for by evaporation and transpiration losses in the river valleys. Comparing with Fig. 1 which shows the storage presently achieved to be only 64 million m^3, it can be seen that an extensive programme of dam building lies ahead, which may result in our present storage capacity becoming five times as big.

It is natural that with the extension of urban development and industries, pollution of water resources will take place and measures have to be taken to safeguard against such eventualities.

The possibility for power development, as mentioned previously, is very limited, but it is anticipated that in certain special cases dams may be used for limited production of energy.

The recreational use of dams and the protection of environment are also important considerations which will contribute to enhancing the value of our projects. Already studies have been made for using the Yermasoyia dam for recreational purposes such as sailing and water skiing. Fish culture has been introduced in many of our dams, especially carp which seems to grow well whilst trout is being cultivated at the Kalopanayiotis Dam where the water temperature and aeration conditions are suitable.

c. Flood Studies

The fact that a great percentage of the dam failures in the world especially where earth embankments are involved is attributed to inadequate provisions for the disposal of floods, makes the study of floods to be one of the most serious aspects in dam design. The estimation of floods however, is very complex, and many components of the watershed are required, such as a long record of rainfall intensities and climatic factors, as well as the topographic, geologic and vegetative characteristics. There can never be an 100% accurate estimate of a flood because of the fact that many of the components cannot be accurately known. Many formulae have been used for evaluating floods which have international application, but for any formula to be sound, it should take into account the local conditions. Both "normal maximum floods" and "maximum probable floods" should be considered in the design.

Normal maximum floods form the basis for the design and should usually be of an occurrence of one in a thousand years. However, maximum probable floods should also be considered depending on criteria such as importance of the dam itself and the extent of likely damages downstream, with more emphasis on the possible loss of life. Although the probability for the occurrence of maximum probable floods is very small, yet such floods cannot be excluded and can occur in a catchment area where such floods have never previously been recorded.

A method in flood study largely used in Cyprus is the one introducing the climatic and physical features of the watershed such as geology, soils, vegetative cover,

Pedhieos River in flood through Nicosia.

river bed length and slope. Using the above components and the climatic factor for the dam location, the maximum discharge is evaluated for any probability of occurrence from available curves for various runoff factors.

With the extensive rainfall records available in Cyprus, however, and the growing reliability of runoff data, another method, based on precipitation and runoff data, is gaining ground.

Using the available storm rainfall and runoff data in the river catchment, a hydrograph is drawn of the discharge in m^3/s versus time in days, from which the base flow contribution is determined and deducted, so that the storm runoff is evaluated and converted to mm depth over the cachment area.

An envelope curve of the determined storm runoff in mm is then drawn against corresponding values of storm precipitation over the catchment in mm.

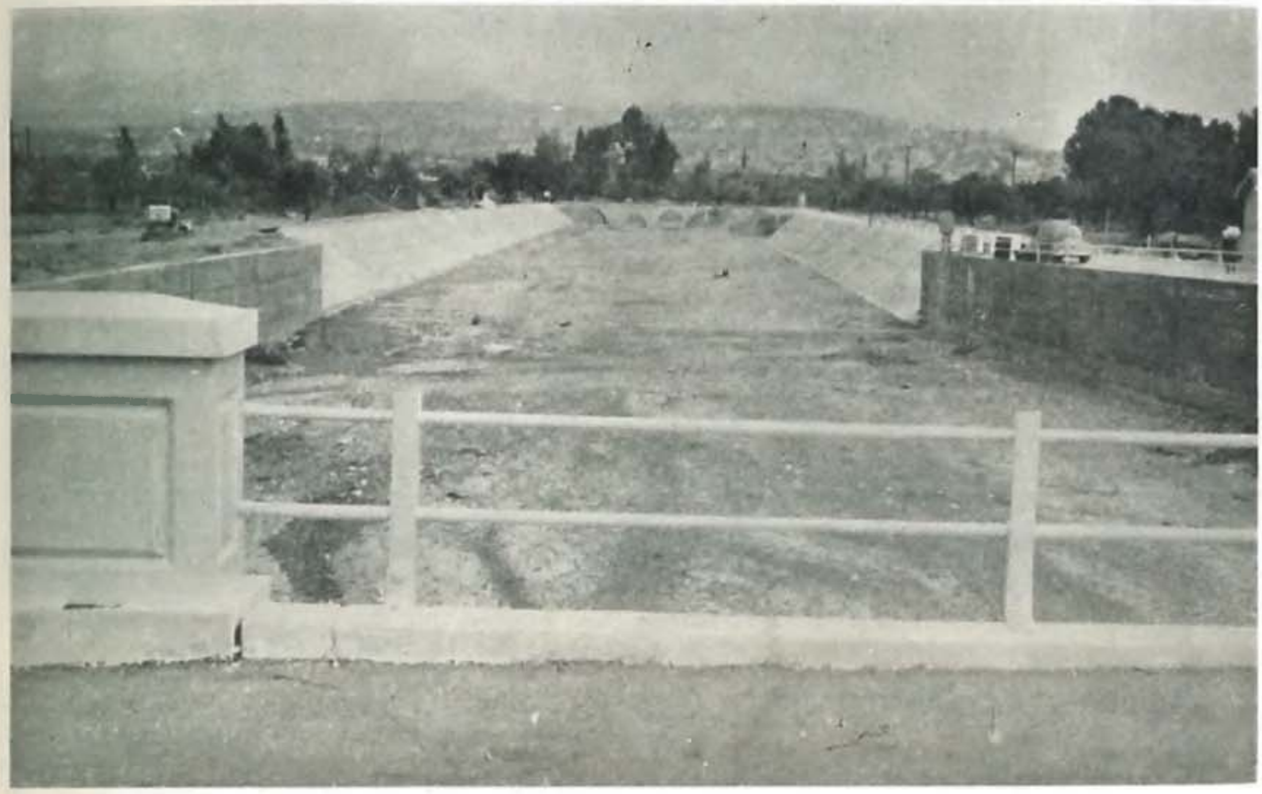
The observed flood peaks of the storms are plotted against depth of storm runoff as calculated earlier to form another envelope curve for the worst conditions.

A nomogram is finally drawn giving the relationship between the 24 hour storm precipitation over the catchment and peak discharge.

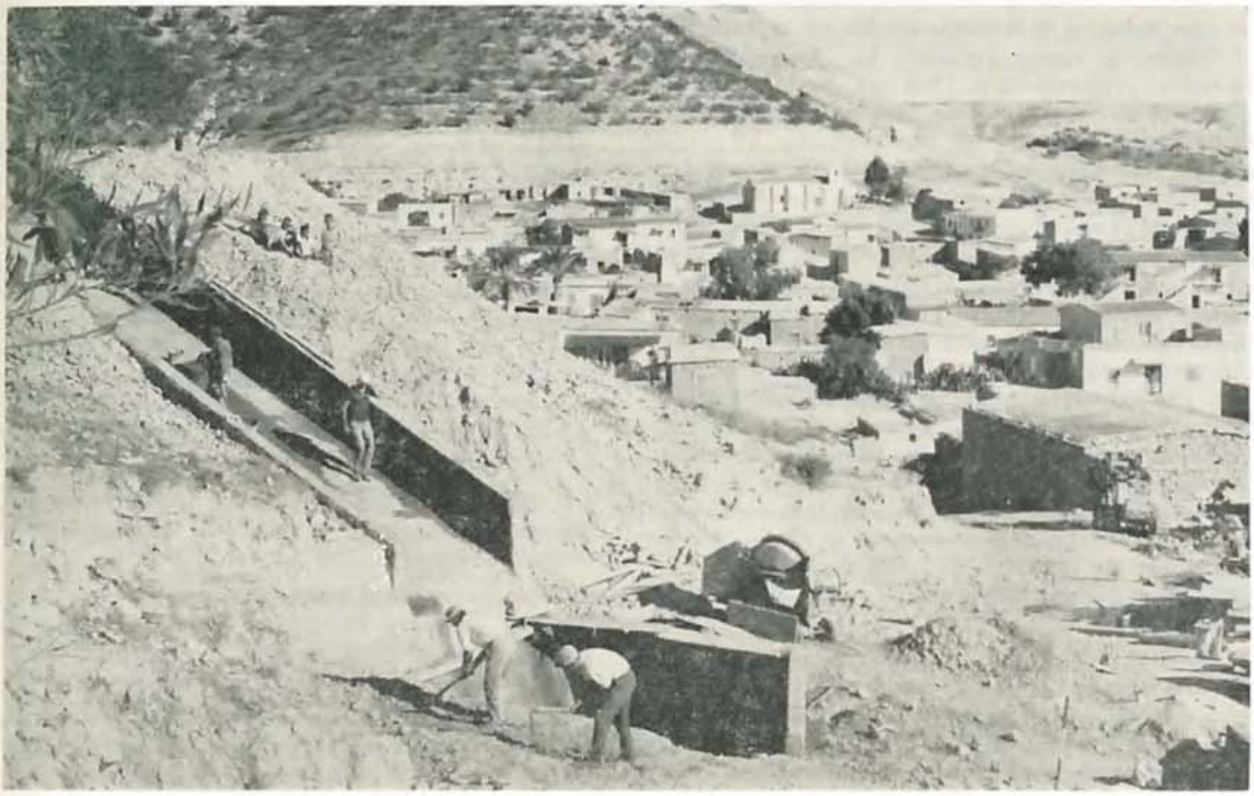
The "probable maximum flood" is determined from the 'probable maximum precipitation". The latter is calculated through an analysis of long records of 24 hour precipitation at several stations in Cyprus to obtain the mean and the standard deviation for each station. Using the maximum 24 hour fall for each station and the mean and standard deviation for the same station, frequency factors are then calculated. The maximum value of this frequency factor obtained is 10.1.

However, in the determination of "maximum probable precipitation" amounts, a value of 15 was adopted for the frequency factor, which complies with the suggestion of other investigators. The "maximum probable precipitation" is obtained by the addition to the mean maximum 24 hour precipitation of fifteen times the standard deviation.

Introducing this maximum probable 24 hour precipitation in the nomogram the maximum probable flood may be determined. Floods of other recurrence intervals may be determined in a similar manner by using the nomogram of the 24 hour precipitation for the desired frequency.



Yermasoyia River Training by canalization and concrete slab facing of banks.



Voroklini village peripheral flood diversion canal.

Application by boat of Cetyl Alcohol in the Yermasoyia Dam Reservoir for the reduction of evaporation rate. In the foreground the treated part can be seen with the film cover showing distinctly.

2. RESERVOIR EVAPORATION LOSSES AND CONTROL

Evaporation losses are significant in Cyprus especially in the low lying valleys where temperatures are high and considerable allowance has to be made in the design to meet these losses. As the construction of dams with larger reservoir areas increases in Cyprus, evaporation losses become more appreciable.

Based on the Smithsonian meteorological tables and on solar radiation, it is estimated that for latitude 35°N of Cyprus, the energy available for evaporation is enough to evaporate up to 1,700 mm of water annually from a rather large surface. By large surface it is meant larger surfaces than our reservoirs in Cyprus expose. For a limited exposed surface such as our dam reservoirs, a coefficient of 0.65 of the figures obtained from solar radiation has been shown to be reasonable.

Fig. 6 and Table 3 give the average rainfall, temperature, potential evaporation and evapotranspiration in various regions of Cyprus. From an analysis made of the Cyprus dams it has been found that in the case of canyon type dams (table 4) which are deep with steep sides and short crest length such as are situated on upstream valleys, the evaporation varies from 7 to 11% giving an average of 8%, whereas in the case of shallow flat reservoirs (table 5) such as are situated in the lower parts of the river valleys used for part of the year only, the evapotranspiration is between 3.5 to 6.5% giving an average of 5%. Finally, for our major dams in the valleys considering multiannual operation, the evaporation can be as high as 15%.

As water in Cyprus is scarce, measures to control evaporation are of great importance. Evaporation control by the use of retarding agents has already been tested as in the case of Yermasoyia dam, elevation 60 m asl and Prodromos reservoir at 1,500 m asl.

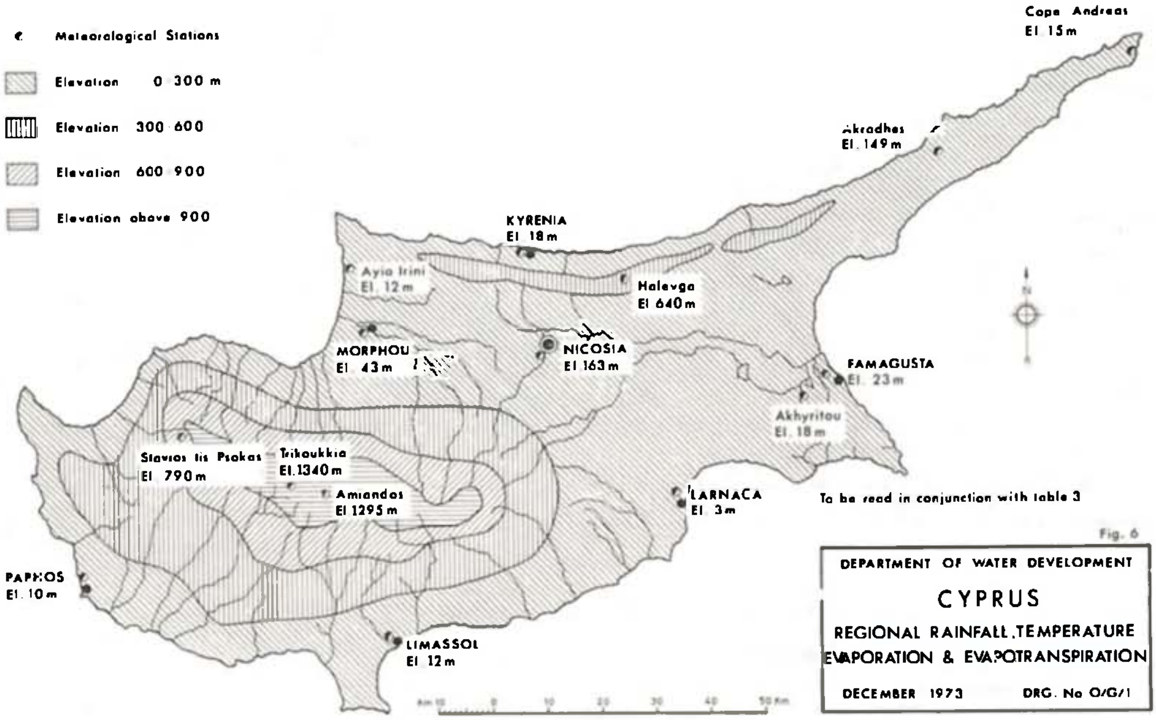
The evaporation retardant use is a fatty alcohol made of a mixture of hexadeganol and octodeganol blended with a dispersing agent. It is not toxic to humans or animals drinking the water or swimming in it. The hexadeganol film is only half a thousandth of a millimetre thickness. Although it prevents evaporation, it does not prevent rain entering it, but the holes made though it heal themselves and close up after the rain is over. It is important to have the hexadeganol at a pressure in the order of 40 dynes per centimetre because evaporation reduction is greatest in that range. Other problems are that evaporation reduction decreases with the rise of temperature, whilst wind and wave action are critical in maintaining film coverage. The destruction of hexadeganol by bacteria is yet another problem and after the water attains a temperature of 26°C, the amount of hexadeganol required by bacteria appears to increase in a geometric ratio.

The application of the film is made more effectively from a boat from where it is spread on the surface of the reservoir. Many times the application has to be done daily in order to maintain a good coverage. A maximum conceivable coverage could be 90% of the reservoir area, whilst a good average cover is of the order of 60%. It appears that the most economical application can be achieved if the hexadeganol is applied for a period of six weeks followed by a period of 3 weeks of remission with a repeated application.

For an effective maintenance of film, it may be necessary to apply up to 1 kilo of hexadeganol per hectare of reservoir area per month.

In the case of Yermasoyia dam it has been estimated that the water saved from evaporation during the period of hexadeganol application was 32% whilst the cost of the water saved was 5 mils/m^3.

Evaporation retarding tests have still to be made in Cyprus to a larger scale and for longer periods in order to assess the economics of using retarding agents, and before such control measures could be widely introduced.



REGIONAL EVAPORATION DATA

(Ref. Fig. 6)

TABLE 3

Region	TEMPERATURE °C (T) EVAPOTRANSP. (mm) (Ep) EVAPORATION (mm) (E) RAINFALL (mm) (R)	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
0—300 m.	T	11.72	11.83	13.28	16.50	20.50	24.39	27.05	27.28	25.05	21.50	17.39	13.50	
	Ep	24.89	25.4	32.25	50.29	79.24	113.53	140.71	143.25	119.88	87.37	56.38	33.27	906.52
	E	47.24	48.26	61.21	95.50	150.62	215.64	267.46	272.28	227.83	166.11	107.18	63.24	1722.62
	R	97.02	66.29	39.62	20.57	16.76	4.57	0.01	0.76	5.58	28.70	45.97	109.22	436.11
300—600 m.	T	10.11	10.44	11.56	15.00	19.44	23.33	26.11	26.11	24.22	20.22	15.17	11.56	
	Ep	21.84	23.36	28.19	46.22	75.18	105.91	131.06	131.06	113.79	81.02	47.24	28.19	833.12
	E	41.40	44.45	53.59	87.88	142.74	201.16	248.92	248.92	216.15	153.92	89.66	53.59	1582.42
	R	126.74	97.79	58.42	28.95	22.86	7.62	22.86	15.24	6.60	35.30	52.07	134.62	574.80
600—900 m.	T	8.55	8.94	9.78	13.44	18.33	22.22	25.11	24.94	23.28	18.89	12.94	9.56	
	Ep	19.55	21.08	24.63	42.16	71.88	99 82	123.19	121.66	108.20	75.69	39.62	23.62	771.14
	E	37.08	40.13	46.73	80.01	136.65	189.73	234.18	231.14	205.48	143.76	75.18	44.95	1465.07
	R	160.27	126.23	77.47	37.33	30.22	14.22	4.57	3.81	9.90	41.40	59.43	159.25	724.15
over 900 m.	T	3.44	4.22	6.72	10.50	14.78	19.05	22.11	22.28	19.05	14.55	9.05	5.22	
	Ep	8.63	11.43	21.59	39.62	63.24	89.66	109.72	110.99	89.66	61.97	32.25	15.24	654.05
	E	16.51	21.84	41.14	75.18	120.14	170.43	208.53	210.82	170.43	117.85	61.21	28.95	1243.07
	R	195.07	146.55	91.69	45.97	35.56	17.27	6.60	5.33	11.43	46.48	68.07	197.86	867.91

EVAPORATION FROM RESERVOIRS TYPE "A"

(Canyon type Reservoirs — Perennial Irrigation)

TABLE 4

Serial No.	Dam	Flow at dam-site Average year mill. m^3	Capacity of dam mill. m^3	Net storage in average year mill m^3	Land to be irrigated perennially donums	Monthly Water Use and Evaporation in million m^3														Evaporation %
						April		May		June		July		August		September		October		
						Water Use	Evap.	Water Use	Evap.	Water Use	Evap.	Water Use	Evap.	Water Use	Evap.	Water Use	Evap.	Water Use	Evap.	
1	Argaka	5.80	1.15	1.9	1470	0.16	0.02	0.21	0.02	0.26	0.03	0.32	0.04	0.04	0.03	0.25	0.01	0.15	0.01	8.1
2	Pomos	2.01	0.86	1.47	1160	0.13	0.01	0.17	0.02	0.21	0.03	0.25	0.02	0.03	0.02	0.19	0.01	0.11	0.01	7.4
3	Ayia Marina (Tyllirla)	0.31	0.31	0.31	235	0.02	0.01	0.03	0.01	0.04	0.01	0.05	0.01	0.06	0.01	0.04	0.01	0.03	0.01	11.0
4	Lefka	6.70	0.37	0.58	455	0.05	0.01	0.07	0.01	0.08	0.01	0.10	0.01	0.01	0.01	0.07	0.01	0.05	0.01	8.2
5	Kalopanayiotis	6.70	0.39	0.71	860	—	—	0.10	0.01	0.12	0.02	0.16	0.02	0.02	0.02	0.12	0.01	—	—	10.0
6	Polemidhia	5.36	3.57	3.86	3100	0.34	0.02	0.45	0.04	0.54	0.06	0.68	0.06	0.80	0.05	0.51	0.03	0.32	0.01	7.0
7	Trimiklini	12.5	0.31	0.34	665	—	—	0.07	0.01	0.10	0.01	0.12	0.01	0.11	0.01	0.09	0.01	—	—	7.6
8	Mavrokolymbos	3.26	2.23	2.18	1980	0.21	0.02	0.28	0.03	0.34	0.04	0.44	0.05	0.51	0.04	0.32	0.02	0.22	0.01	8.2

Take average evaporation from Type "A" as 8%.

EVAPORATION FROM RESERVOIRS TYPE "B"

(Shallow Reservoirs — Winter and Spring Irrigation)

TABLE 5

Serial No.	Dam	Flow at damsite average year mill. m^3	Capacity of Dam mill. m^3	Net storage in average year mill. m^3	Land to be irrigated perennially donums	Monthly Water Use and Evaporation in mill. m^3								Evaporation %
						February		March		April		May		
						Water Use	Evap.	Water Use	Evap.	Water Use	Evap.	Water Use	Evap.	
1	Morphou (Serakhis)	6.70	1.88	3.57	8000	0.17	0.03	0.62	0.03	1.12	0.01	1.51	0.04	4.3
2	Kiti	3.57	1.61	2.37	5300	0.11	0.01	0.42	0.02	0.73	0.02	1.03	0.03	3.5
3	Mia Milea	0.25	0.36	0.25	550	0.01	0.01	0.04	0.01	0.07	0.01	0.10	0.01	6.5
4	Liopetri	0.54	0.34	0.40	900	0.02	0.01	0.07	0.01	0.12	0.01	0.17	0.01	5.4

Take average evaporation from Type "B" as 5%.

Meteorological Station at Agricultural Research Institute, Athalassa

Dam	Capacity Million m³	Catchment area km²	Years of operation	Total reservoir inflow Million m³	Average annual inflow Million m³/a	% of Capacity to average annual inflow.	Total sediment Million m³	% of total sediment to total inflow.	Average annual sediment — Million m³/a	% of annual sediment to capacity.	Annual sediment per km² of catchment m³/a/km²	Average slope of catchment %.	Geology	Vegetation
Mia Milea	0.36	7.8	9	1.5	0.17	210	0.040	2.70	0.0045	1.25	578	10.5	20% Hard sedimentary 80% Soft sedimentary	Fair
Argaka	1.15	50	9	50	5.5	21	0.170	0.34	0.0190	1.65	380	7.4	Hard igneous	Forest partly burnt by fire.
Pomos	0.86	36.3	7	32	4.6	18.6	0.030	0.10	0.0043	0.50	118	7.4	Hard igneous	Forest
Mavrokolymbos	2.18	39	7	22	3.2	68	0.060	0.27	0.0085	0.39	218	3.9	Soft igneous and sedimentary clays	Fair
Yermasoyia	13.6	156.7	5	96	19.3	70.5	0.130	0.14	0.0260	0.19	166	3.9	Hard igneous Small part hard chalks	Good Vegetation 25% Forest
Morphou	1.88	458	11	125	11.5	16.5	1.33	1.05	0.1200	6.4	240	3.4	50% Hard igneous 50% Soft sedimentary	Fair 10% Forest
Polemidhia	3.86	85	8	42	5.2	74.5	0.150	0.36	0.0190	0.24	224	3.4	20% Hard igneous 80% Hard chalks	Fair
Ovgos	0.85	212	9	32	3.5	24	0.170	0.53	0.0190	2.25	90	2.0	Soft sedimentary	Sparse
Kiti	1.61	150	9	35	3.9	41.5	0.150	0.43	0.0166	1.03	110	2.0	50% Igneous 40% Chalk 10% Soft sedimentary	Fair
Athalassa	0.79	33.7	11	4.5	0.4	198	0.04	0.90	0.0037	0.47	108	1.45	Soft sedimentary	Sparse

Note:- Sediment is given in terms of the volume deposited in the reservoir.

3. SEDIMENTATION

The steep terrain and the dry summer months followed by flood conditions during winter, result to much soil erosion and high sediment transport into the reservoirs. Added to this, in many cases sparse vegetation cover, soft sedimentary rocks and man's activity contribute to the silting up of our reservoirs.

Work on sediment sampling in the various rivers under various conditions of flow has been carried out on a regular basis during the last five years. However, until now it has not been possible to have reliable sediment-flow rating curves for computing the sediment load under various flow conditions. The difficulty is that our rivers have very great fluctuations in their flow, being dry in summer, with floods following the dry months containing high suspended load, while higher discharge in winter may be almost sediment-free. It will therefore require considerably more work in this field, before a reliable evaluation of sediment load can be made using this method.

In order to obtain more reliable estimates, especially in connection with the silting up of reservoirs, a survey has been undertaken of some of the existing dams to determine the volume of the sediment accumulated over a period of years. The results are given in Table 6.

A study of the table exposes some interesting conclusions about the annual sediment per unit area of catchment under different slope, geologic and vegetative characteristics, the percentage of sediment to runoff and the percentage of annual silting up of the reservoirs.

The data show very definitely the striking relationship between sediment and catchment slope which appears to be the most important factor contributing to the sediment load.

Based on slope considerations and average geologic and vegetative conditions four main groups of catchments may be identified.

Very steep slopes > 8% Sediment load 400—600 m³/km² of catchment

Medium steep slopes 4—8% Sediment load 250—400 m³/km² of catchment

Medium slopes 2.5—4% Sediment load 150—250 m³/km² of catchment

Flat slopes 1—2.5% Sediment load 50—150 m³/km² of catchment

By comparing groups of equal slope, the contribution of geology and vegetation can be observed, but not enought data exist to make any factual conclusions. An indication of the contribution of vegetation can be seen by comparing the sediment in the Pomos Dam with that of Argaka Dam of equal slope and of the same geology.

Old waste dumps at Amiandos Mines showing timber staking and tree planting for the prevention of erosion

Mountain terracing for the prevention of erosion and planting of almond trees and vines.

Because of the high sediment load in comparison with dam capacities in Cyprus, which as we can see from table 6 reaches up to 6.4% per year in the case of Morphou dam, important provisions in the design have to be applied. Some of those that have been applied are:

Off-channel reservoirs where that is possible as in the case of Prodhromos reservoir.

Upstream silt traps which are accessible and can be easily desilted as in the case of Agros dam.

Upstream terracing and reforestation. This has been done in the case of Argaka dam catchment, part of which had been burnt by forest fire.

Desilting gates. These have application only in the case of small dams which are operated seasonally and where the morphology of the damsite and reservoir is of a canyon type. It is not possible to apply desilting if the water is required for multiannual operation. However, in our case most dams in the mountainous regions are small in comparison with the river flows and are usually situated in suitable valleys to enable easy desilting during Autumn months when the reservoirs are emptied and left open until spring time when they have to be closed again to store water for the summer months requirements. Many of these small dams such as Trimiklini, Perapedhi, Pyrgos and Lefka are operating in this way successfully.

In the case of Trimiklini dam a hugo sediment load, in the wet year of 1969 brought down from the erosion of the dumps of the Asbestos Mines situated upstream, nearly filled the reservoir to top level and was successfully desilted through operating the desilting gate.

For our main dams, which are multiannually operated such as the Yermasoyia, Polemidhia, Mavrokolymbos, Argaka and Lefkara, it is not economic to allow wastage of water by desilting operations in Autumn. Also, the topography of the damsites and reservoirs being rather flat, would not favour gate desilting techniques. In these cases which constitute our main dam development programme, the only way is to allow for a sufficient **dead storage** for accommodating the sediment over the life of the project between 40 to 50 years.

It can be seen from Table 6 that quite a significant volume has to be allowed for sediment storage which obviously is one further reason for our high dam costs.

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4. ENGINEERING GEOLOGY

A knowledge of the geology of the damsite, the reservoir and the watershed is of fundamental importance in dam design. The design and engineering work depends to a large extent on the properties, strength, elasticity, permeability, stability, durability, density, volume change and solubility of rocks. Adequate site investigations and laboratory tests have to be carried out to determine these properties. For this purpose suitable field equipment are used such as core, overburden and auger drills as well as the necessary laboratory equipment for carrying out triaxial, shear, consolidation, permeability, compaction and other tests as may be required.

The properties of rocks depend upon their mineral composition, texture and structure. Mineral composition governs the hardness, density and solubility. Texture and structure control the properties of rocks as a whole such as strength, permeability and durability. Rocks, as a whole, are classifield in three categories. **Igneous rocks** which have their origin in solidified molten material. **Sedimentary rocks** which are the result of deposition and consolidation of weathered and eroded materials of the earth's surface. **Metamorphic rocks** which were formed through recrystallization in a solid state of igneous or sedimentary rocks having been subjected to high temperatures, pressures and stresses within the crust of the earth.

The geology of Cyprus is quite variable and includes almost all types of rocks. Fig. 7 shows on outline of the general geological features.

About 25% of the Island is formed of igneous rocks around the Troodos Massif consisting mainly of Lavas, Diabase, Gabbros and Serpentines. Cyprus is covered by one of the largest recorded positive gravity anomalies, reaching a maximum of over 250 milligals. The axis of the anomaly lies over the Troodos Massif.

The remainder of the Island is a rich variety of sedimentary rocks. Metamorphic rocks such as marble and schists are found in small quantities mainly in the Kyrenia hills.

Igneous rocks in Cyprus normally form good foundations for dams except where structural weaknesses appear such as faults, shear zones, slides and surface decompositions. Also on igneous rocks we often find buried river valleys and terraces. Problems of this nature have been encountered in many of our dams such as at Lefka and Kalopanayiotis.

Sedimentary rocks, of-course, present many problems such as settlement, seepage, bearing capacity, landsliding, erosion, washing away of chemicals from the soils and consequent contamination of the reservoir water. All the above problems have been faced in Cyprus in our dams. Serious landslides have occurred in the Mavrokolymbos Dam reservoir, significant leakages from Polemidhia Dam reservoir and heavy contamination of the water at Ovgos Dam reservoir.

A description of the main rocks of Cyprus and their engineering properties in connection with dam building follows:-

a. The Mamonia Complex

The Mamonia Complex consists of an heterogenous melange of rocks exposed over wide areas occurring mainly in the Paphos District. Rocks present include red and purple marls, radiolarian cherts, umberiferous shales and bentonitic clays, micaceous, carbonaceous and calcareous sandstones, quartzitic sandstones, massive crystalline reef limestones, tuffs, pillow lavas, serpentines and basic igneous rocks. The sandstone member consists of well-bedded grey sandstones interbedded with organic material and in places with shale and chert bands. Most of the Mamonia Complex, however, is occupied by shales whose depth has not been established yet. The serpentine in the area is highly sheared and fractured.

The various rocks in the Formation never contain water with the exception of the blocks of quartzitic sandstone which are permeable. The high erosion rate would cause silting problems. A very serious problem in these kinds of rocks is landslides on a large scale as have been experienced in the Mavrokolymbos Dam reservoir. The serpentine member may present seepage problems.

b. Igneous Rocks

i. Serpentinites

The serpentinites are considered to have been diapyrically emplaced between the igneous rocks after the volcanic episodes, as strongly suggested by field evidence. The host rocks are very rarely thermally metamorphosed thus suggesting a low temperature of intrusion. This leads to the idea that emplacement must have occurred by solid intrusion of a relatively cold mass. Based on petrological grounds the serpentinites may be divided into three types.

The Bastite-serpentinite, the shattered (or crushed) serpentinite, and the silicified serpentinites.

The bastite serpentinite and the crushed serpentinite have a similar mineralogical composition and are the main types of serpentinite of the Troodos massif. The bastite serpentinite is strongly jointed.

The crushed serpentinite is greyish-green in colour and appears to have suffered much shearing and is completely smashed.

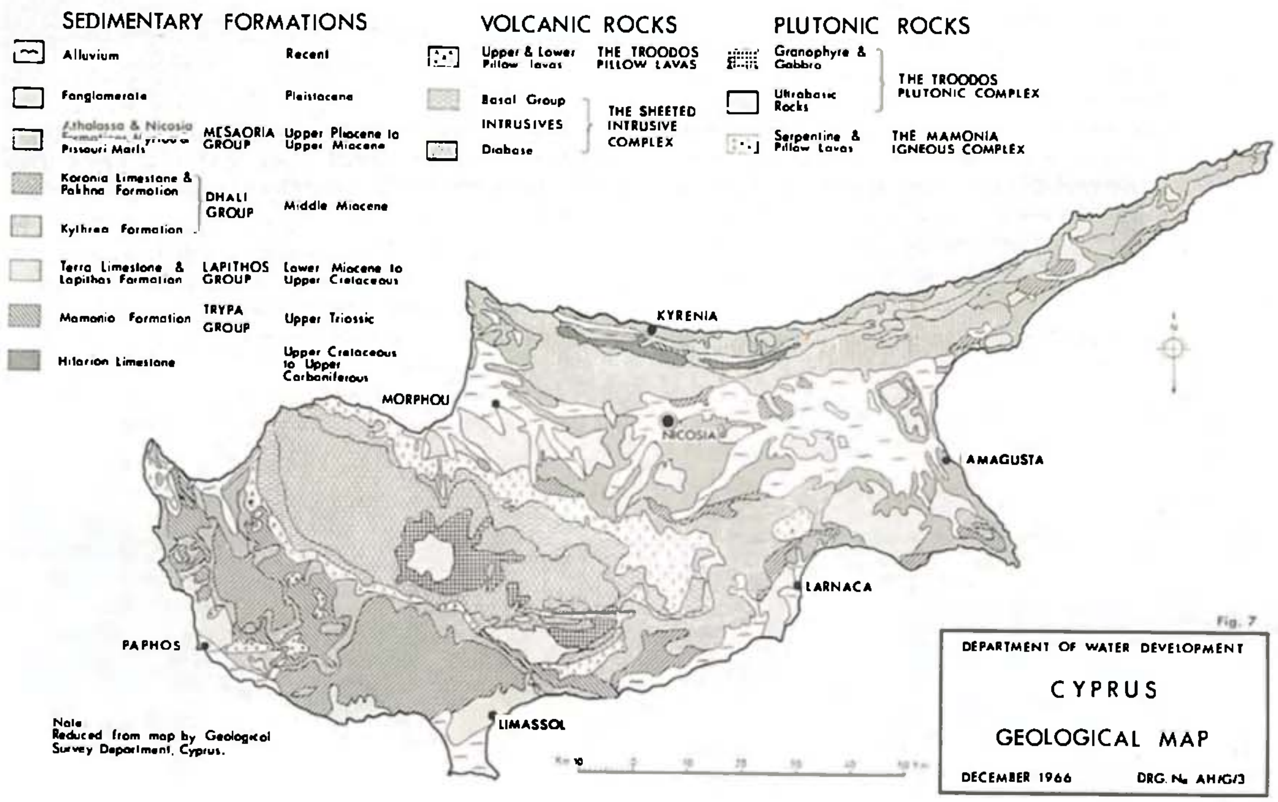
The silicified serpentinite is synonymous to the "Mamonia serpentine". This kind of serpentinite has suffered strong brecciation and silicification.

The serpentinites are characterized by leakages. The rock itself is quite impermeable, but it possesses a fracture permeability and is evidenced by a number of springs. It also becomes soft under saturation and therefore unstable as a foundation rock becoming also liable to slides.

ii. Gabbros

This is a group of coarsely crystalline rocks which ranges from Dunite to Peridotite to Gabbro.

These rocks are characterized in many places by differential weathering. There is considerable intermingling of zones of very friable rock with bands of relatively fresh rock.



Furthermore, these rocks have suffered tectonic disturbances which are crossed by extensive crushed zones. Weathering and erosion are particularly active in these zones of crushed rock resulting in the formation of scree.

Water may seep and percolate through the zones of structural weakness resulting in the formation of springs. The shattered parts of the rocks are usually unstable.

Because of the limited catchment and steep topography, it is not economic to build large reservoirs on this geological formation. In any case, the leakage problems as proved in the case of the Agros dam are excessive and such a formation is not therefore attractive for dam building.

iii. Diabase

The Diabase consists of a closely packed series of multiple dykes and dyke swarms separated by narrow screens of varying types of lava as a host rock.

The Diabase is characterized, in the field, by rugged outcrops resistant to erosion and weathering and of a greyish to light brown colouration. The Diabase has been strongly affected by earth movement, interweaving faulting being very common. The dykes are generally discontinuous with variable strike and steep dips. The general trend of the dykes is North-South, but local departures from this trend are met with, these having been brought about either by faulting or by variations in the tensional forces during the intrusion.

Almost all dykes in the diabase have been altered to a greater or lesser extent by saussuritisation and concomitant uralitisation. The end result of these alterations is the filling of joints and cracks in the diabase. This increases the rock's watertightness and consolidation.

Slight movements or sliding may occur along fault planes parallel to the strike of the dykes.

In general we may say that the Diabase is water tight and stable forming one of the best rock foundations for dam building.

iv. Pillow Lavas

The Pillow Lavas are divided into two parts: the Lower Pillow Lavas and the Upper Pillow Lavas.

The Lower Pillow Lavas are composed of well formed ellipsoidal pillows and a considerable number of dykes and sills. The pillows range in size from a few inches to about four feet in length. The sills usually have a well developed columnar jointing.

The Upper Pillow Lavas are composed of well-formed pillow lavas with only minor amounts of intrusives. The length of the pillow ranges from a few inches to about eight feet. The pillows are in many places fractured or jointed. The spaces between the pillows are filled with calcareous material.

The Pillow Lavas are usually impermeable except in areas of tectonic disturbance. There are many extensive and minor faults in the Pillow Lavas causing considerable fracturing and brecciation. Water usually leaks through these zones of weakness to form springs. In spite of the tectonic disturbances, the Pillow Lava rocks are very stable.

Topographically the Pillow Lavas are characterized by smooth rounded low hills. Weathering is usually more active on the pillow rather than on the sills or dykes.

In general pillow lavas are considered to be amongst the best rocks for dam building.

c. Sedimentary Rocks

i. Hilarion Limestone

The Hilarion Limestone forms the backbone of the Kyrenia Range. The outcrop is 90 km long and 4 km wide and is in a form of an arc concave to the North. The upthrust of the Hilarion Limestone to its present position took place during the Alpine orogeny. It is a hard massive crystalline limestone varying in colour from black to dark grey and white and has been considerably faulted and fractured. There are two main trends of faulting, one East-West and one North-South. The North-South faults have caused considerable fracturing and brecciation of the limestone and are now occupied by streams which have eroded deep into the fault zones leaving steep slopes on both sides.

The almost vertical faults and fissures favour a high infiltration. Not only the high infiltration from solution channels, but also due to the steep topography and the very small streams formed, no dams are envisaged to be built on this limestone.

ii. Lapithos Chalks

The rocks of this Group occur as outcrop peripheral to the Troodos Massif and the Hilarion Limestone mass and their age ranges from Upper Cretaceous to Lower Miocene. The rocks of this Group can be sub-divided into three units:-

The lower part consisting of marl being the thinnest unit. The middle part consisting of chalk and chert which is the most widespread and thickest unit. The upper part which consists of bedded chalks and marls. Generally the rocks of this Group to the South of the Troodos Range dip to the South and those to the North of the Range dip to the North. In many places the rocks have been disturbed by faulting and folding. The permeability of these rocks when undisturbed is negligible. In zones of fracturing, however, the permeability and percolation can be very high. The chalk is considerably soluble and fracture zones in some cases develop into underground solution channels. Due to the solubility of the chalk the water extracted in many cases is fairly hard. Such solution channels could cause serious problems of seepage. Fractured chalk if exposed in stream-bed can be effectively recharged.

Obviously there are many problems with building dams on the Lapithos Chalks and field investigations and Laboratory tests carried out on certain prospective damsites proved serious seepage problems. However, it is envisaged that a limited number of dams on this formation will be built in particular where marly chalks exist where with proper grouting, any seepage losses may be reduced to the minimum.

iii. Kythrea Formation

The Kythrea Formation occupies a belt along the Southern part of the Kyrenia Range with a maximum

Pakhna Formation (white brownish chalks) overlying **Lapithos Chalks** (lighter colour, left centre). Proposed damsite at Asprokremmos, Xeropotamos River, Paphos District.

Athalassa Formation, alternating horizontal beds of calcareous sandstone and sandy marls. Leontari hill, Athalassa Farm near Nicosia.

Mamonia Complex, igneous and sedimentary rocks overlain at top by Lapithos Chalks. Lower part of photo shows reworked sedimentary rocks of the Complex with obvious landslides. Mavrokolymbos reservoir area, Paphos District.

The Kyrenia Range viewed from Kythrea village showing the **Hilarion Limestone** flanked by younger sediments **Lapithos Chalks** (whitish colour) and **Kythrea Formation** (brownish colour).

The Troodos Sheeted Intrusive Complex (Diabase) showing multiple swarms of dykes. Skouriotissa—Kalopanayiotis road.

Upper **Pillow Lavas** of the Troodos Igneous Complex. On the Nicosia—Limassol main road (21 m.p.).

width of 13 km and a much narrower belt along the Northern flank of the range. It consists of flysch type Middle Miocene sediments which are mainly made of alternating beds of marls, sandstones and thin beds of limestones. All these beds have been folded and thrusted and gave rise to a very characteristic topography. The rocks of this Formation are impermeable because of their high clay content. The coarser facies sometimes yield water which, however, is highly mineralized and saline due to connate salts. In many areas surface water is also mineralized from a similar source. Most fracture and fault zones are filled with clay material and the possibilities of leakage are very little. Erosion and silting, however, is serious in areas of mainly clayey outcrops.

From the impermeability point of view therefore, these rocks are very good for damsites and the fact that in many cases the topography is also ideal giving good storage and good damsites, makes possible the building of some of the cheapest dams in Cyprus. A number of such dams have already been built and a number of other potential damsites exist. However the greatest problem is that of the water quality coming from these formations and its deterioration under storage conditions.

iv. Pakhna Formation

Rocks of this Formation which are of Middle to Upper Miocene age overly the rocks of the Lapithos Formation and outcrop mainly in the area peripheral to the Troodos Range, especially to the South of it. In contrast to the white Lapithos chalks, the rocks of the Pakhna Formation maintain a cream or light brownish colour. Rocks in this Formation include bedded chalks, marls, sandy and chalky limestones. The upper part consists of shales, grits, gypsum and reef limestone. Structurally these rocks have undergone less distrurbance than the rocks of the Lapithos Formation. Still a number of major faults and many smaller ones have caused considerable fracturing giving rise to significant seepage paths. The sandy limestone, gypsum and reef limestone are considerably porous, and when below water table they form rich, productive aquifers. The gypsum however, causes mineralization and consequently yields water unsuitable for human consumption, rich in sulphates, chlorides and hardness. The location of this Formation and its more or less gentle topography gives us many of the most important damsites. Already the largest dams of Cyprus have been built on this Formation and a number of other larger ones have still to be built. Leakage problems have been encountered both through the Formation itself and deep alluvial river beds, but in both cases extensive grouting works proved satisfactory. In the case of the alluvial river bed at Yermasoyia, chemical grouting was undertaken successfully.

v. Nicosia — Athalassa Formation

Rocks of the Athalassa and Nicosia Formation are exposed along the Northern coastal plain of the Island, and in the Mesaoria Plain, resting unconformably on the Kythrea Formation. The rocks of the two Formations are similar in lithology. They consist of yellow sandy limestones, calcareous sandstones and marls. The Athalassa Formation of Upper Pliocene age overlies the Nicosia Formation of Lower Pliocene age. Both Formations have not suffered serious tectonic disturbance and they are usually horizontal. The sandy limestones and sandstones are very porous and permeable and in many areas yield satisfactory quantities of good quality water. The sandstones form almost vertical escarpments.

Many small dams have been built on the lower marly horizon of this Formation, without any serious problems. Small leakages through sandstone joints have however occurred.

vi. Fanglomerate Series

These are conglomerates found along the foot of the Troodos and Kyrenia mountains and South of Famagusta consisting of boulders, angular and rounded gravels, calcareous sands and silts and were formed during Pleistocene times. The Fanglomerates are permeable and yield large quantities of water when below water table. Such a Formation could present seepage problems as well as landslides when not consolidated and when it is disturbed.

Many small dams have been built on this Formation both in Famagusta and Kyrenia mainly for groundwater recharge purposes either directly through the permeable reservoir bed or as check dams regulating recharge water downstream.

vii. Alluvium

The large thichnesses of alluvium and the formation of river terraces took place at a time when the two mountain ranges of the Island suffered extensive erosion. Boulders and gravels infilled river valleys to varying depths. Further down the slopes of the mountains the rivers had to cut through the sedimentary rocks and have deposited big quantities of transported material. The Morphou plain is the most important alluvial region composed entirely of gravels and boulders largely recharged through the Serakhis river, forming the largest aquifer of the Island.

The alluvial valleys are usually wide with gentle slopes thus giving very good storage possibilities. In most cases, the depth of the alluvial river beds composed of gravels and sands which sometimes exceed 100 m, is such, that it makes it impossible to build water tight dams. However, groundwater recharge dams provided with upstream aprons have been built, on this Formation.

5. EARTHQUAKE PROBLEMS

Cyprus is covered by one of the largest recorded positive gravity anomalies over 100 milligals, and exceeding +200 milligals on the Northwest parts of Troodos, which is attributed to an extensive slab of high density rock at least 11 km thick underlying Cyprus at shallow depth. This slab is considered to have been once part of the upper mantle underlying the sea between the Continents of Africa and Eurasia. With the continental shields approaching each other during the Apline orogeny, this slab of mantle was underthrusted by the edge of the African Shield and thereby raised to its present level in the upper part of the crust.

It is believed that the Troodos summit has been elevated by something like 3,000 m since Cretaceous times whilst the rest of the Island has risen some 700m. This differential uplift which is still in progress is considered to be the result of isostatic adjustment initiated by the underthrusting of the Eurasian hinteland bv the African foreland.

The zone of underthrusting seems to be related to the fault line in the Southwestern parts of the Island and the graben of the Mesaoria plain in the East, both of which are the seats of very active focus of energy release and earthquakes.

As a result of the differential uplift still prevailing in the region and the concentration of the earthquake foci in the zone of underthrusting, it is considered that the seismic activity in Cyprus is a result of the isostatic adjustment initiated by the above mentioned under thrusting.

The earthquake history of Cyprus does not in fact mention earthquakes of very high intensity as are known to have occurred in neighbouring countries such as Greece and Turkey. The maximum earthquake intensities in Cyprus since the last 2,000 years are estimated to be of X degree on the Mercalli — Sieberg scale and to have occurred only twice, in 76 AD and 342 AD when many of the cities were ruined. In all other cases the highest intensities reached were near or below the IX degree. Seventeen important earthquakes have been recorded during the last 2,000 years which were of intensities ranging between VII—X degree. The last important earthquake, of IX degree occurred in 1953 when a number of villages in the Paphos district were almost totally destroyed.

An assessment of earthquake effects, has to be made in dam design, even if such effects cannot be accurately determined. Because of the uncertainty in evaluating earthquakes, conservative criteria have to be used. The dam should be designed to resist the inertia effects caused by the sudden earthquake movements.

In the case of concrete gravity dams, to counteract the earthquake force, the dam is designed to resist an additional horizontal force acting through it's centre of gravity which is proportional to 10% of gravity acceleration or equal to 10% the weight of the structure.

For earth embankments, the design measures taken to increase safety, consist of a more conservative approach to the design of the main dam elements such as flatter slopes of embankment, thicker cores, better filter and higher capacity drains downstream. Such aspects cannot be analytically determined and the designer should rely heavily on his personal experience and on the knowledge and history of other dams which have been subjected to earthquake forces in the past.

In Cyprus we have not as yet experienced any damages on dams from earthquakes, but this is due to the fact that our dams are of very recent times.

The 1953 major earthquake occurred in Paphos where dams did not exist at the time.

6. ECONOMIC CRITERIA

The economic feasibility of a project depends on the long term costs and benefits expected. In Cyprus every project, whether for irrigation, domestic or other purposes of water utilization, has to be proved viable in order that its construction may be authorized. The viability of a project is based on the calculation of the benefit:cost ratio which normally should exceed one, or more properly, as is done nowadays in most cases, the internal rate of return should be calculated which should show an interest of more than 7%. For international financing, the internal rate or return should normally be at least 12%.

Usually the investment costs are amortized at an interest rate of 7% over a life of 50 years, except in the case of mechanical equipment for which replacement at shorter intervals is provided.

Dead storage for sedimention has to be provided for the 50 years over and above the useful storage. Maintenance and operation costs are also added being about 2% of the annual investment costs.

In Cyprus dam building is very costly per unit volume of water produced and as shown in Table 7, the average rate is 17.5 mils/m^3 of water at the dam for irrigation and 28 mils/m^3 including the distribution to the farmers. In the case of Lefkara dam for domestic water supply the cost is 20 mils/m^3 at the dam, and 45 mils/m^3 including a long conveyance system and treatment works.

In the case of the groundwater recharge dam at Masari the cost of water is 17 mils/m^3 at the dam and 27 mils/m^3 including the spreading grounds.

The reasons for the high costs are:

i. Steep topography of the Island which requires relatively high dams to store the required quantities of water.

ii. Unfavourable geology of the sites and reservoirs requiring a lot of treatment.

iii. High erosion in the watersheds requiring the provision of considerable dead storage to maintain the useful storage.

iv. Irregular river flow followed by many dry years, thereby requiring big storage in order to provide the required dependable supply.

v. Evaporation from the reservoirs in many parts of the Island is rather high.

UNIT COST OF MAIN GOVERNMENT DAMS

TABLE 7

Ser. No.	Project	Costs £ Dam only: Capital Cost £ 1,000	Dam only: Annual costs 50 years amort. 7% interest incl. O & M £ 1,000	Including Irrigation System: Capital Cost £ 1,000	Including Irrigation System: Annual costs 50 years amort. 7% interest incl. O & M £ 1,000	Water Supplied annually Million m³	Cost per m³ mils: Dam only	Cost per m³ mils: Including Irrigation System	Remarks
1.	Kiti	140	12	220	19	1.0	12	19	Irrigation Schemes
2.	Pomos	242	20	326	27	1.1	18	25	
3.	Ayia Marina	100	9	128	11	0.2	45	55	
4.	Argaka	270	22	398	35	1.4	16	25	
5.	Kalopanayiotis	236	19	300	28	0.5	38	56	
6.	Polemidhia	809	69	968	84	2.5	28	33	
7.	Yermasoyia	930	80	1 900	165	6.2	13	27	
8.	Mavrokolymbos	335	28	490	42	2.0	14	25	
	Weighted Average						17.5	28	
1.	Masari	250	21	370*	32	1.2	17	27	* Groundwater Recharge Scheme
1.	Lefkara	1,266	110	3,128**	250	5.5	20	45	** Domestic Water Sypply Scheme

O & M = Operation & Maintenance.

vi. Conveyance of water to long distant areas of demand is necessary in many cases.

vii. Water for downstream and upstream water rights has to be allowed for, which reduces the net flow available in the reservoir.

viii. Steep land in many parts, in the case of irrigation schemes, require costly land levelling.

ix. Badly divided land tenure in the case of irrigation schemes, is another adverse economic factor.

x. Labour costs is yet another problem recently added, as we now have reached a state of full labour employment.

In water projects, whether irrigation or domestic water supply, high efficiency systems are introduced aiming at maximizing benefits at minimum water consumption. Irrigation systems are in most cases pressure piped supply enabling the utilization of sprinklers, hose-basin or other efficient systems. In the case of irrigation works, high return crops such as citrus are planted which have good marketing prospects. The land is levelled and proper agricultural methods applied. In the case of domestic water supplies these are always piped house-to-house connections with a water meter installed whilst an increased scale of charges based on the consumption is enforced in order to make wastage prohibitive.

Because of the extensive information that economic studies nowadays are expected to give, scholastic studies have to be made involving a great number of inputs and alternative project studies. The only way that these projects can be analyzed is by the use of mathematical models which can easily be processed by a computer. Economic models used in Cyprus are:-

i. A cash flow model, utilizing inputs of investment phasing, operation and maintenance costs analyzed over 50 years as well as the water supply available and the income derived to be balanced over the 50 years analysis at the appropriate unit cost of water.

ii. For a complete project economic evaluation, in order to determine the internal rate of return, a more complex model is used which includes inputs of investment costs, both for the dam, conveyance, distribution and other project costs, operation and maintenance, production yields and prices, and phasing of the project implementation. This model allows a detailed analysis of a water development project based on many alternative criteria, variable development phasing and allowing for a sensitivity check of critical inputs.

CHAPTER II

MASONRY DAMS

INTRODUCTION

In Cyprus stone had been, until the end of the 1950's, the main building material for all types of buildings. From ancient times up to the present time sandstones, limestones, igneous rocks and sometimes marble have been extensively used for building works.

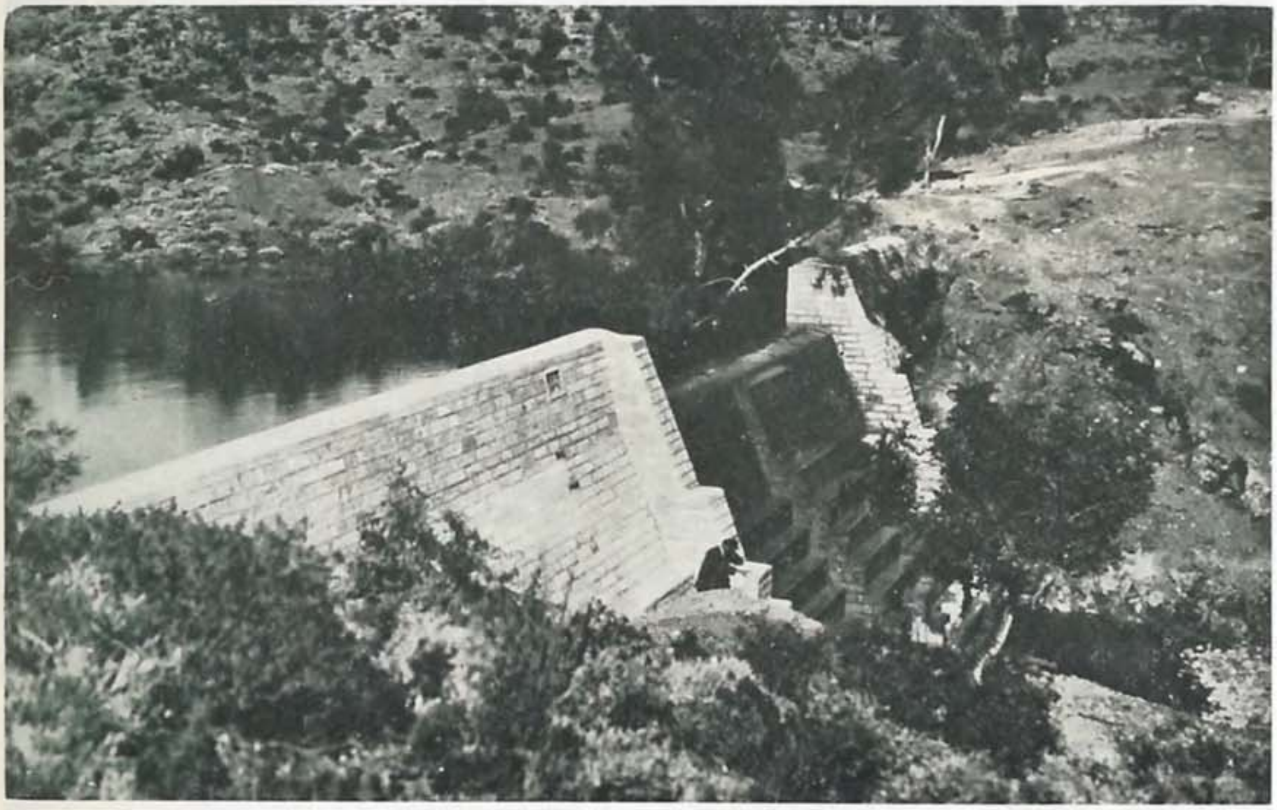
In ancient buildings such as Greek temples (about 400 B.C.) marble was also used but due to the fact that marble is rarely found in Cyprus and had to be imported, a lot of the temples were built chiefly in stone.

Parts of these buildings are still well preserved, thus giving a fine example of the durability of stone.

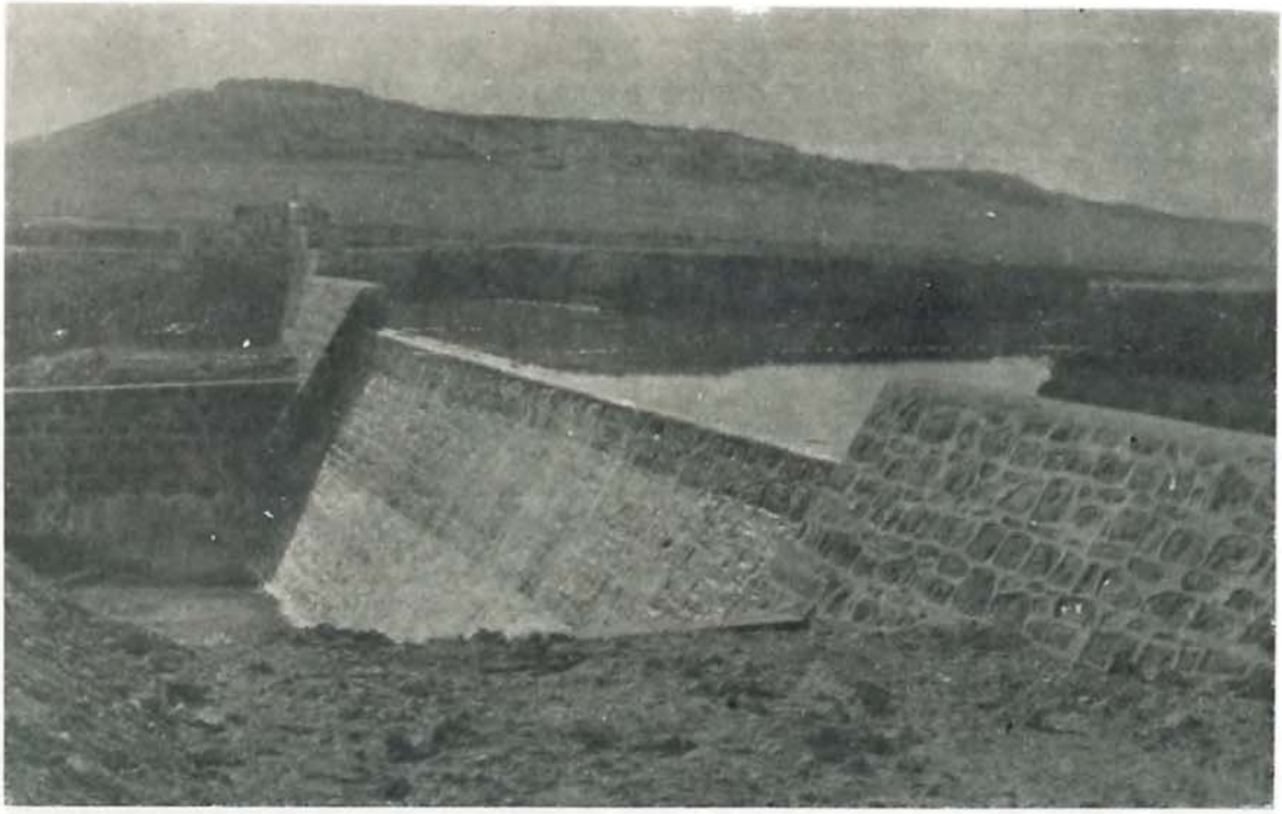
Roman aqueducts (about 300 A.D.) were built in stone and quite a few of them are still preserved in good condition. But the most striking examples of masonry in Cyprus are the medieval buildings (1100—1600 A.D.). The most imposing of them are the city walls of Famagusta and Nicosia, the castles of Kyrenia, Kantara and St. Hilarion, and the churches of St. Sophia Nicosia (now a mosque) and St. Nicholas Famagusta.

In spite of the development of concrete and steel in modern times (about 1900 A.D.), the Architect and Engineer have until recently used predominantly stone as the main building material. Sometimes when it was found that it was more suitable to use concrete, this would be faced with nicely dressed masonry.

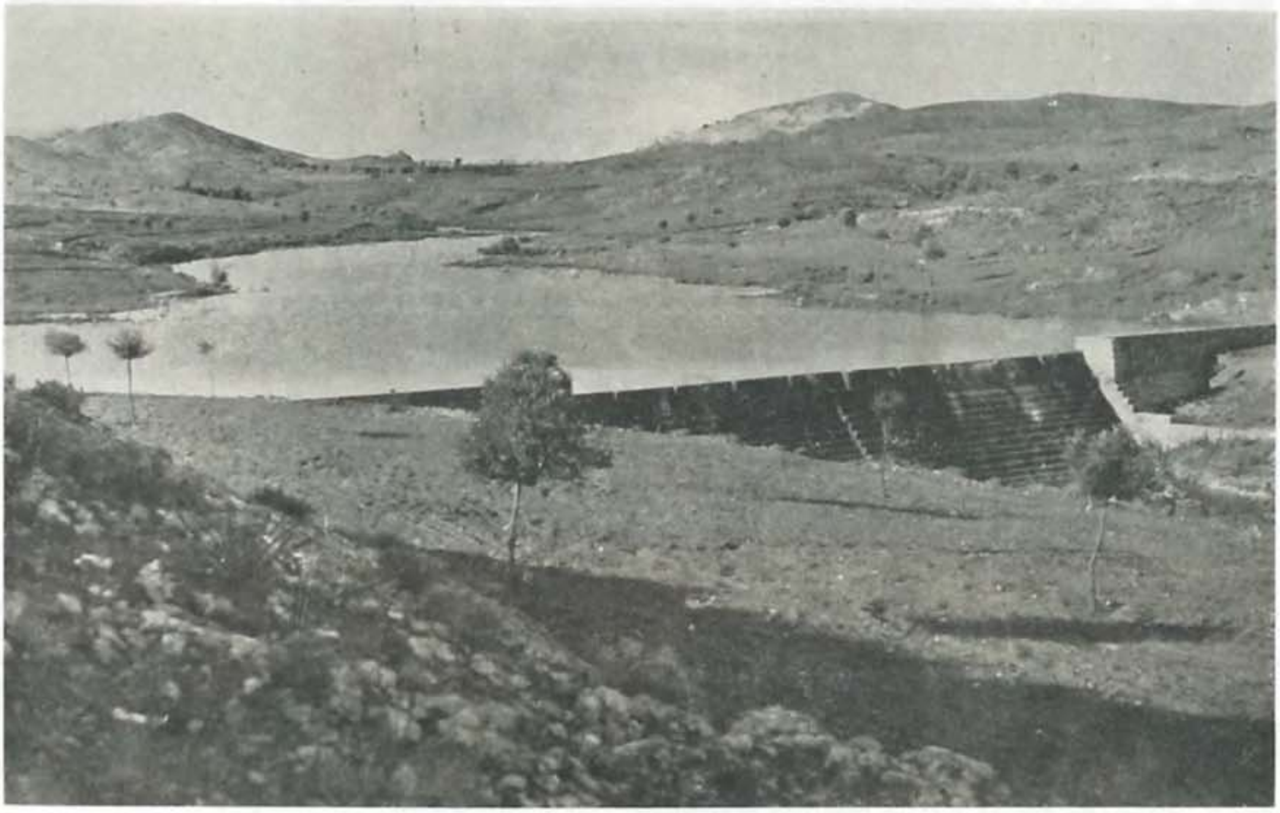
Because of the stone building tradition prevailing, the first diversion weirs, channels, dams, reservoirs and other water supply and irrigation structures were built in masonry. A number of small dams mainly for diverting water have been built starting with the decade of 1940. The Kophinou, Lymbia, Akrounda and Geuneyli small dams are examples shown in the accompanying photographs. The nine largest masonry dams which are of some importance will be described in this chapter. The eight of them are gravity dams, and one is a buttress dam. The properties of the stone used and the main design characteristics for each dam



The Kophinou masonry dam



The Geunyeli masonry dam.



The Lymbia masonry dam.

are given in Table 8.

A lot of work has been done in rubble using stones which are not dressed or are roughly dressed having wide joints, whilst better work was done in ashlar with stones carefully dressed and pointed with narrow joints. On the highlands where rock is in abundance the villagers used to build their irrigation reservoirs, channels and other building works in rubble which they obtained from the site or nearby quarries. Also in the lowlands, where quarries are plenty, reservoirs and tanks have been built in sandstone.

Building in sandstone is usually more expensive than building with igneous rocks which are mostly used

The Old Bekir Pasha aqueduct for the Domestic Water Supply of Larnaca.

in rubble work whilst sandstones are used for ashlar work. Also the purchase and transport of sandstones costs usually more than igneous rock, which is obtained locally whilst sandstone is usually bought from special quarries.

The general use of masonry in Cyprus necessitated the development of good artisans in building, as masonry requires much more skilled artisans than the use of concrete does.

In choosing a building material the Engineer has to consider the suitability of the material for the required use, which depends on some or on all of the following properties:

Crushing strength, porosity, durability, resistance to frost action, efflorescence, hardness, weight, workability, appearance and cost.

A comparison between concrete and the various types of stones used in Cyprus follows.

i. Crushing strength

Crushing strength is not the chief factor in choosing a stone for the building of a dam. In a water retaining structure the moment of resistance can be increased by increasing the thickness of the wall and is independent of the crushing strength. But there are cases where the walls are required to bear load and in such cases the crushing strength is of great importance.

Values of the crushing strength of sandstones, limestones, igneous rocks and concrete are as follows:

Crushing strength of	sandstones	up to	140	kg/cm^2
" "	limestones	"	350	"
" "	igneous rocks	"	2000	"
" "	concrete 1:2:4	"	350	"
" "	concrete 1:8	"	70	"

ii. Porosity

Porosity is the percentage of air voids in the material. This is very important in the contruction of dams and waterworks in general, because it governs the permeability or absorption properties of the material. The more air voids in the material, the more the permeability and the worse the material for use in waterworks. The water absorption percent being an indication of the porosity is as follows:

Sandstones porosity 10—30%
Limestone porosity 5—30%
Igneous Rock porosity 0.5—10%
Concrete porosity 1—30% varying considerably and depending on the water cement ratio. The less the water cement ratio, the denser the concrete and therefore the less permeable. But for the low water cement ratio the concrete is difficult to work and there is also danger of seggregation.

iii. Durability

As previously stated, masonry buildings in Cyprus exist since at least 4000 years and their durability is very good. Retaining walls in masonry are better preserved when plastered inside, and more so if before plastering an impermeable coating is applied. Concrete is made more durable when using special types of cement like pozzolanic, sulphate resisting, supersulphate and high alumina cements.

iv. Resistance to frost action

In this case also the more porous or laminated a material is, the worse it suffers from frost. This happens due to the absorption of water which expands in the pore spaces and bedding planes when it becomes frozen and thus disintegration of the stone accurs. In Cyprus this occurs only on the mountains where, most of the work is built with igneous rock which is quite impermeable.

As for concrete, frost can damage it, both before and after setting and therefore special precautions have to be taken when concreting in very cold weather.

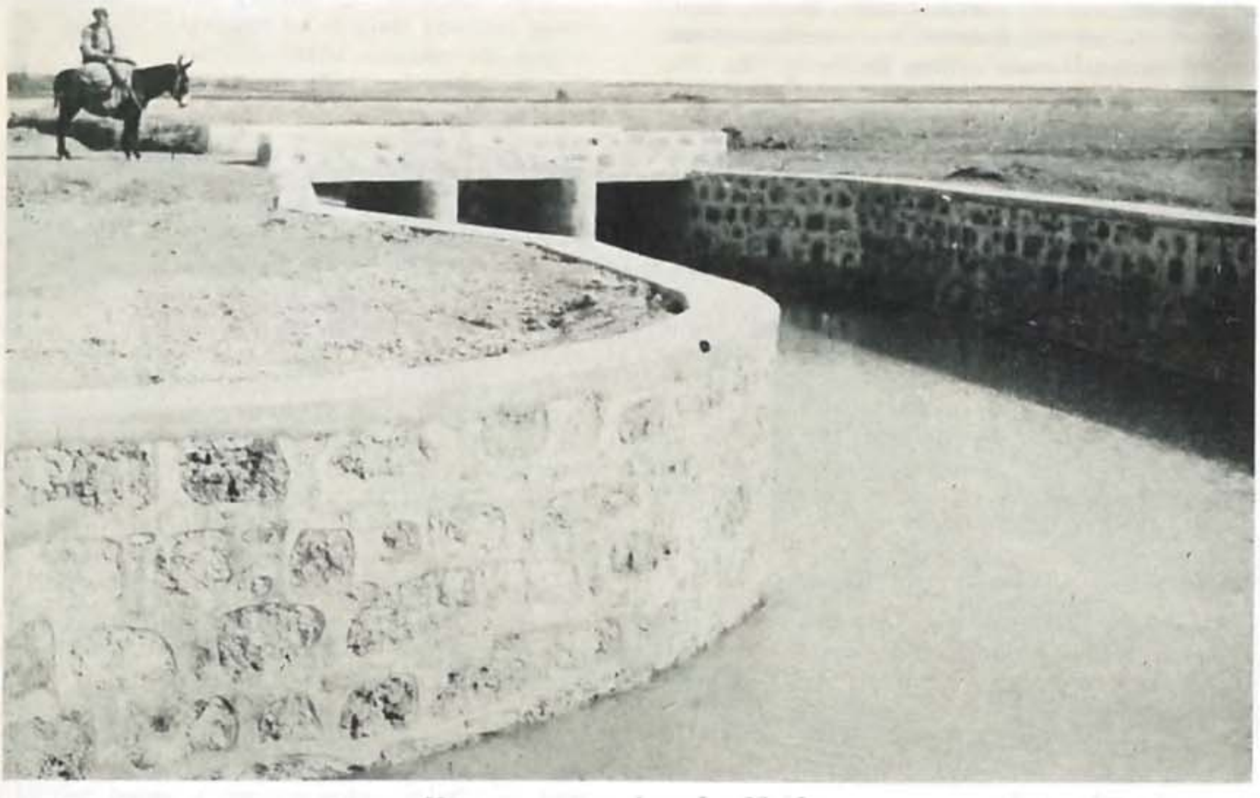
TABLE 8

MASONRY — CONCRETE DAMS — DESIGN DATA

Dam	Type of Dam	Type of rock foundation	Building Material			Resistance to sliding Tan. Θ		Uplift consideration	Downstream slope	Stresses on Fountation Base							
										Reservoir Empty				Reservoir Full			
										With earthquake —0.1g		earthquake		With earthquake +0.1g		Without earthquake	
			Type Description	Weight Kg/cm³	Crushing strength Kg/cm²	With earthquake 0.1 g	Without earthquake			U/S	D/S	U/S	D/S	U/S	D/S	U/S	D/S
Lythrodhondas (D/S)	Masonry Buttress	Pillow Lava	Medium grained vesicular lava	2480	460	1.12	1.00	Half Full Reservoir Head For Upstream Zero Reservoir Head For Downstream	0.75/1	— 3%	+	+	+	+	+	+	+
Lythrodhondas (U/S)	Masonry Gravity	Pillow Lava	Highly vesicular lava	2480—2550	560— 630	0.77	0.64		0.67/1	— 7%	+	+	+	+	+	+	+
Petra (D/S)	”	Pillow Lava	Medium grained vesicular lava	2300	270— 500	0.63	0.51		0.56/1	+	+	+	+	+	+	+	+
Petra (U/S)	”	Pillow Lava	”	2480—2600	”	0.71	0.58		0.67/1	— 4%	+	+	+	+	+	+	+
Galini	”	Diabase	Fine to medium grained diabase	2730	300—1900	0.68	0.56		0. 5/1	+	+	+	+	+	+	+	+
Kafizes	”	Diabase	Fine to medium grained diabase	2620—2790	1600—1840	0.68	0.56		0.67/1	+	+	+	+	+	+	+	+
Kalokhorio	”	Pillow Lava	Fine to medium grained vesicular	2500—2850	400— 600	0.93	0.80		0.42/1	—10%	+	—2%	+	—28%	+	—24%	+
Kandou	”	Limestone	Fine to medium grained calvareous sandstone	1930	120— 380	0.81	0.68		0.70/1	— 3%	+	+	+	+	+	+	+
Perapedhi	”	Diabase	”	2770	890	0.78	0.66		0.75/1	— 5%	+	+	+	+	+	+	+
Trimiklini	Concrete Gravity	Diabase	Concrete	2380—2430	200— 400	0.68	0.56		0.61/1	— 4%	+	+	+	+	+	+	+
Pyrgos	”	Diabase	”	”	200— 400	0.70	0.57		0.67/1	— 4%	+	+	+	+	+	+	+
Lefka	”	Basal	”	”	200— 400	0.68	0.55		0.58/1 0.84/1	— 3%	+	+	+	+	+	+	+
Palekhori	”	Diabase	”	”	200— 400	0.78	0.66		0.70/1	+	+	+	+	+	+	+	+

Tensile stress on the base given as % of the compressive stress   U/S = upstream   D/S = downstream

Masonry river diversion weir and intake channel.



Masonry conveyor channel and bridge.

Masonry reservoir and intake channel

v. Resistance to efflorescence

The salts in cement and lime mortars may be absorbed by the stone and crystallization of the salts may occur setting up a decay of the stone or causing efflorescence on the surface. Such defects usually occur if the jointing material is a rich impermeable cement mortar, as water will be absorbed by the stone and not by the mortar.

As the water dries out from the stone surface, the salts are either brought to the outer surface of the stone causing efflorescence or they crystallize in the pores immediately behind it. Efflorescence also occurs in the external surface of concrete especially when it is porous, but it is very little in the case of good impermeable concrete. Thus, the more porous a stone or concrete is, the more liable it is to be attacked by efflorescence. To reduce efflorescence as much as possible in masonry the interior of the reservoir walls is either covered with lime mortar or asphalt, the latter being preferable as it is waterproof and prevents the backing mortar penetrating the stone. Because of the influence of salts on efflorescence, stones are more liable to attack in seaside places.

vi. Hardness

Hardness is of importance in ashlar work when the stones require dressing. The hardest a stone, the more difficult is to dress and build it. Also hard stones are more impermeable.

Igneous rocks are usually the hardest. Limestones are in many cases as hard as igneous. Sandstones are usually the softest rocks.

Concrete is comparable with the harder type of rocks.

vii. Weight

The weight of the building material in dams and other retaining walls is of importance because it increases the resistance of the structure.

The following are the weights:

Sandstones	1900—2300 kg/m^3
Limestones	1900—2700 "
Igneous Rocks	2400—2800 "
Concrete 1:2:4	2300—2400 "

viii. Workability

In masonry, if walls are built in ashlar they entail a lot of work which includes dressing of stones. For an average sandstone like that used for the walls of the big reservoirs of Nicosia and Famagusta, a good mason can dress about 30 stones a day (each stone about 50 cm long, 30 cm high and 18 to 30 cm thick. A day's work is 8 hours).

The stones are then carefully built in courses and finally pointed with cement in the joints to give them a nice appearance. Inside, the walls are covered with a layer of asphalt and then plastered (Plaster 1:3).

In building dams, reservoirs and tanks in masonry, care must be taken at corners and other joints because of danger of leakage through weak joints.

A usual joint is made up of an open space of about 4 cm x 8 cm throughout the depth of the wall in the masonry. In this space a copper plate about 6 mm thick or suitable rubber material is placed parallel to the 8 cm side and finally the hole is filled with asphalt.

There are also difficulties with concrete work such as:- Retaining walls in concrete need reinforcement and formwork is required to hold the concrete during the setting period. Also, machinery is required for mixing and transporting concrete. However, concrete work is a more rapid method of construction.

ix. Appearance

The appearance required depends largely on the surroundings and on the architectural character of the environment. Otherwise, both stone and concrete can be made to have a nice appearance. Building in stone gives the structure a more massive, impressive and solid appearance. Concrete looks more light and sometimes weak, unless proper treatment of the outside faces is made.

x. Cost

The cost of a building material depends on the following:

Cost of purchase, transport to the building site, preparing, building and preserving the material.

Usually the material is bought at the site and includes the cost of transport but there are cases where the Contractor buys the stones at the quarries and then uses his own or other lorries for transport.

To give an idea of the cost of masonry building in Cyprus as compared with concrete the following analysis given represents costs in the 1950's at a time when masonry was extensively used in Cyprus.

Masonry Costs/m³
Ashlar building (7.5 stones dressed/m³ of masonry)

	£ mils
Cost of purchase	1.125
Cost for dressing stones	0.225
Rubble filling	0.750

Lime cement mortar quantities/m³ and costs are:

	£ mils
Sea sand 0.35 m³	0.150
Lime 60 kg	0.300
Cement 120 kg	1.200
Building Labour	3.750
Miscellaneous	0.500
Total cost/m³	8.000

Plastering of masonry walls (1:3) quantities and cost/m² are as follows:

Sea sand 0.06 m³	0.025
Cement 27 kg	0.325
Labour	0.550
Miscellaneous	0.200
Total cost/m²	1.100

Total masonry cost therefore is
£8.000 + 1.100 = £9.100/m³

Comparing with cement concrete 1:2:4 quantities and cost/m³ are as follows:

	£ mils
Sea gravel 0.97 m³	1.025
Sea sand 0.50 m³	0.225
Cement 250 kg	3.250
Labour	2.800
Miscellaneous including mixer, elevators, formwork etc.	2.000
Total	9.300/m³

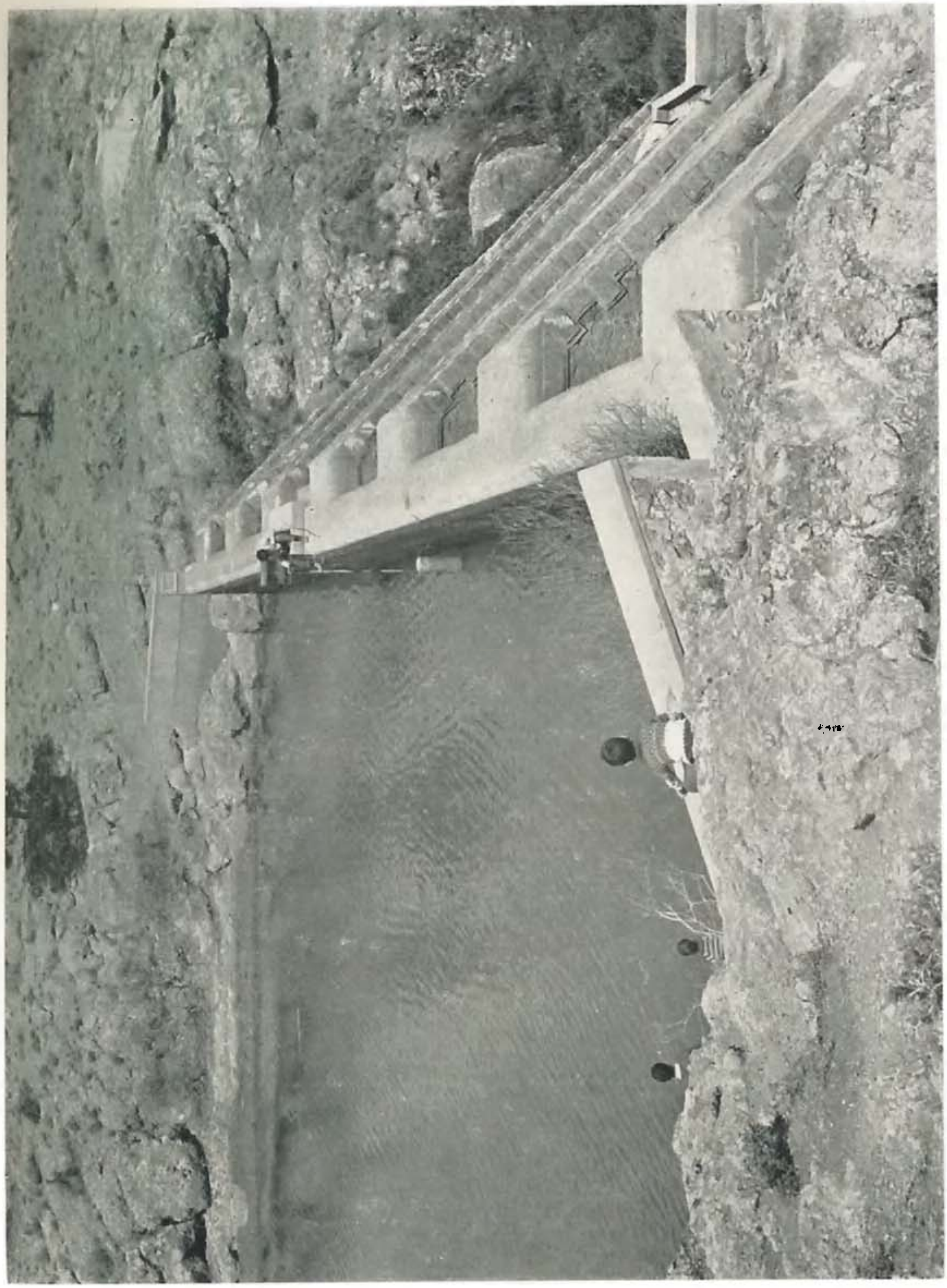
Steel reinforcement used to cost at the time £50 per ton.

Timber formwork used does not of course lose its value completely, but considering depreciation, the cost of formwork was about 150 mils/m² of concrete.

Therefore, the total cost of masonry work in the 1950's as shown above was somewhat cheaper than concrete and this was yet another reason for its preference.

The Akrounda masonry dam.

LYTHRODHONDAS



The Lythrodhondas downstream dam.

YIALIAS - LYTHRODHONDAS DOWNSTREAM AND UPSTREAM PROJECTS

1. PURPOSE

The two dams at Lythrodhondas were built in order to store winter water of the spate flows of the Koutsos tributary of the Yialias river to ensure an adequate supply of water during early spring and summer months for the irrigation of vegetables. First the downstream dam was built in 1944. It had to be raised in 1950 to yield more water and a new dam was built upstream to give even more water to the same system of irrigation.

2. LOCATION

The two dams were built 1.5 km apart, the downstream one being at a distance of 0.5 km from Lythrodhondas village and at an elevation of about 414 m asl, whilst the upstream dam is at a distance of 2 km from the village and at an elevation of about 450 m asl.

3. PLAN

a. Water and Land

For the upstream dam the average annual runoff is about 0.4 million m^3, while for the downstream dam the average flow is about 0.9 million m^3 annually. The land commanded is 230 donums downstream of the village. Much more irrigable land is available but scarcity of water is the limiting factor to irrigation expansion.

b. Geology

i. Downstream Dam

The dam is built on the Lower Pillow Lava series. They appear as iron stained fine grained vesicular rocks cut by darker diabasic intrusives. The rocks are highly brecciated and weathered whilst the pillow structure at the dam site area is obscured because of the very high fracturing and brecciation. Due to the above limitations their strength is considerably lowered.

ii. Upstream Dam

This small dam is also built on the Lower Pillow Lava series. However, the basaltic intrusives are predominant. They are harder, of dark brown colour, and iron stained with smaller vesicles.

4. MAIN FEATURES

Both dams and their common distribution system were designed and constructed by the WDD.

a. Dams

They are mass masonry gravity dams built of igneous rocks from nearby quarries. The upstream faces are vertical whilst the downstream are 1:0.75 and 1:0.67 for the downstream and upstream dams respectively.

Sluicing galleries and operating penstocks are also provided for both dams. In the case of the upstream dam a system of perforated pipes was laid in the foundation and the masonry structure for drainage purposes which were extended to discharge into the gallery.

b. Distribution System

This is common to both dams and more than half of it is unlined, the remaining being built in masonry.

5. CONSTRUCTION

The first dam to be constructed was the downstream one, work on which started in November 1944. It was completed in May 1945 including excavations of the channels and building of some of them in masonry.

The rocks used for the building of the dams are grey hard diabase and few gabbros. These rocks were brought from a site between Analiondas and Lythrodhondas villages, 2 miles away from the damsite. Rocks from the site were also used. Chalks were also used for the construction. These were brought from Lymbia village area.

The foundations were filled in lime cement concrete 1:2:9 and the dambody was built in random rubble set in lime cement mortar 1:2:9. The upstream faces were plastered with three coats of cement mortar to prevent seepage through the structure. The downstream face and the wingwall masonry were pointed in cement mortar. The total cost of the dam was £2,000 and that of the distribution system £4,000.

Subsequently, in July 1950 the dam was raised by 0.67 m at a cost of £1,100. Also in November 1969 a new gallery gate was installed at a cost of £300. Thus the total cost on the downstream dam was £3,400.

Work on the upstream dam was started in January 1952 and completed in September 1952 at a cost of £3,500.

The foundations were filled in lime cement concrete 1:2:3.6 and the masonry dambody in 1:3:12 lime cement mortar.

The contribution of the Irrigation Division on all works described was £3,475 payable over 15 years at a rate of interest of 3%.

6. MANAGEMENT

The works are managed by the Irrigation Division of Lythrodhondas which commands about 6,000 donums of land. A small portion of this area is irrigated in beans, potatoes and other early summer vegetables, whereas the remainder is cultivated in wheat, barley and olive trees.

7. PROBLEMS OF SPECIAL INTEREST

Some leakage developed from the abutments of the upstream dam which was measured to be 1 l/s from the left abutment and 0.1 l/s from the right abutment at full reservoir level.

In May 1971 an exploratory grouting programme was carried out on the left abutment by drilling and grouting two boreholes drilled through the dambody. The one borehole was drilled to a depth of 10.5 m into the rock, the first 1.5 m being through the masonry dambody. The other borehole was drilled through a depth of 12 m into rock, the first 5 m being through the masonry. The geology was all pillow lava, but at the top it was highly fractured and weathered and became tighter as the depth was increased.

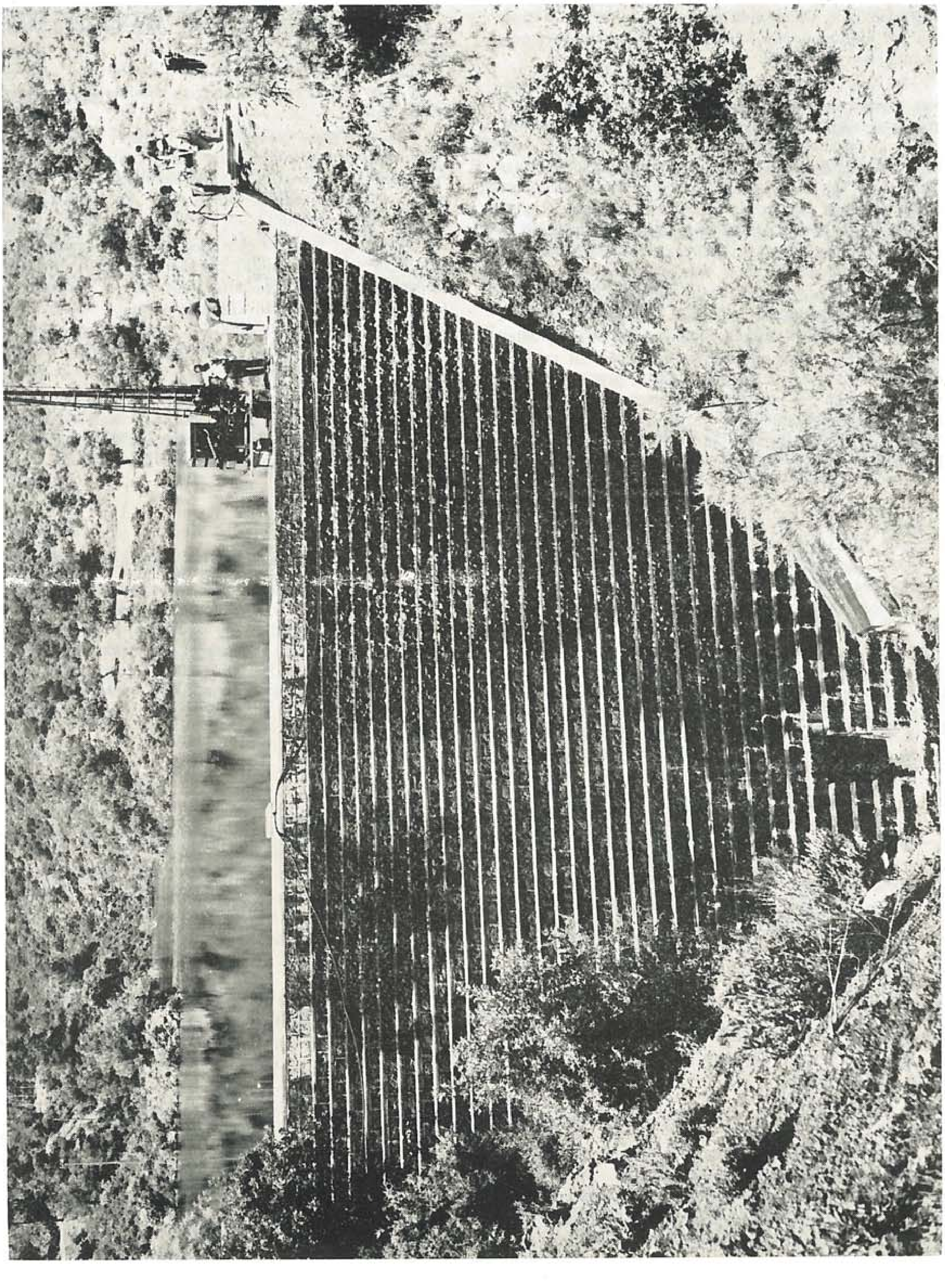
Water pressure tests were carried out into the rock at 1.5 m stages, the maximum permeability reached being 152 lugeons. The boreholes were then grouted with a mixture starting from a water cement ratio of 6:1 and ending with a ratio of 2:1 at a pressure of 1.35 to 0.45 kg/cm^2 per metre depth in stages of 1.5 to 3 m, the maximum consumption having reached 115 kg/m. The total cement injected in both boreholes was 525 kg. The result was that the leakage decreased by about 43% from 1 l/s to 0.57 l/s. The cost of this exploratory grouting operation was £260.

Between March and April 1973 additional grouting was carried out on the left abutment by drilling and grouting a row of 13 additional holes, 1.2 m apart, having a total depth of 115 m with the aim to stop or at least reduce to the minimum possible the leakages of the dam.

Ten of these holes were drilled through the dambody and the other three on the abutment. The total grout consumption of all the holes was 2,100 kg of dry materials out of which 2.000 kg were cement and 100 kg bentonite.

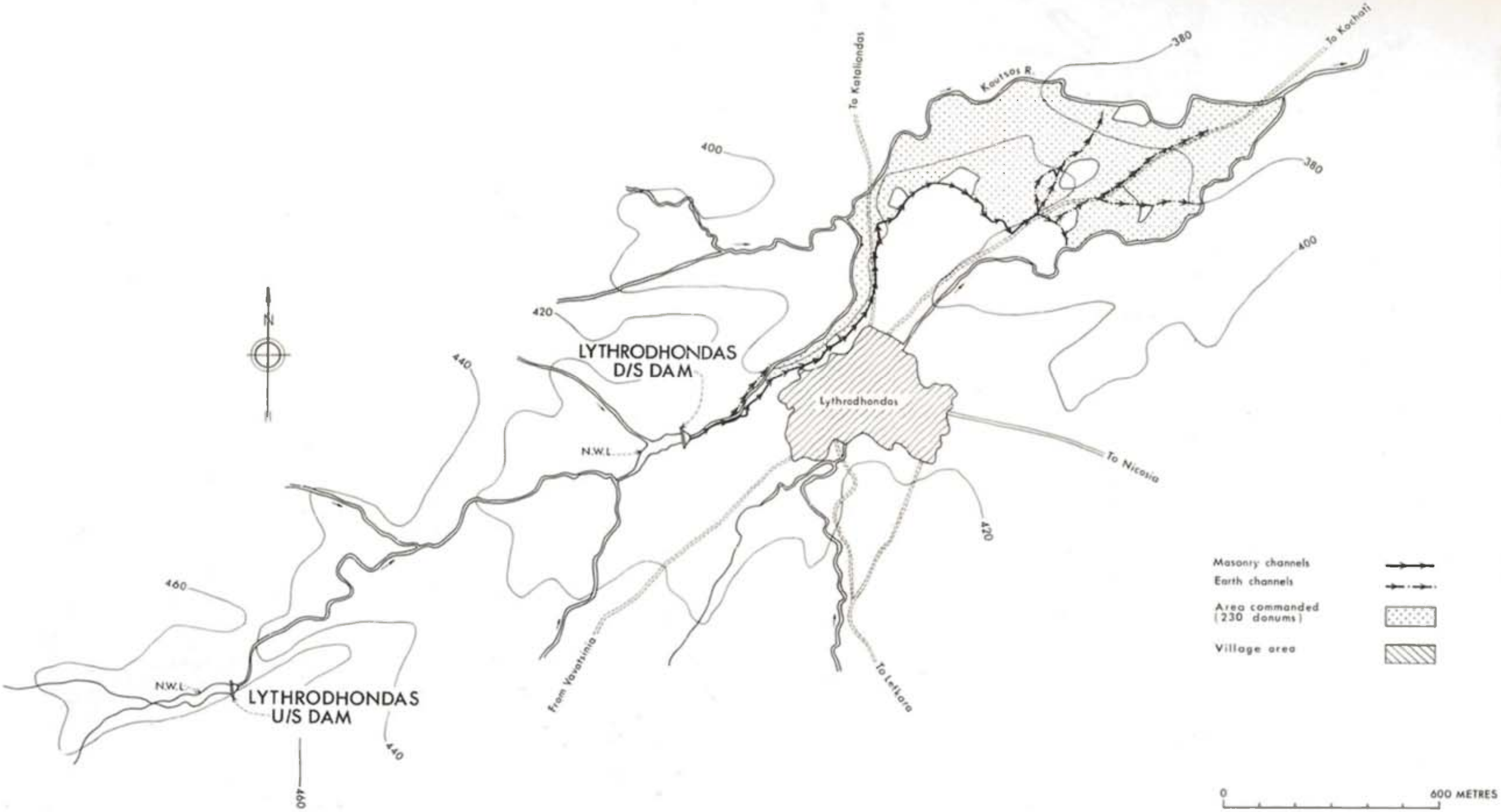
The grout mixture consisted of water, cement and bentonite in ratios ranging between 6:1 to 2:1 of water to cement-bentonite, the latter being 5% of the cement. Grout pressures of 0.45 kg/cm^2 per metre depth were applied. The total drilling for grouting was 137 m with a total consumption of 2,625 kg giving an average consumption of 19.2 kg/m.

An inclined check hole was drilled through the grouted zone and the permeabilities obtained ranged from 3.5 to 5.5 lugeons which are considered satisfactory. The total cost for these works was £450.

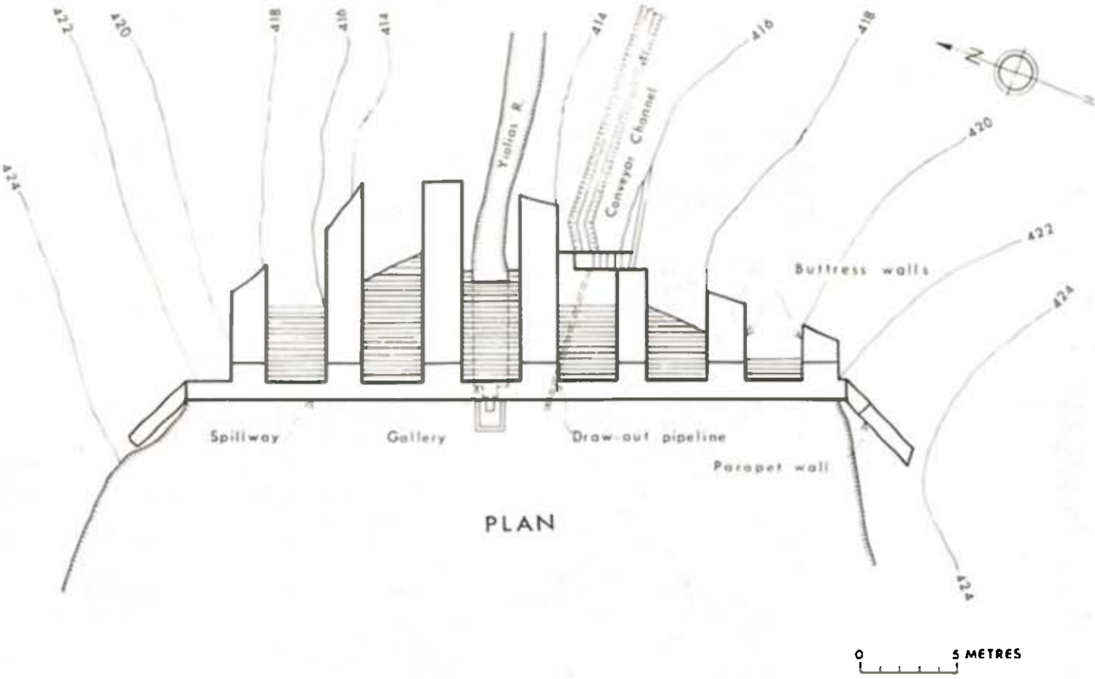


The Lythrodhondas upstream dam showing subsequent grouting works.

YIALIAS-LYTHRODHONDAS-DISTRIBUTION SYSTEM



YIALIAS-LYTHRODHONDAS D/S DAM



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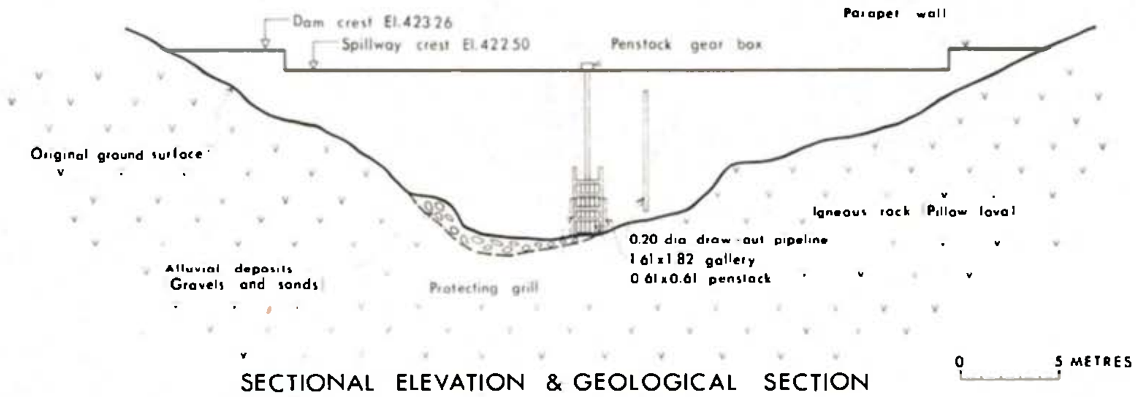
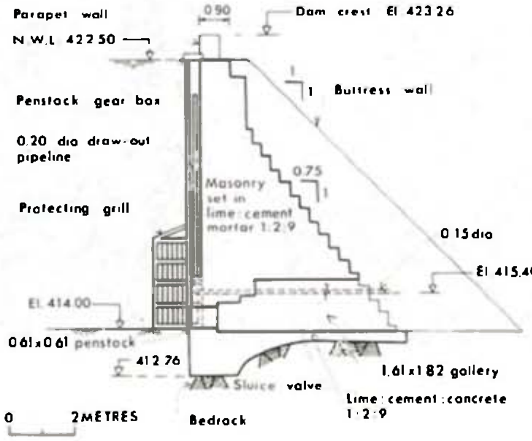


SECTIONAL ELEVATION & GEOLOGICAL SECTION



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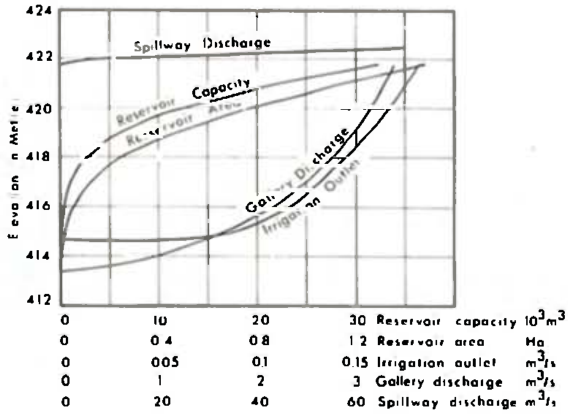
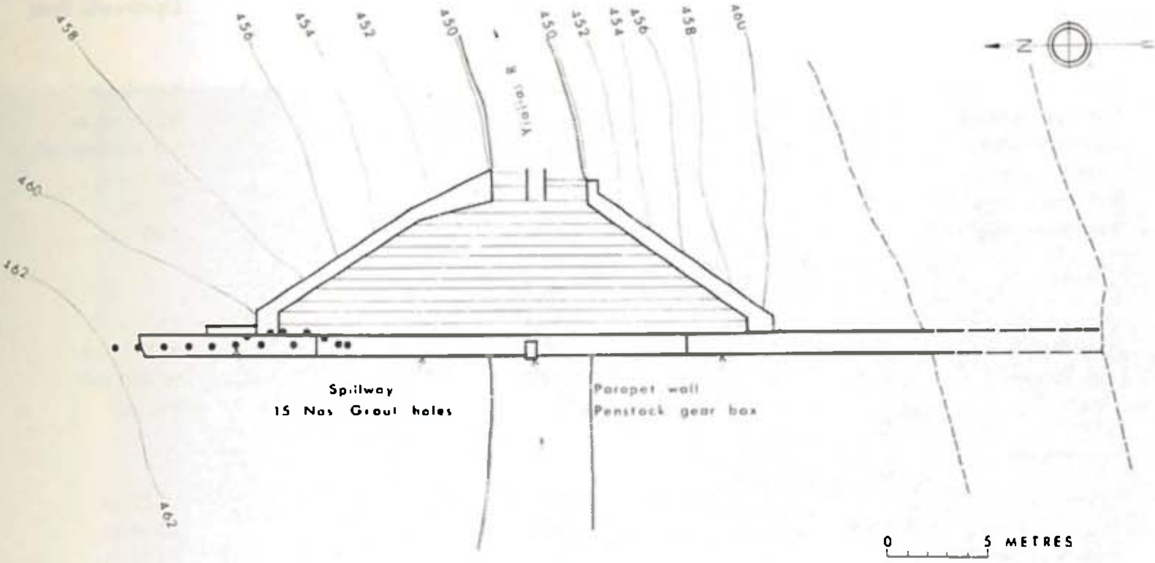


CAPACITY-AREA-DISCHARGE CURVES

YIALIAS-LYTHRODHONDAS U/S DAM



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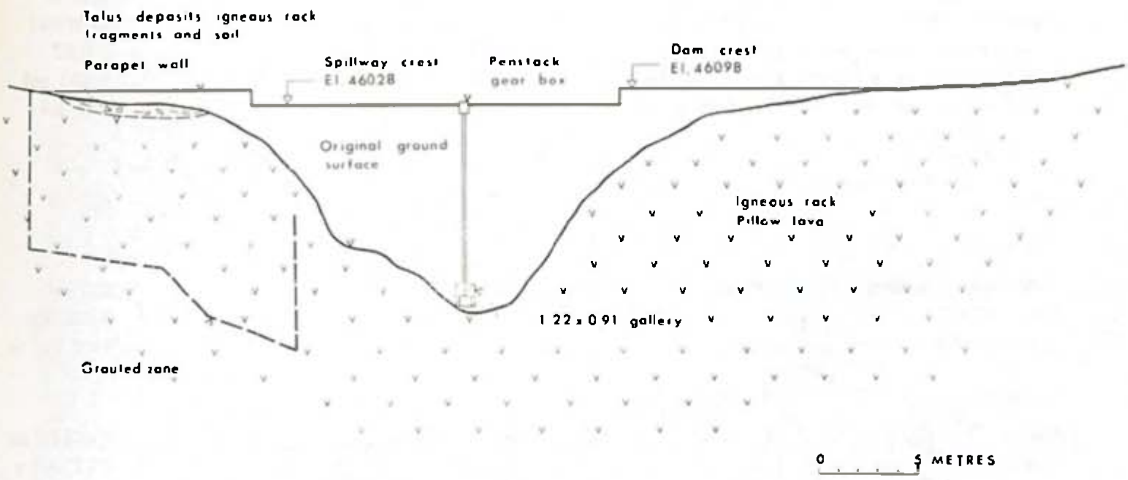

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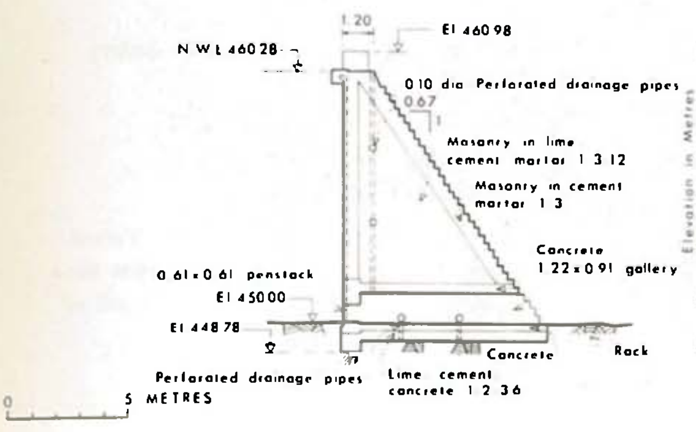
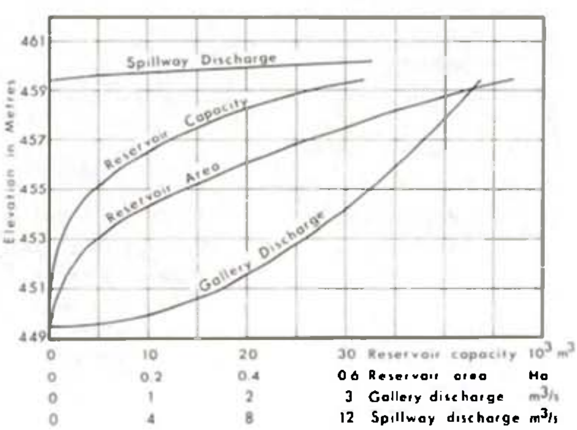


MAXIMUM SECTION



CAPACITY-AREA-DISCHARGE CURVES

Data

	Downstream Dam	Upstream Dam
i. Catchment		
Area	8.68 km^2	3.03 km^2
Average rainfall	562 mm/a	585 mm/a
Average runoff	0.93 million m^3/a	0.4 million m^3/a
1/1000 years flood	72 m^3/s	29 m^3/s
Maximum height	745 m	745 m
Maximum length	4.27 km	2.65 km
ii. Reservoir		
Area	1.5 ha	1.0 ha
Capacity	39,600 m^3	29,800 m^3
Live storage	32,000 m^3	29,000 m^3
Length	355 m	275 m
iii. Dambody		
Structural height	10.50 m	12.20 m
Height above ground level	9.26 m	10.98 m
Hydraulic height	8.44 m	10.28 m
Depth of foundation cut-off	1.24 m	1.22 m
Freeboard	0.76 m	0.70 m
Crest length	42.25 m	21 m
Top thickness	0.9 m	1.2 m
Base thickness	6.30 m	8.40 m
Upstream slope	Vertical	Vertical
Downstream slope	1:0.75	1:0.67
Total volume of masonry fill of dambody	2,500 m^3	2,000 m^3
Total volume of concrete foundation fill	400 m^3	330 m^3
iv. Excavations		
Total	400 m^3	330 m^3
v. Foundation Treatment		
Total drilling depth for grouting	—	137 m
Total grout take	—	2,625 kg
Average grout take	—	19.2 kg/m
vi. Spillway		
Size	33.84x0.76 m	19x0.70 m
Capacity	70 m^3/s	13.1 m^3/s
vii. Gallery		
Size	1.82x1.61 m	1.22x0.91 m
Capacity	3.37 m^3/s	4.35 m^3/s
Length	5.23 m	7.55 m
Operating gate size	0.61x0.61 m	0.61x0.61 m
viii. Outlet (steel pipes)		
Size	0.15 and 0.20 m dia	The gallery
Capacity	0.182 m^3/s	
Length	7 m	

ix. Distribution System

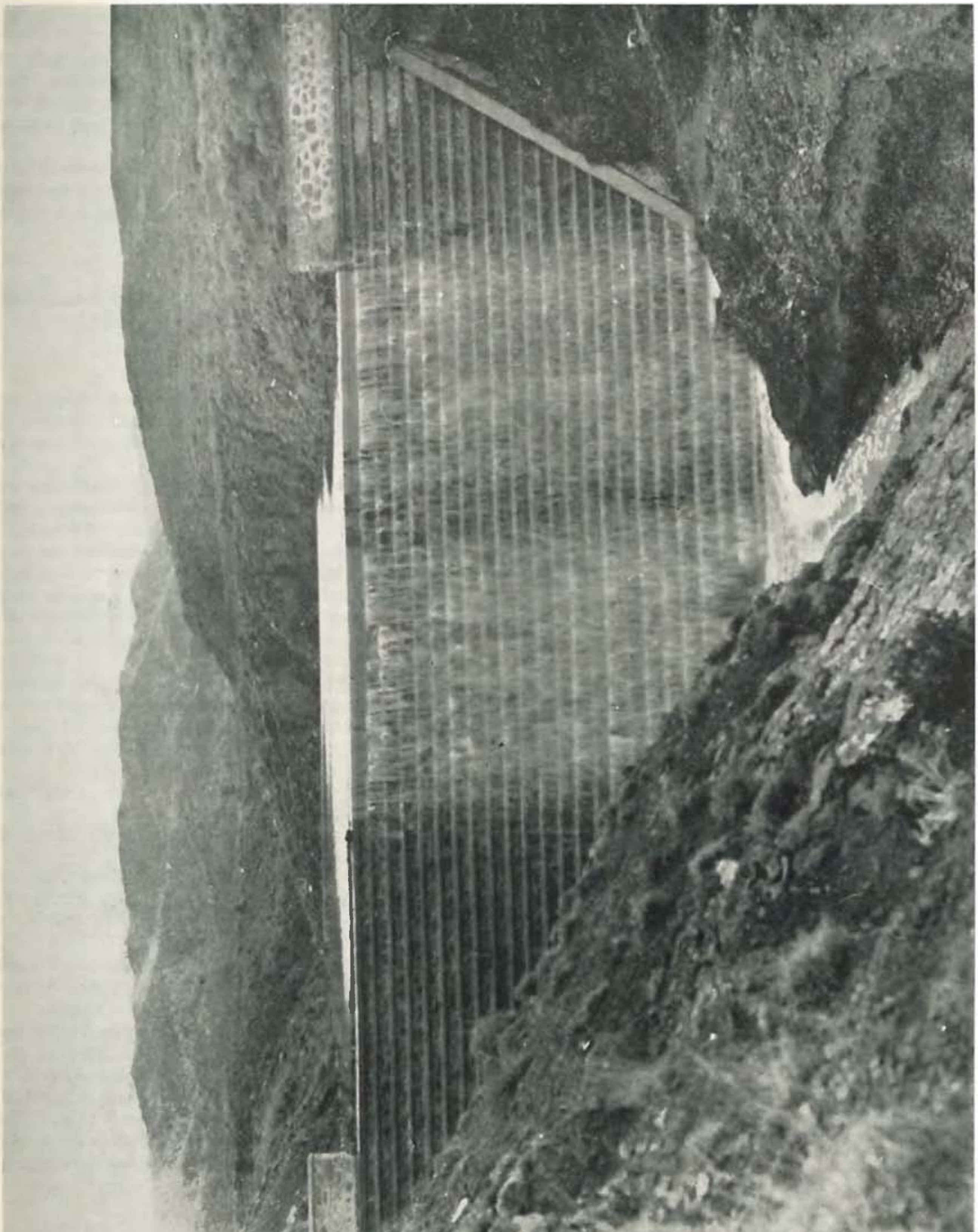
(i) Masonry channels

Size	Rectangular	0.70x0.60 and 0.60x0.60 m
	Trapezoidal	(0.55 + 0.40)x0.30 m
	"	(0.55 + 0.40)x0.60 m
	"	(0.55 + 0.50)x0.30 m
Capacity		0.056 m^3/s
Length		1,430 m

(ii) Earth channels

Size	Varied
Capacity	0.056 m^3/s
Length	1,930 m

PETRA



The Petra downstream dam.

ATSAS—PETRA DOWNSTREAM AND UPSTREAM PROJECTS

1. PURPOSE

Both dams were built to provide summer water by storing winter and spring flows of the Atsas river, for early summer crop irrigation of the lands belonging to the members of the Petra - Evrykhou Irrigation Division. Before the construction of the dam the available land was planted in wheat and barley whereas the availability of water from the dams makes it possible to irrigate spring crops such as legumes, potatoes and other vegetables.

First the downstream dam was built in 1947 and subsequently the upstream dam in 1951.

2. LOCATION

The two dams are situated on the Atsas river, 0.7 km from each other, located at about 4 km upstream of Petra, the downstream one being at an elevation of about 250 m and the upstream 270 m asl.

3. PLAN

a. Water and Land

For both the upstream and downstream dams the average annual runoff is about 3.8 million m^3. An area of 4,690 donums of land is commanded by the distribution system. However, much more water would be needed to irrigate the whole area.

b. Geology

i. Downstream Dam

The dam is built on the basal group lavas which appear to consist of multiple altered andesitic or basaltic dykes and pillow lavas into which the former have been intruded. The dykes constitute almost 100% of the exposures. Pillow lavas were not observed in the damsite area.

Near the dam axis the dykes are fine grained highly fractured jointed and iron stained giving thus a reddish brown colour to the rock. Near the tail of the reservoir the dykes are grey, fine grained and less iron stained.

A thin layer of loosely cemented gravels is also observed near the tail of the reservoir. The gravels are of igneous origin, their composition varying from diabase to gabbros.

ii. Upstream Dam

This dam is also built on the basal lavas the composition of which is as previously described. The dykes are greenish grey vesicular and have a general strike NS with a dip of about 40° W.

The main sets of joints were found to have a strike of 44° NE with a dip of 60° E and 58° NE with a dip of 75° N.

On the right abutment there is a thin layer up to 1.3 m of talus and fanglomerates.

4. MAIN FEATURE

Both dams and their joint distribution system were designed and constructed by the WDD.

a. Dams

They are mass masonry gravity dams built of igneous rocks obtained from nearby quarries. The upstream faces are vertical whilst the downstream are 1:0.56 in the case of the downstream dam and 1:0.67 in the case of the upstream dam.

Sluicing galleries and operating penstocks are provided for both dams. In the case of the upstream dam a drainage system of perforated pipes was laid in the foundation.

b. Distribution System

This is common to both dams, part of it being built in masonry and part of it in cement concrete.

5. CONSTRUCTION

The first dam constructed was the dowstream one, work on which started in May 1947 and completed in June 1948. The work included the first phase masonry canalization system.

The rocks used for the building of the dams are of various types, mainly of fine grained andesites and diabase. Some gabbros were also used. The building rocks were obtained from the rocks found in the dam site area and the gravels near the tail of the reservoir.

On the second dam, built 0.7 km upstream, work started in June 1951 and completed in November 1951.

The foundations of both dams were filled in lime cement concrete 1:3:9. The dambody was built in masonry set in lime cement mortar 1:3:8. The upstream faces were plastered with three coats of cement mortar to prevent seepage through the structure. The downstream faces and wingwalls were pointed in cement mortar.

The cost of the downstream dam was £4,750 and of the upstream one £5,000.

The cost of the first phase distribution system lined in masonry was £4,750.

As more reservoir capacity was required, the two reservoirs were excavated between October and November 1957. This increased their capacity by 6000 m^3 in each case at a total cost of £2,000.

Finally, a second phase of lining more earth channels in cast-in-situ concrete was carried out between October 1968 and April 1969 at a cost of £4,000.

Thus the total cost on both dams was £11,750 and that of the distribution system £8,750. The contribution of the Irrigation Division towards this expenditure was £2,375 on the dams and first phase distribution, £667 on the enlargement of the reservoir and £1,333 on the second phase of channel lining, giving a total contribution of £4,375 payable at a low interest long period loan.

6. MANAGEMENT

The Irrigation Division of Petra — Evrykhou manages the irrigation works commanding an area of about 4690 donums.

Data

	Downstream Dam	Upstream Dam
i. Catchment		
Area	37.1 km^2	33.73 km^2
Average rainfall	599 mm	618 mm
Average runoff	3.8 million m^3/a	3.8 million m^3/a
1/1000 years flood	260 m^3/s	250 m^3/s
Maximum height	1185 m	1185 m
Maximum length	16.3 km	15.7 km
ii. Reservoir		
Area	0.8 ha	0.4 ha
Capacity	31,800 m^3	18,000 m^3
Live storage	31,000 m^3	17,000 m^3
Length	265 m	170 m
iii. Dambody		
Structural height	12.23 m	13.30 m
Height above ground level	10.23 m	10.25 m
Hydraulic height	9.11 m	9.14 m
Depth of foundation cut-off	2.00 m	3.05 m
Freeboard	1.12 m	1.11 m
Crest length	36 m	35 m
Top thickness	2.40 m	1.10 m
Base thickness	7.00 m	8.38 m
Upstream slope	Vertical	Vertical
Downstream slope	1:0.56	1:0.67
Total volume of masonry fill of dambody	1100 m^3	1000 m^3
Total volume of concrete foundation fill	450 m^3	500 m^3
iv. Excavations		
Total	450 m^3	500 m^3
v. Spillway		
Size	21.2x1.21 m	30.47x1.11 m
Capacity	31.6 m^3/s	52 m^3/s
vi. Gallery		
Size	1.35x1.21 m	1,22x0.91 m
Capacity	7.2 m^3/s	4.16 m^3/s
Length	5.63 m	6.85 m
Operating gate size	1.25x0.70 m	0.61x0.61 m
vii. Outlet (steel pipes)		
Size	0.20 m dia	0.10 m dia
Capacity	0.25 m^3/s	0.0485 m^3/s
Length	5.80 m	10.50 m

viii. Distribution System

(i) Masonry channels

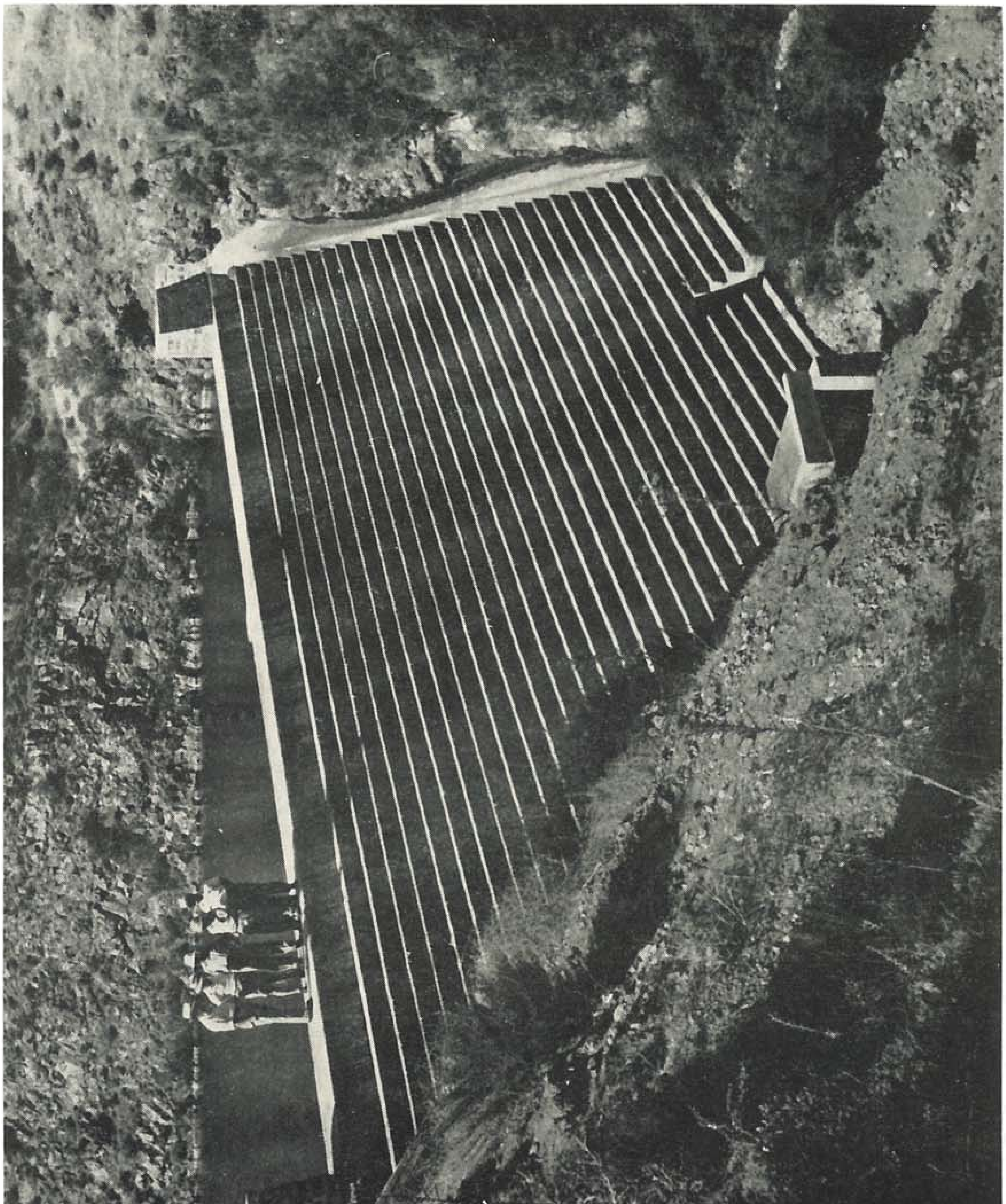
Size	1x0.75 and 0.60x0.60 m
Capacity	0.075 m^3/s
Length	2360 m

(ii) Concrete rectangular channels

Size	1x0.50 and 0.80x0.40 m
Capacity	0.075 m^3/s
Length	870 m

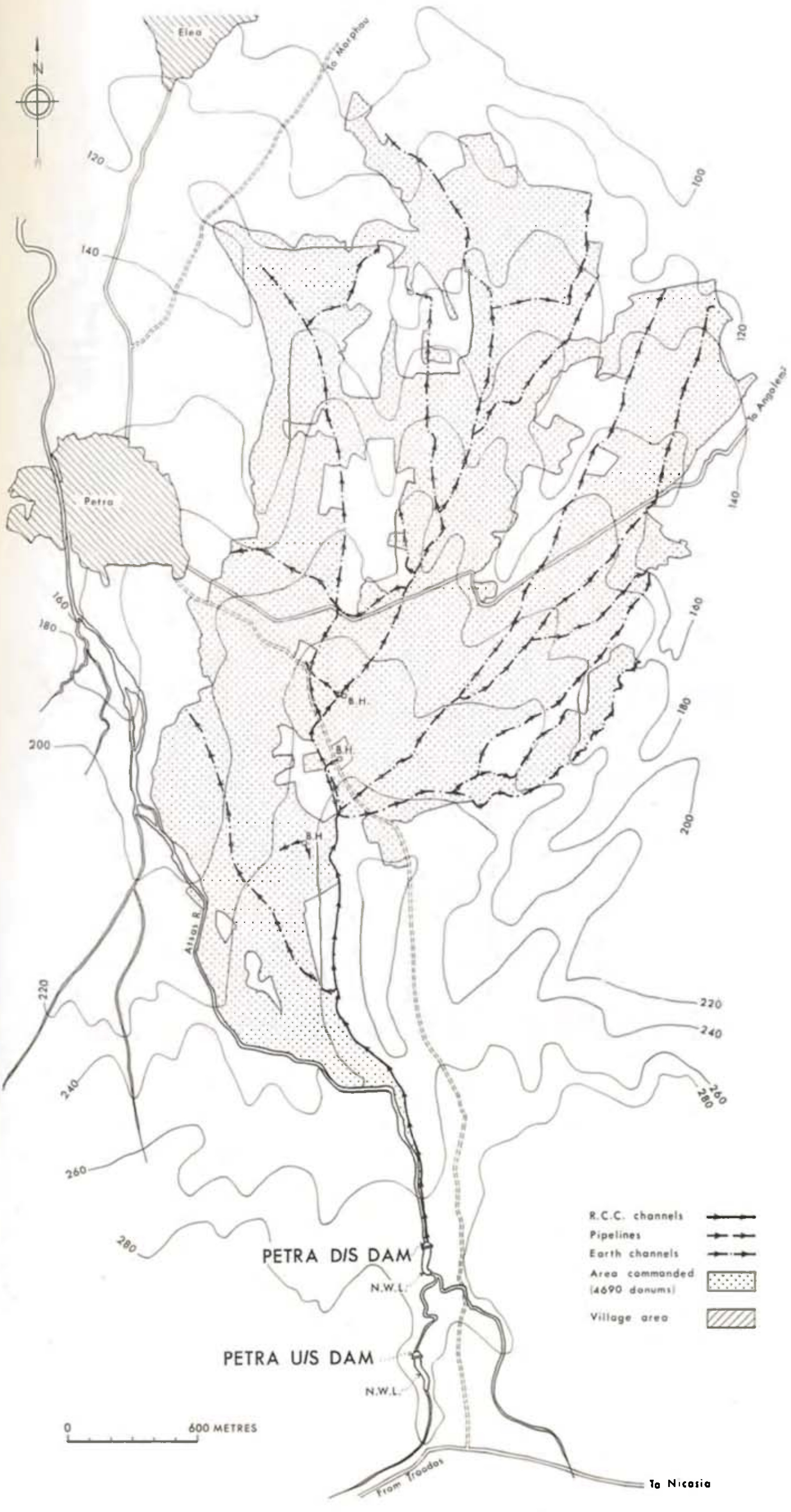
(iii) Earth Channels

Capacity	0.075 m^3/s
Length	13,000 m

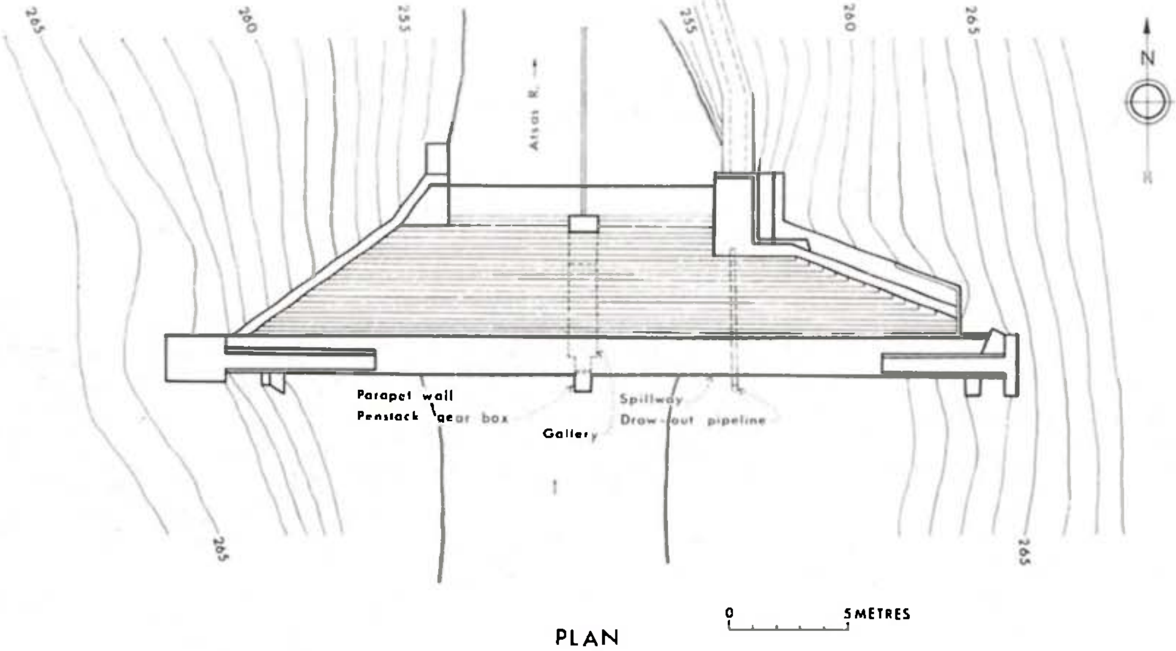


The Petra upstream dam.

ATSAS-PETRA-DISTRIBUTION SYSTEM



ATSAS-PETRA D/S DAM



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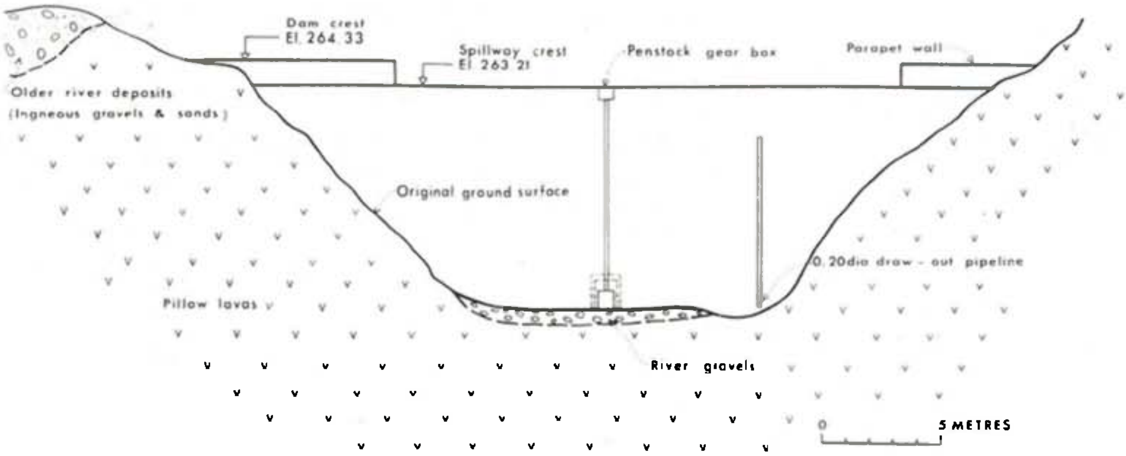

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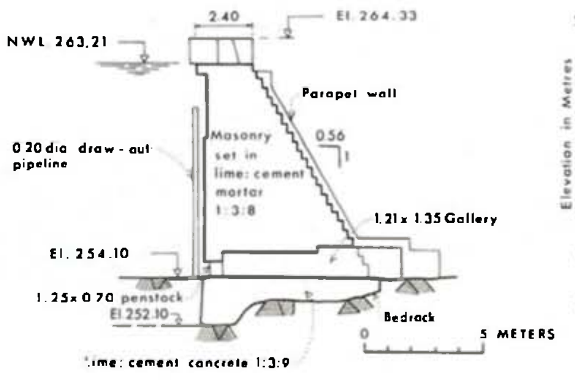
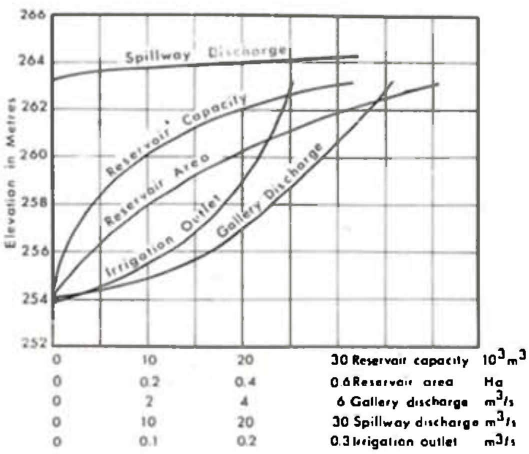
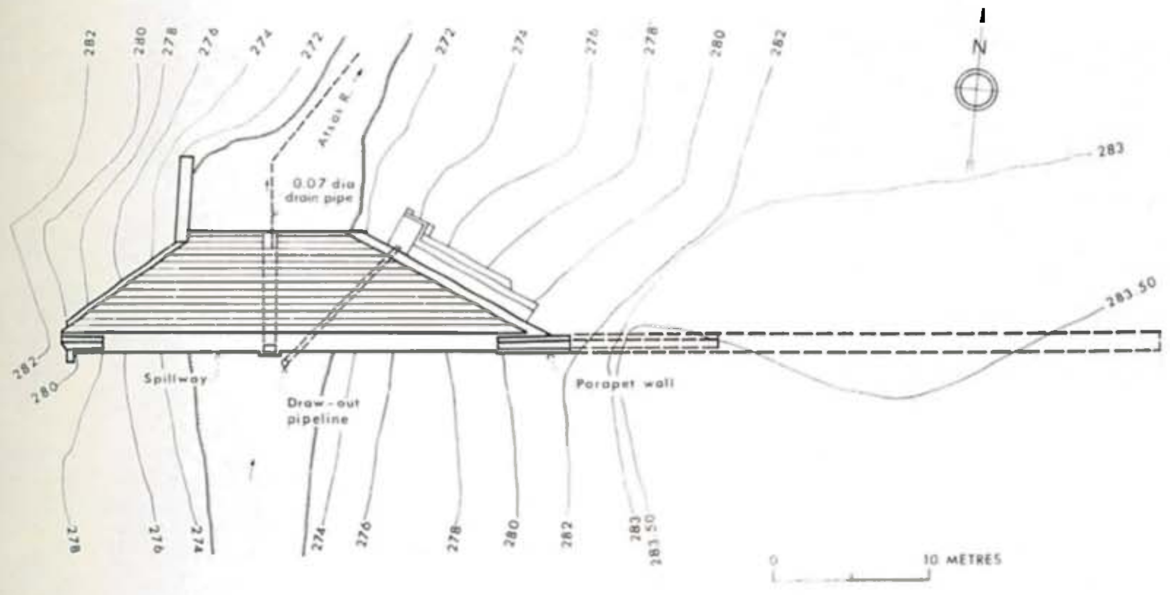


MAXIMUM SECTION



CAPACITY-AREA-DISCHARGE CURVES

ATSAS-PETRA U/S DAM



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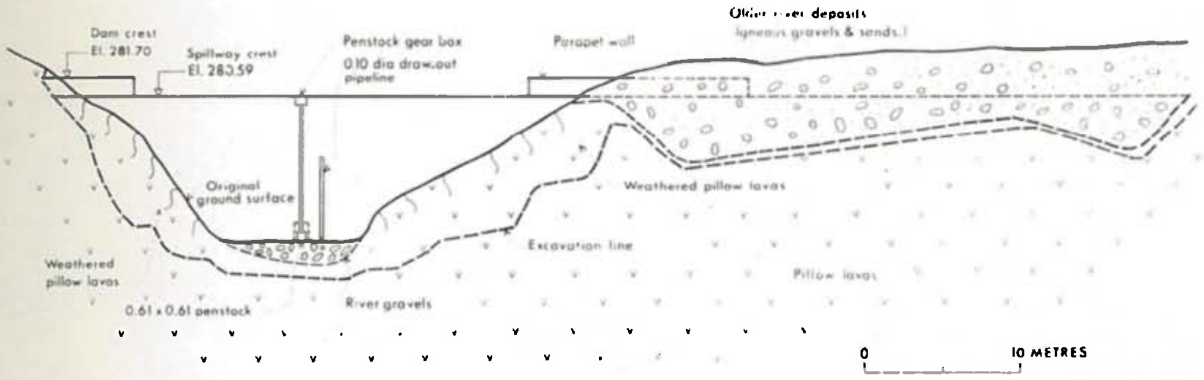

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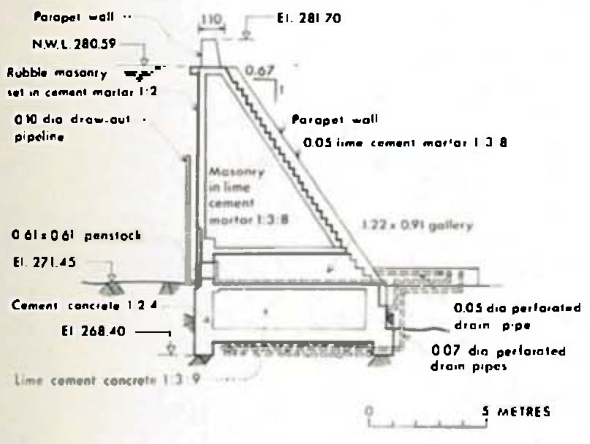

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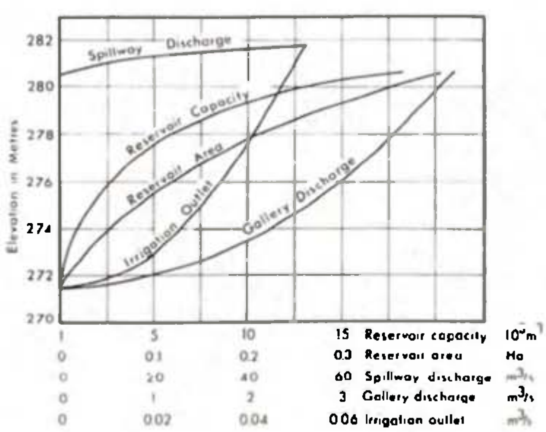


CAPACITY-AREA-DISCHARGE CURVES

GALINI

The Galini dam.

KAMBOS—GALINI PROJECT

1. PURPOSE

The dam and distribution system were constructed to ensure perennial water supply to the lands of the Irrigation Division of Galini which previously received water from an intake on this river for the spate irrigation of winter and spring crops. The supply of perennial water enables the cultivation of more remunerative perennial crops. Galini had at the time no other source of water than the intake on the Kambos river, but in later years the groundwater in the river alluvial terrace of Kambos has been privately developed by the same owners of land. The importance of building the dam at the time could not be ignored for a village like Galini which has very poor resources.

2. LOCATION

The dam has been built on the Kambos River at a distance of 6 km from the sea and at an elevation of about 170 m asl.

3. PLAN

a. Water and Land

The average annual runoff is about 5.2 million m^3. The Irrigation Division lands now extend to 1,140 donums, out of which 710 citrus, 110 olives, 20 bananas and 300 potatoes and vegetables. All the land extends as a border strip on either side of the river bed starting from the dam and reaching as far as the coast.

b. Geology

The dam is constructed on the Basal Group series. Here the diabase intrusives are dominant. These intrusives are fine grained, grey, having a general NS trend dipping 66° W.

The intrusives are fractured with an average strike of EW and dipping 58° S.

4. MAIN FEATURES

a. Dam

The dam was designed by the WDD as a masonry gravity dam using diabase rock from the site. The upstream face is vertical with two steps whereas the downstream face has a slope of 1:0.5. The downstream face is stepped to reduce the velocity of the overflow water.

There is an outlet gallery through the masonry 1.55x1.50 m used for desilting purposes. An upstream gate regulates the entrance to the gallery opening by a gear mechanism on the crest of the dam.

b. Distibution System

The conveyance system designed by the WDD was at first excavated in earth and was later built in masonry and finally in reinforced concrete.

5. CONSTRUCTION

The dam was constructed by the WDD. Work started in April 1946 and completed in June 1947.

The total volume of excavations was 220 m^3, and that of the masonry fill 700 m^3. The foundations were filled in lime-cement-concrete 1:2:9 and the masonry fill was placed in lime-mortar 1.2:9. Diabase rocks used for the masonry work were obtained from the site and the concrete aggregate from the nearby sea shore. The upstream face was plastered in three coats of cement mortar to prevent seepage through the structure, whilst the downstream face and wingwall masonry were pointed in cement mortar. The total cost of the dam was £4,000.

The excavation and lining of the distribution system was done in stages.

The first stage was between April 1946 and June 1947 with channels in rubble masonry built in lime-cement mortar at a cost of £11,280. Between November 1950 and March 1951 another £2,200 was spent on the lining, in reinforced concrete, of earth channels. Between January and April 1969 £6,000 was spent on similar work and finally from March 1970 to March 1971 £19,000 was spent on completing the lining of all the canalized system in reinforced concrete. Thus the total amount spent on the distribution system reached £38,480. For the construction of the works the Irrigation Division signed a loan to pay £11,884 over 10 years at a rate of interest of 3%.

6. MANAGEMENT

The project is managed by the Galini Irrigation Division the members of which own 1,300 donums of land. A number of the owners of the Division have also private boreholes which they use for the irrigation of their lands in addition to the water received from the dam. The use of ground water is the main factor enabling the cultivation of the permanent crops made up of 710 donums of citrus and 20 donums of bananas. The Irrigation Division recently requested the raising of the dam and this is now under consideration.

Data

i. Catchment

Area	25.5 km^2
Average rainfall	750 mm/a
Average runoff	5.2 million m^3/a
1/1000 years flood	140 m^3/s
Maximum height	880 m
Maximum length	12 km

ii. Reservoir

Area	0.5 ha
Capacity	22,700 m^3
Live storage	22,700 m^3
Length	200 m

iii. Dambody

Structural height	13.24 m
Height above ground level	11.65 m
Hydraulic height	10.71 m
Depth of foundation cut-off	1.59 m
Freeboard	0.94 m
Crest length	19.20 m
Top thickness	2.25 m
Base thickness	6.10 m
Upstream slope	Vertical
Downstream slope	1:0.5
Total volume of masonry fill of dambody	700 m^3
Total volume of concrete foundation fill	220 m^3

iv. Excavations

Total	220 m^3

v. Spillway

Size	13.4x0.94 m
Capacity	17.2 m^3/s

vi. Gallery and Outlet

Size	1.55x1.50 m
Capacity	51 m^3/s
Length	7.31 m
Operating gate size	0.61x0.61 m

vii. Distribution System

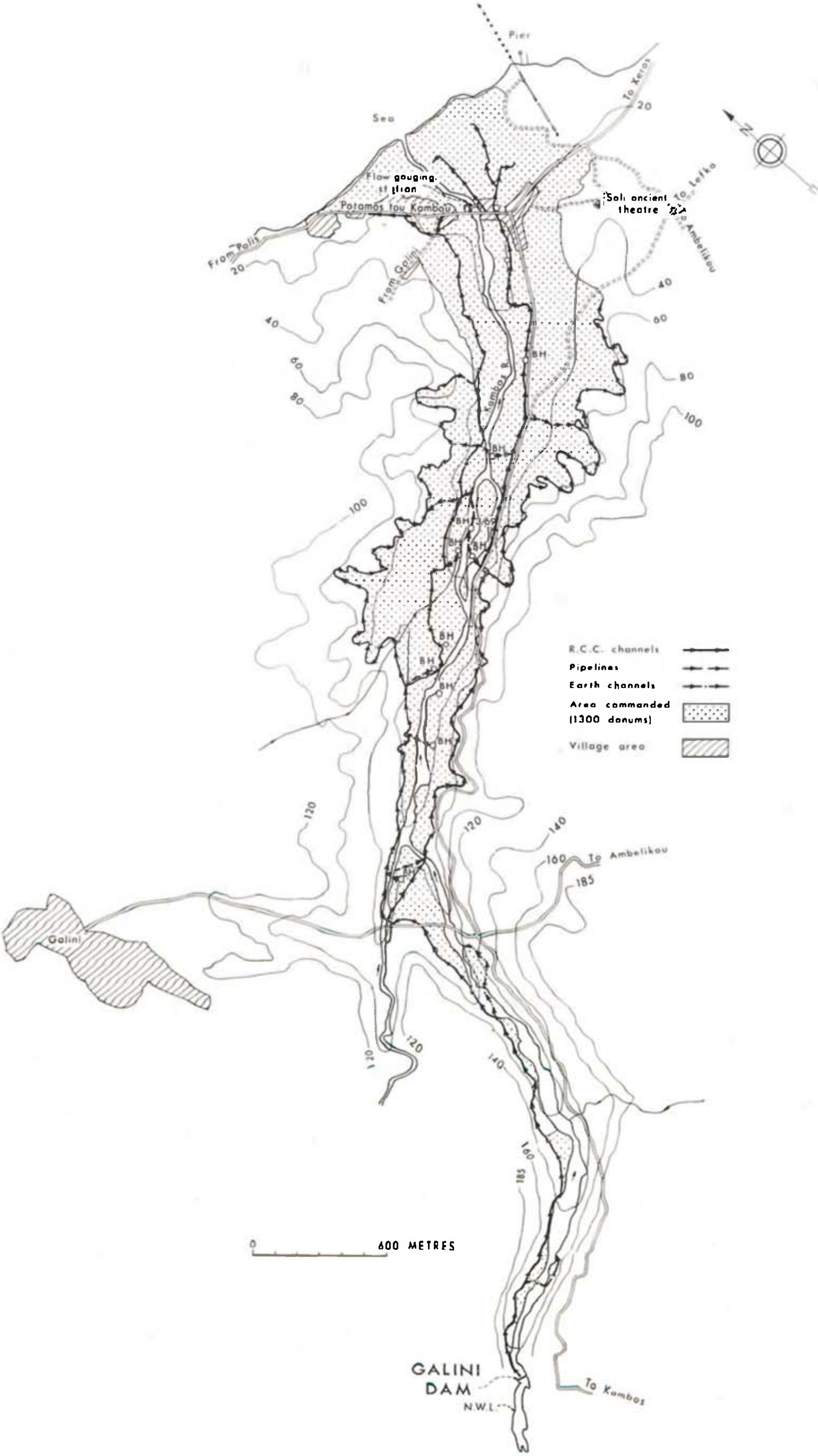
(i) Main Conveyor

Size	0.65x0.40 m and 0.65x0.50 m
Capacity	0.1 m^3/s
Length	2,530 m

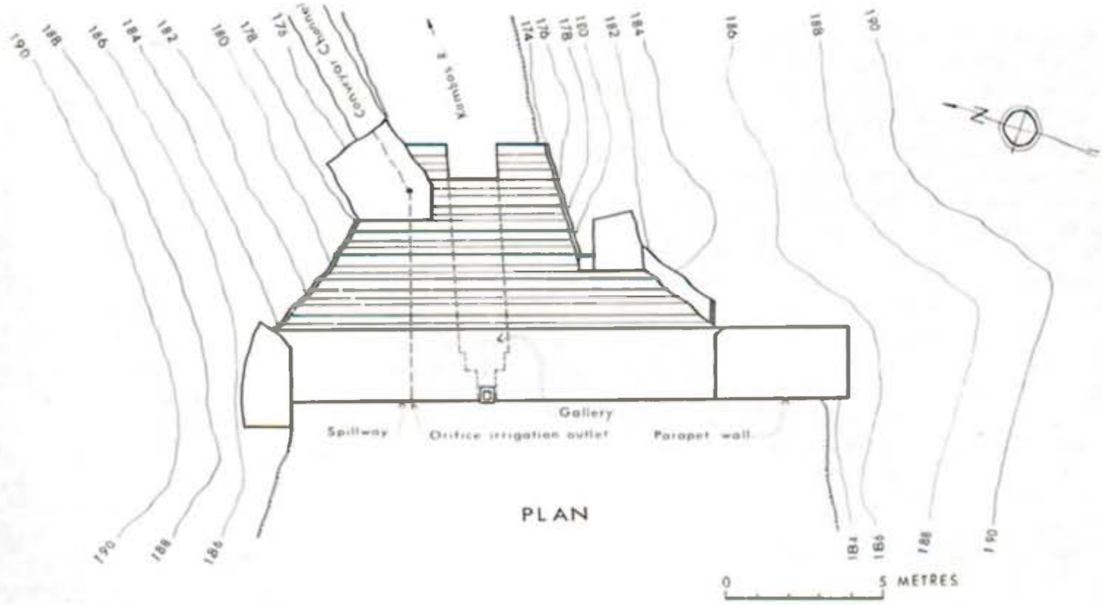
(ii) Distribution network (concrete channels)

Size	0.25x0.30 0.65x0.25
Capacity	0.07 m^3/s
Length	15,000 m

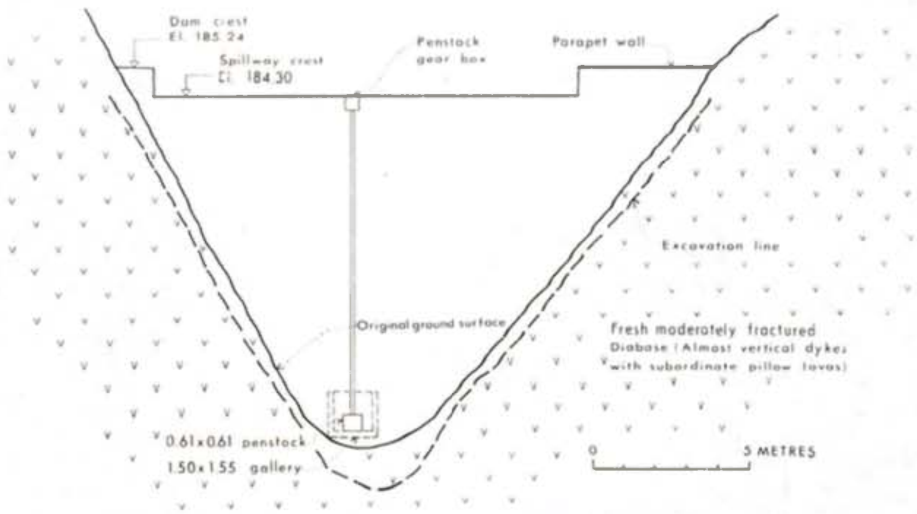
KAMBOS - GALINI - DISTRIBUTION SYSTEM



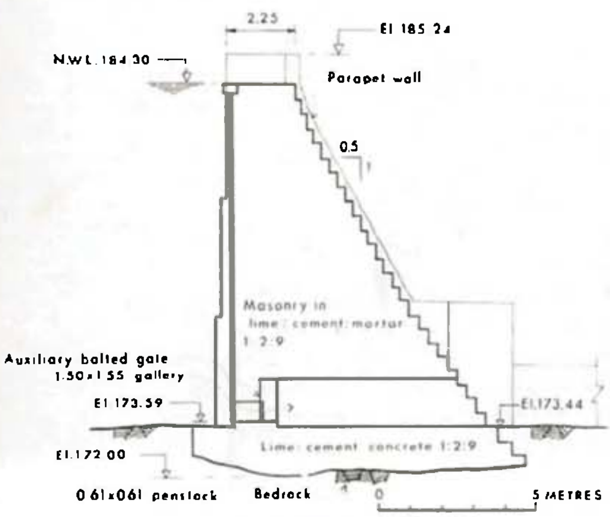
KAMBOS-GALINI DAM



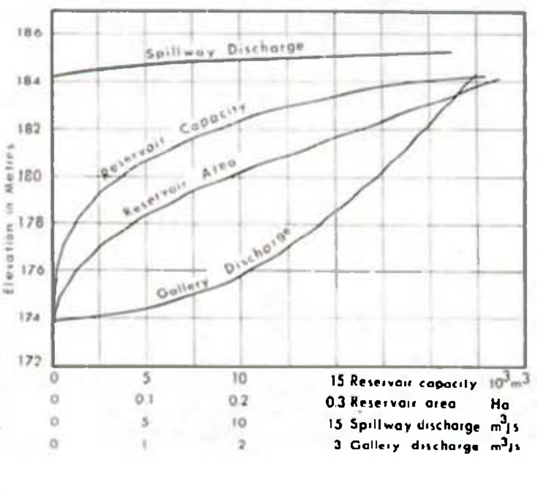
PLAN



SECTIONAL ELEVATION & GEOLOGICAL SECTION

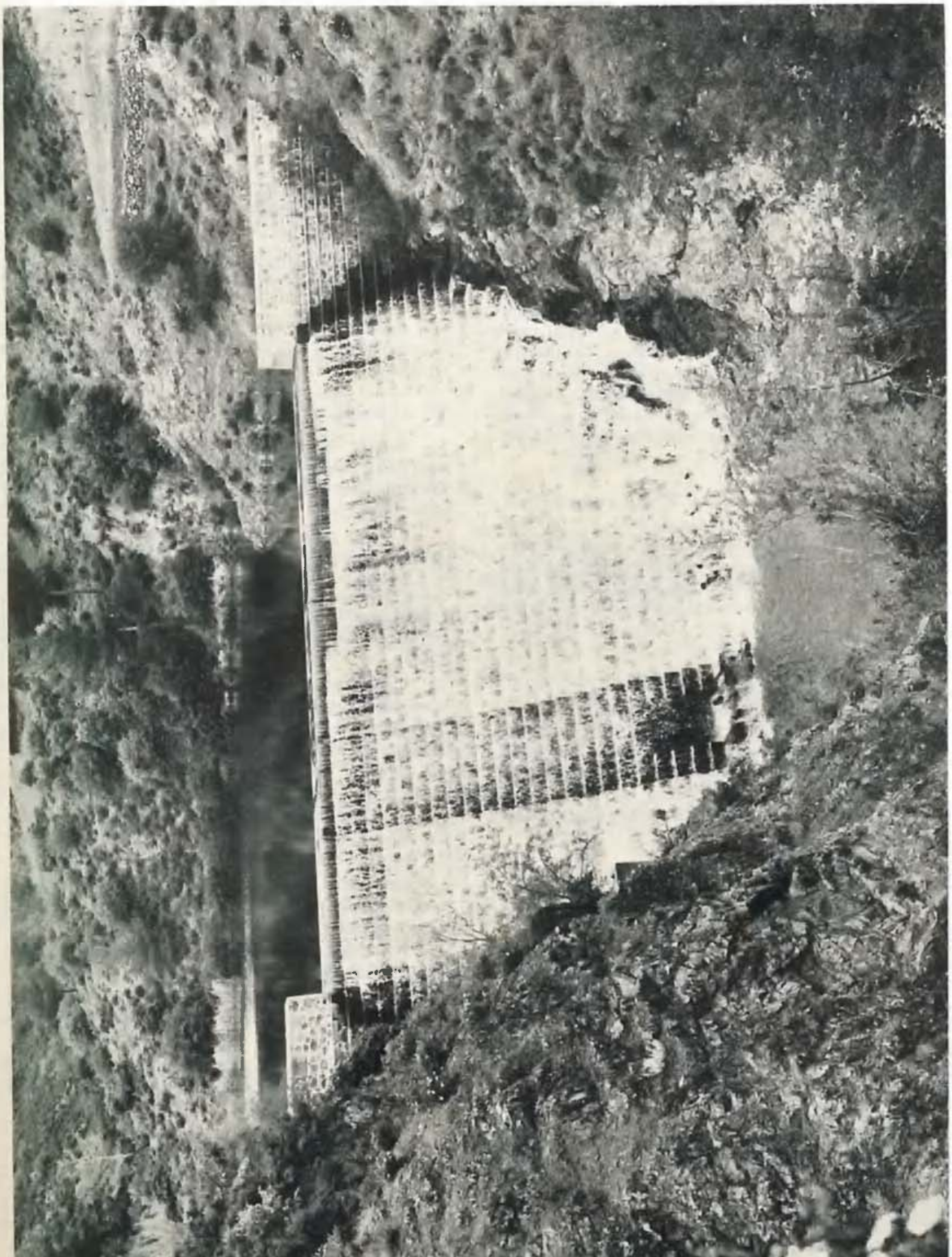


MAXIMUM SECTION



CAPACITY-AREA-DISCHARGE CURVES

KALOKHORIO



The Kalakhorio dam

SERAKHIS — KALOKHORIO (KLIROU) PROJECT

1. PURPOSE

The dam was built in order to ensure adequate water supply for the irrigation of perennial crops mainly vegetables, beans and potatoes. Kalokhorio lies on igneous rocks and very little groundwater of any significance is available. Therefore, the construction of a dam on the Serakhis river was an important contribution to provide some water for perennial irrigation.

2. LOCATION

The dam is situated on a tributary of the Serakhis river at a distance of about 2 km upstream of the Kalokhorio village and at an elevation of about 520 m asl.

3. PLAN

a. Water and Land

The annual average runoff is about 4.5 million m^3.

A lot of good fertile land is available, which is mostly cultivated in dry farming crops such as barley, wheat, olive and almond trees. The provision of summer water enabled the cultivation of vegetables, beans and potatoes. A total of 1350 donums is commanded by the canalization system.

b. Geology

The dam is constructed on the Basal Group series. Diabase intrusives occupy the damsite area.

Here the diabase is greenish-grey in colour, epidote rich, hard and fine grained. The intrusives have a general trend of 330° NW dipping to the West at an average angle of 75°.

The right abutment is terraced and cultivated. River terraces of varying depths are found in the area.

4. MAIN FEATURES

The dam and distribution system were designed by the WDD.

a. Dam

The design is of a mass masonry gravity dam using igneous rock from nearby quarries. The upstream face is vertical with two steps and the downstream face is of stepped design with a slope of 1:0.42.

A sluicing gallery 1.62x1.62 m incorporated in the masonry body is provided as well as a draw-off pipe for irrigation.

b. Distribution System

Most of the conveyance system originally in earth was lined partly in masonry of trapezoidal section at the beginning of the project and later on in rectangular concrete section.

5. CONSTRUCTION

The construction of both dam and distribution system was carried out by the WDD. Construction on the dam started in August 1947 and completed in July 1948.

The total foundations excavation was 450 m^3 which was filled in lime cement concrete of 1:2:9. The masonry used is diabase rock for the dambody and was random placed in lime cement mortar 1:2:9. The rock was obtained from the site.

The upstream face was plastered in three coats of cement mortar to prevent seepage through the structure. The downstream face and wingwall masonry were pointed in cement mortar.

The total cost of the dam was £6,300 including certain repairs on the dam gallery gate carried out at a later period.

The distribution system was carried out in three stages; originally in masonry, later in cast-in-situ concrete and finally it was raised to increase its capacity.

The first stage started in August 1947 and the last completed in 1964 at a total cost of £16,700.

The Irrigation Division contributed a total of £5,475 which was granted to it as a loan by the Government payable over 20 years at 3% rate of interest.

6. MANAGEMENT

An Irrigation Division was formed of land owners of the 1350 donums commanded by the distribution system who are responsible for the distribution of the water.

Data

i. Catchment

Area	23.4 km²
Average rainfall	670 mm/a
Average runoff	4.5 million m³/a
1/1000 years flood	128 m³/s
Maximum height	1170 m
Maximum length	9.7 km

ii. Reservoir

Area	1.3 ha
Capacity	36,000 m³
Live storage	30,000 m³
Length	360 m

iii. Dambody

Structural height	13.77 m
Height above ground level	12.12 m
Hydraulic height	10.90 m
Depth of foundation cut-off	1.65 m
Freeboard	1.22 m
Crest length	37.00 m
Top thickness	1.70 m
Base thickness	8.50 m
Upstream slope	Vertical
Downstream slope	1:0.42
Total volume of masonry fill of dambody	1,500 m³
Total volume of concrete foundation fill	450 m³

iv. Excavations

Total	450 m³

v. Spillway

Size	22.5x1.22 m
Capacity	44.1 m³/s

vi. Gallery

Size	1.62x1.62 m
Capacity	1.13 m³/s
Length	7 m
Operating gate size	0.61x0.61 m

vii. Outlet (steel pipes)

Size	0.15 m
Capacity	0.133 m³/s
Length	8.6 m

viii. Distribution System

(i) Masonry Trapezoidal Channels

Size	0.67 + 0.30x0.45 m
Capacity	0.113 m³/s
Length	900 m

(ii) Masonry Rectangular Channels

Size	0.60x0.30 and 0.60x0.45 m
Capacity	0.113 m³/s
Length	900 m

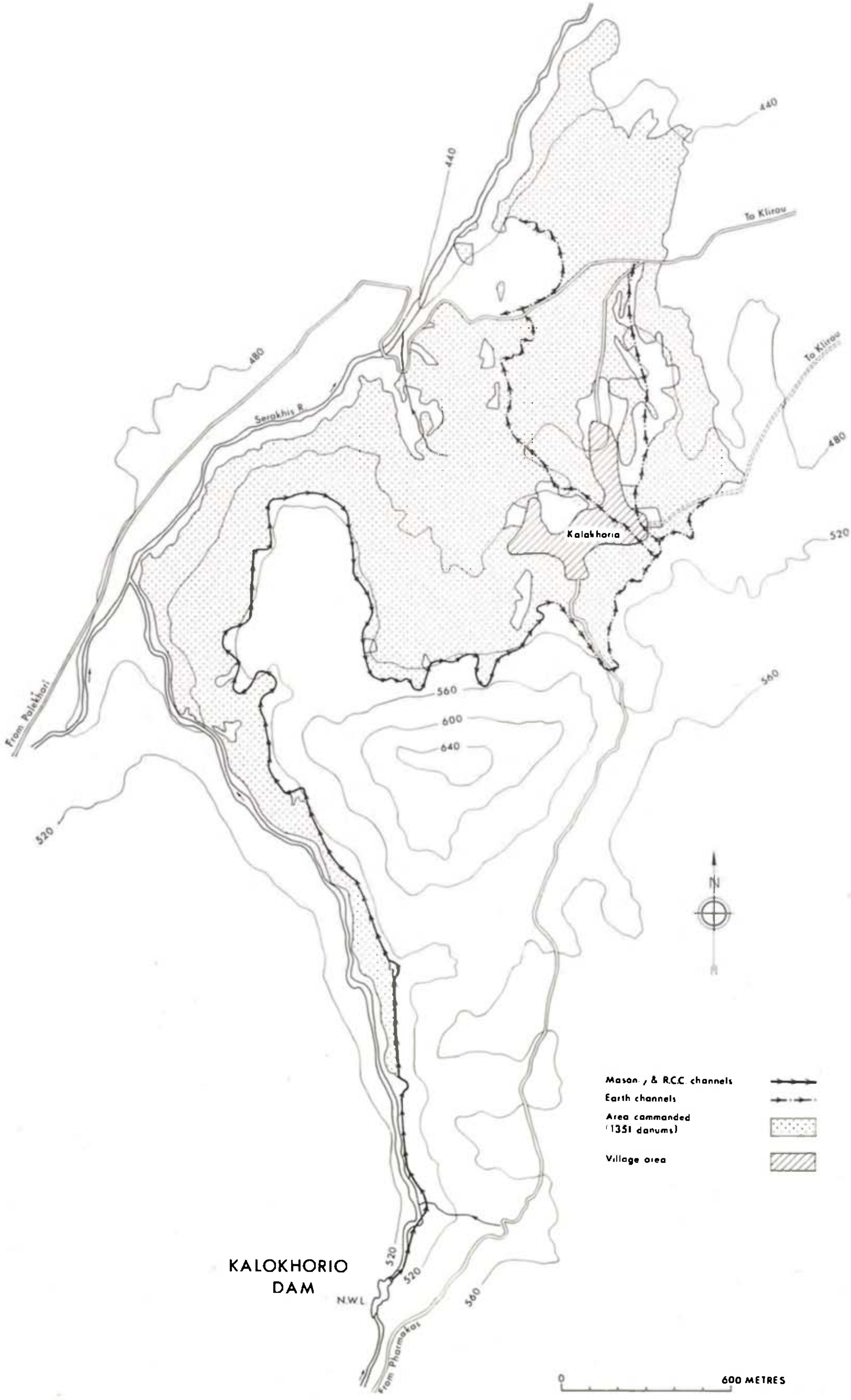
(iii) Concrete rectangular channels

Size	0.52x0.37 m
Capacity	0.113 m³/s
Length	6000 m

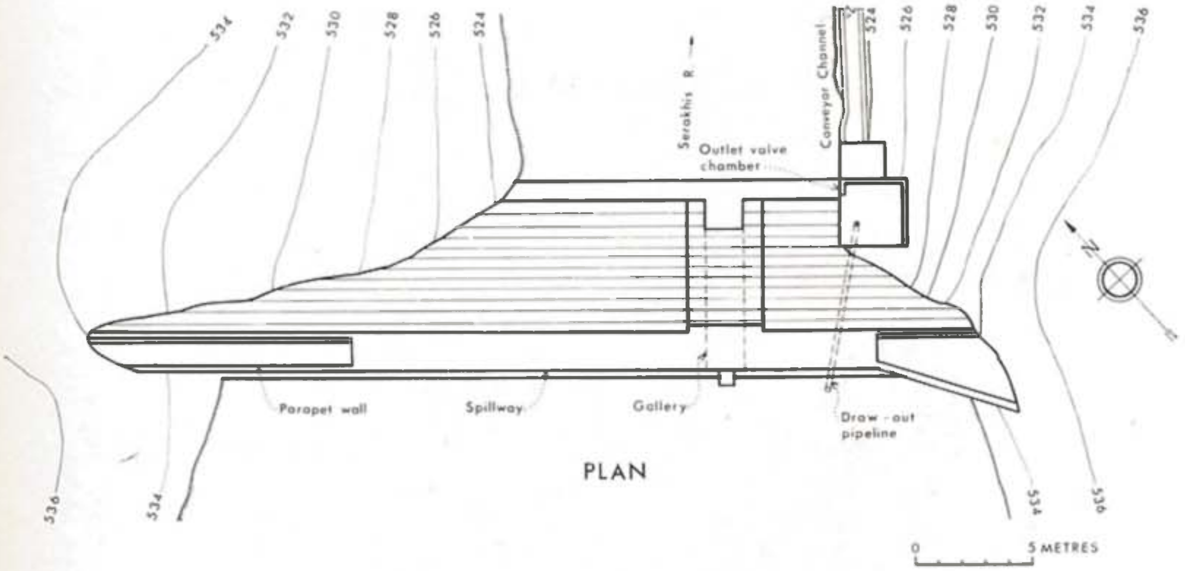
(iv) Earth Channels

Size	0.375x0.35 and 0.30x0.30 m 0.325x0.30 and 0.30x0.275 m
Capacity	0.028 m³/s
Length	6000 m

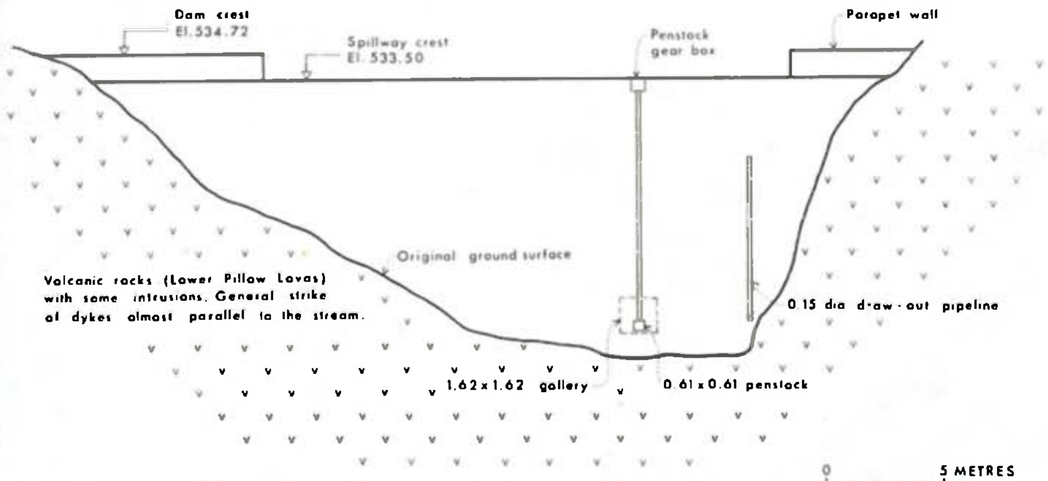
SERAKHIS-KALOKHORIO (KLIROU)-DISTRIBUTION SYSTEM



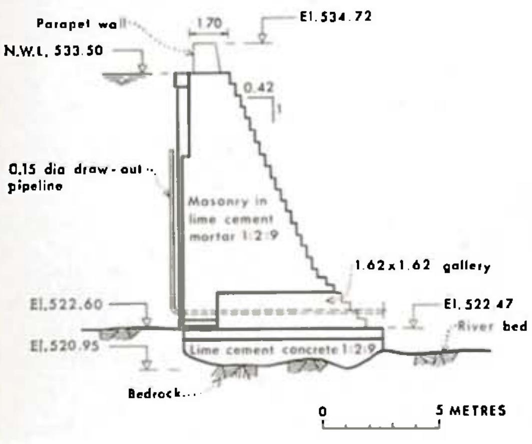
SERAKHIS-KALOKHORIO (KLIROU) DAM



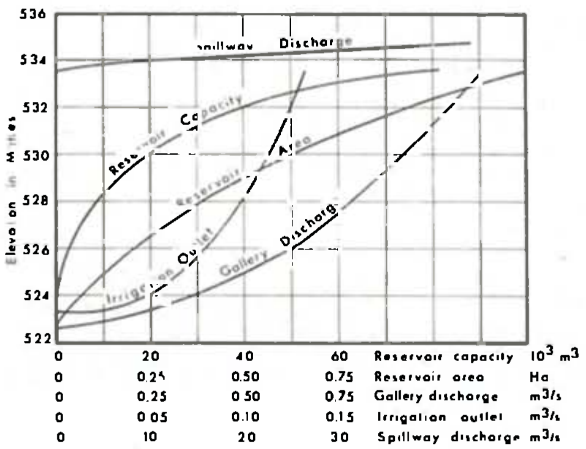
PLAN



SECTIONAL ELEVATION & GEOLOGICAL SECTION



MAXIMUM SECTION



CAPACITY-AREA-DISCHARGE CURVES

KAFIZES

The Kafizes dam.

XEROS — KAFIZES PROJECT

1. PURPOSE

This project was carried out in order to supply additional water to the orange groves of Lefka, which were previously relying on an intake and canalization system from the Marathasa river. The water rights of Lefka from the Marathasa river were not enough to satisfy the 560 donums of orange groves which at the time, forty years ago, represented the second largest orange plantations in Cyprus after Famagusta. As no more summer water could be allocated to Lefka from the Marathasa river due to the heavy requirements of other villages upstream, it was considered necessary to bring water from another river. The nearby river of Xeros on the West of Lefka was selected and a diversion scheme was designed in 1925 and constructed in 1932. This was then considered the most important irrigation work carried out on the Island except for the Eastern Mesaoria Irrigation Works made up of the Kouklia, Akhyritou and Syngrasi reservoirs. Later on, as the summer flow diversion was not found to be reliable and adequate, the storage dam at Kafizes was built which supplies water through the pipeline laid in 1932.

2. LOCATION

The dam is built on the Xeros river in the Nicosia District. It is situated at a distance of 11 km from the sea and at an elevation of about 260 m asl.

3. PLAN

a. Water and Land

There was no automatic water level recorder on this river and the runoff estimates giving an average annual flow of 8.1 million m^3 were based on rainfall data.

The Lefka people had no previous intake or rights on the use of the Xeros river prior to the diversion works of 1932. The Lefka gardens had to rely for their water supply on the Marathasa river. Apart from the 570 donums planted with citrus the irrigation system from Lefka Marathasa Dam and subsequently that from Xeros enabled a further 1500 donums of land to be brought under irrigation. Most of it is citrus groves in the Marathasa river valley. The only other interesting crop is the palm tree which can be seen in many parts of Lefka amongst the orange groves and projecting above them reminiscent of old Turkish times. In no other part of Cyprus are so many palm trees to be seen.

b. Geology

The dam is built on the Diabase group. At the damside the lithology is uniform. Sheeted diabase covers the whole area. It is relatively fresh, greenish-grey, fine grained, fractured and joined. The sheets vary in thickness from 0.3 to 1.8 m. The general trend of the sheets is 19° NE dipping 68° E. Two main sets of fractures were found with a trend 100° ES and NS dipping at an average angle of 80° N and 10° W respectively.

The river bed itself is free of gravels and compact diabase rock is exposed on the surface.

4. MAIN FEATURES

The dam and distribution system were designed by the WDD, except for the main conveyor from the dam which was designed in 1931 by Consultants Beeby Thompson of London.

a. Dam

The dam is of gravity design built in masonry.

The upstream face is vertical whilst the dowstream side is of a stepped design with a slope of 1:0.67.

A drainage system to relieve hydrostatic pressure is provided both in the dambody and foundations made up of horizontal and vertical perforated pipes conveying drainage water into the gallery.

The gallery is 1.22x1.22 m passing through the dambody, the main function of which is desilting and conveyance of the drainage water.

There is a penstock 0.25 m dia fitted at the upstream side of the gallery operated by a gearbox through a spindle rod. There is also a 0.15 m dia outlet pipeline which conveys the water from the dam into the irrigation pipeline to Lefka.

b. Distribution System

The steel conveyor pipeline designed in 1931 is of 0.20 m dia and is 9,600 m long. Along a 650 m length of the route a tunnel 0.75x1.25 m with shafts at intervals was provided to house the pipeline which discharges into the Mavrovouni balancing tank.

The main conveyor from the Mavrovouni tank to the gardens, is of masonry lining rectangular section. The main distribution system was designed when the Marathasa dam was built and is covered under the relevant chapter.

5. CONSTRUCTION

Both the dam and distribution system were constructed by the WDD.

The work on the dam started in March 1951 and was completed in January 1952. The foundation cut-off reached up to 6 m depth to the sound bed rock and was filled in with concrete.

The foundations are in cement concrete 1:3:6. The lower 4 m part of the dambody is of masonry set in line-cement-mortar of 1:2:9, whilst the upper part is set in 1:3:12 lime-cement-mortar. The facing upstream and downstream is of rubble masonry set in cement mortar 1:3. The upstream face is plastered in three coats of cement mortar to avoid seepage through the dam. The downstream face and wingwall masonry is pointed in cement mortar.

Gravel for the concrete was obtained from the river bed nearby whilst the sand was transported from the sea 11 km downstream.

Diabase rock for the masonry was excavated from a nearby quarry. The total concrete fill for the foundations was 950 m^3 and the total masonry fill for the dambody was 1,900 m^3. The total cost of the dam was £17,000.

The conveyor pipeline from the dam to the Mavrovouni masonry tank at Lefka was started in June 1931 and completed in May 1932. For 650 m of its length the pipeline was placed through a tunnel excavated through rock.

The cost of this scheme including the tunnel was £10,600.

In 1951 the main channel from the Mavrovouni tank to the citrus gardens was built in masonry of 0.49x0.37 m. The cost of this scheme was £3,000.

In 1958 the part of the Kafizes pipeline laid in the tunnel corroded to a point beyond repair and as part of the tunnel collapsed, a scheme was prepared for a new tunnel and pipeline.

This scheme was made up of 470 m of tunnel lined in a precast concrete oval section 0.51x1.07 m. 80 m of 0.2 m dia AC pipeline and 170 m of 0.25 m dia AC pipeline. This scheme started in March 1959 and was completed by the end of 1959 at a cost of £8,000.

Thus, the total cost of the distribution system until 1959 reached £21,600.

6. MANAGEMENT

The original 1931 Kafizes pipeline scheme was constructed as a Government Waterwork and the Municipality of Lefka undertook to pay off the sum of £10,600 over 25 years at a rate of interest of 2.5%.

As the beneficiaries would not pay their loan, finally the Government formed an Irrigation Co-operative Society in 1947 which took over the management of the scheme from the Municipality and undertook to pay the loan, Government having written off part of the loan.

For the 1951 Scheme including the dam and distribution system at £20,000 the Irrigation Co-operative Society undertook to pay a loan of £8,000.

At the time of building the dam, there were 82 beneficiaries owners of water from the Marathasa intake, their water ownership varying from 2 to 18 hours per month.

Finally, for the 1959 repairs costing £8,000, the Irrigation Co-operative Society undertook to pay the whole amount.

7. PROBLEMS OF SPECIAL INTEREST

What is of some interest regarding this dam is the measurements of the hydrostatic pressure on the gauges from the foundation drains. There are four such gauges which gave pressures as follows:

The 1st gauge at a distance of 13.3% from the upstream face gave a pressure of 87% of the total upstream hydrostatic pressure.

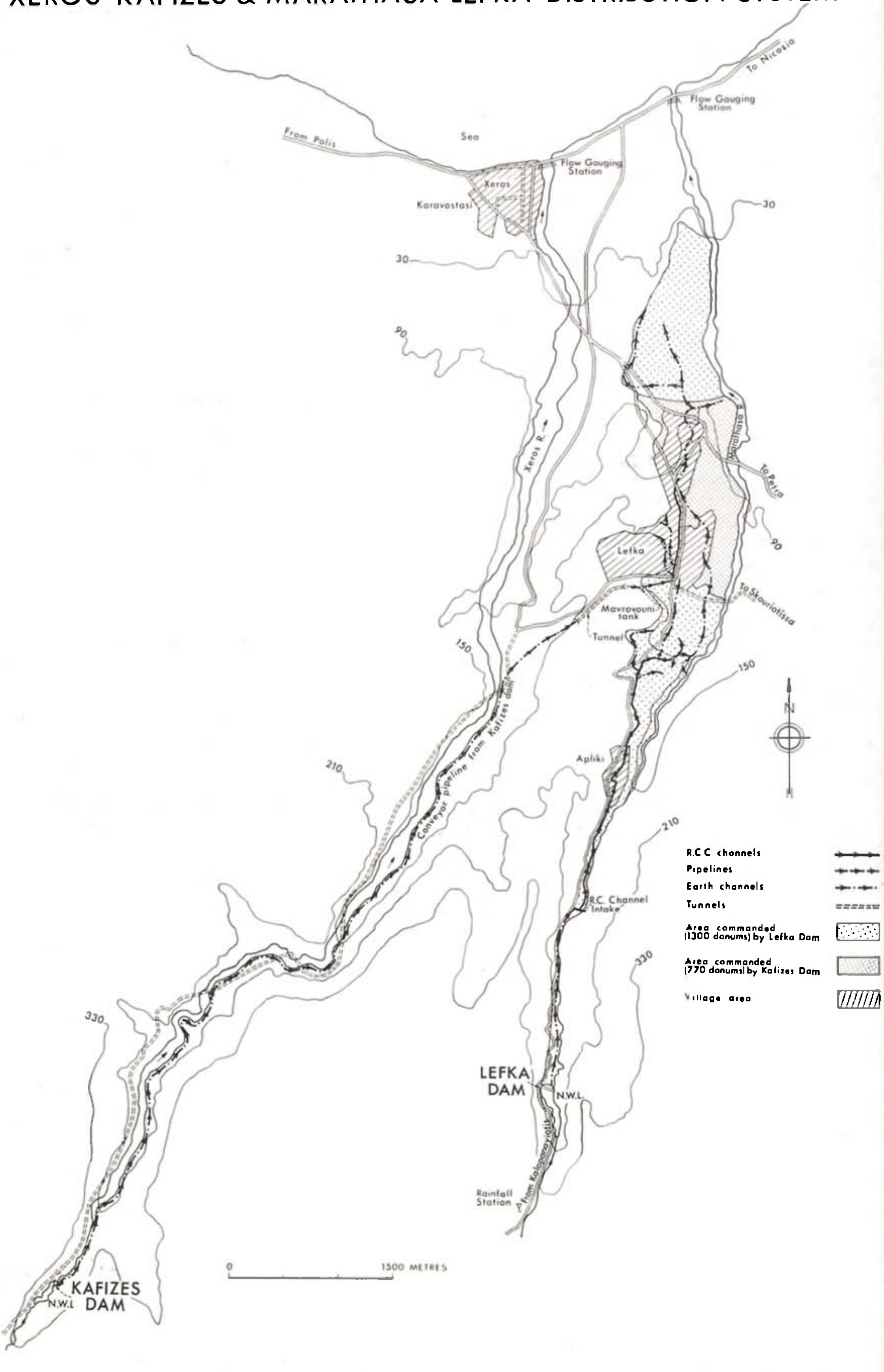
The 2nd gauge at a distance of 33.3% from the upstream face gave a pressure of 84% of the total upstream hydrostatic pressure.

The 3rd gauge at a distance of 40% from the upstream face gave a pressure of 83% of the total upstream hydrostatic pressure.

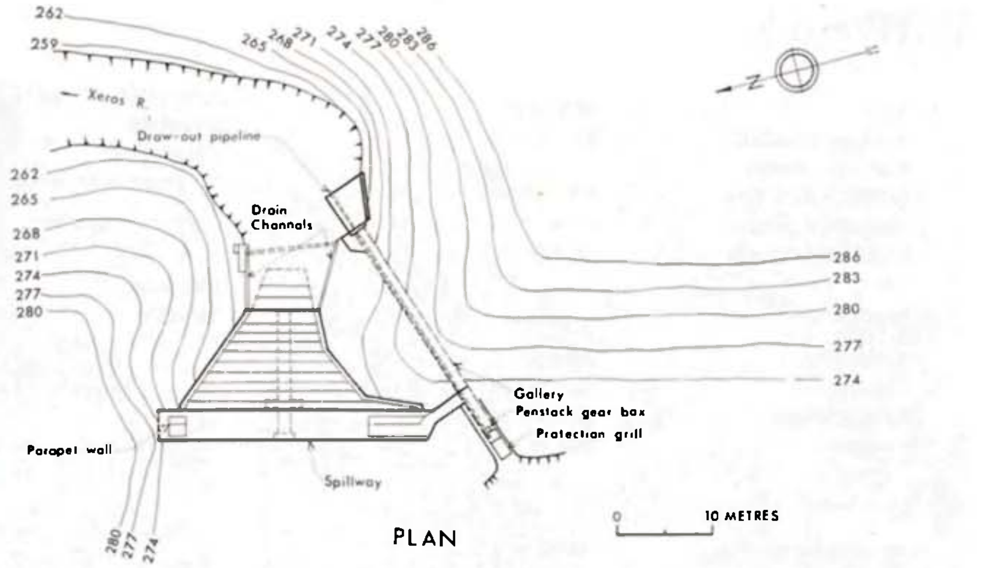
The 4th gauge at a distance of 52% from the upstream face gave a pressure of 79% of the total upstream hydrostatic pressure.

These readings indicate quite high pressures under the foundations and exceed the limits of a triangular pressure distribution at the base of maximum uplift at the upstream face to zero uplift at the toe.

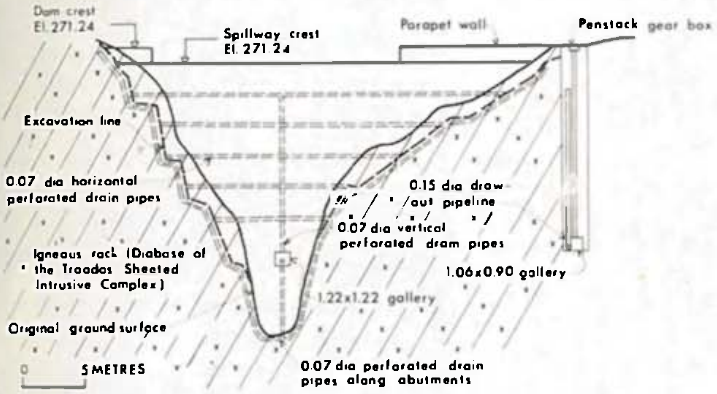
XEROS-KAFIZES & MARATHASA-LEFKA-DISTRIBUTION SYSTEM



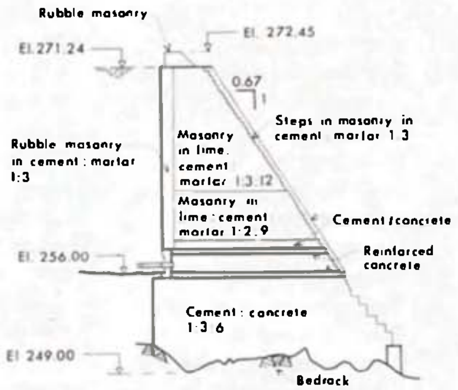
XEROS-KAFIZES DAM



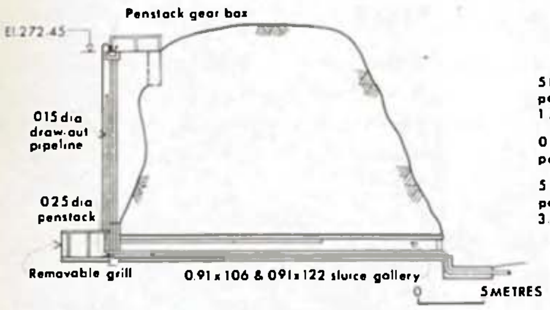
PLAN



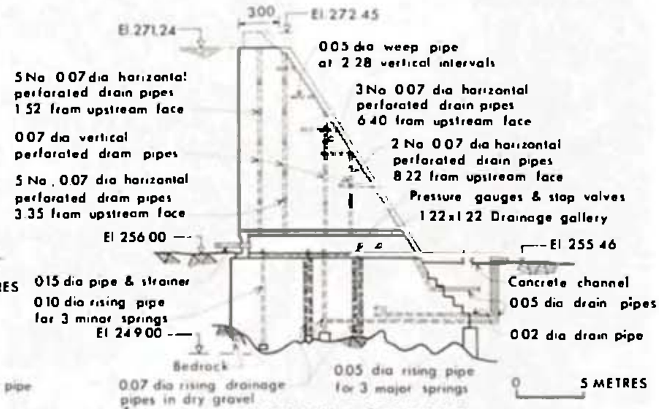
SECTIONAL ELEVATION & GEOLOGICAL SECTION



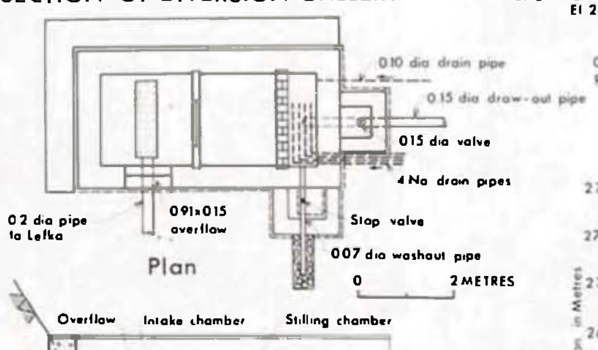
MAXIMUM SECTION



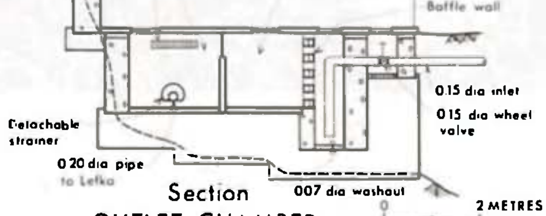
SECTION OF DIVERSION GALLERY



MAXIMUM SECTION
Drainage system

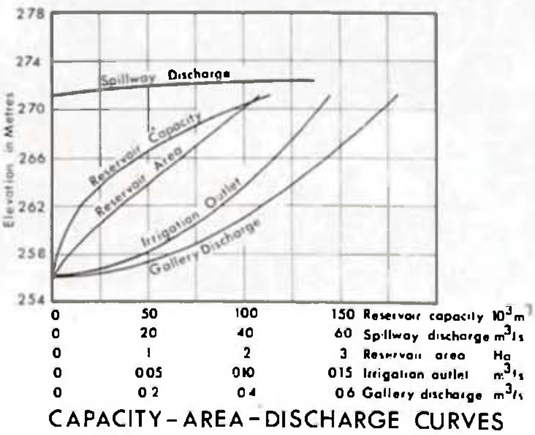


Plan



Section

OUTLET CHAMBER



CAPACITY-AREA-DISCHARGE CURVES

Data

i. Catchment

Area	38 5 km^2
Average rainfall	845 mm/a
Average runoff	8.1 million m^3/a
1/1000 years flood	340 m^3/s
Maximum height	1.240 m
Maximum length	11.4 km

ii. Reservoir

Area	2.0 ha
Capacity	113,000 m^3
Live storage	113,000 m^3
Length	540 m

iii. Dambody

Structural height	23.45 m
Height above ground level	16.45 m
Hydraulic height	15.24 m
Depth of foundation cut-off	7.00 m
Freeboard	1.22 m
Crest length	25.82 m
Top thickness	3.00 m
Base thickness	18.28 m
Upstream slope	Vertical
Downstream slope	1:0.67
Total volume of masonry fill of dambody	1,900 m^3
Total volume of concrete foundation fill	950 m^3

iv. Excavations

Total	2,000 m^3

v. Spillway

Size	18.59x1.22 m
Capacity	53 m^3/s

vi. Drainage Gallery

Size (Oval shape)	1.07x0.51 m
Capacity	—
Length	12 m
Strainer pipe inlet	0.15 m dia

vii. Diversion Gallery

Size	1.06x0.91 & 1.22x0.91 m
Capacity	14.8 m^3/s
Length	23.92 m
Operating gate	0.25 m dia

viii. Outlet (steel pipes)

Size	0.15 m dia
Capacity	0.14 m^3/s
Length	30 m

ix. Distribution System

(i) Main conveyor from the Dam to Lefka (Mavrovouni) tank (steel pipes)

Size	0.20 m dia
Capacity	0.05 m^3/s
Length	8,880 m

(ii) Tunnel lined in precast concrete oval segments

Size	10.07x0.51 m
Length	470 m

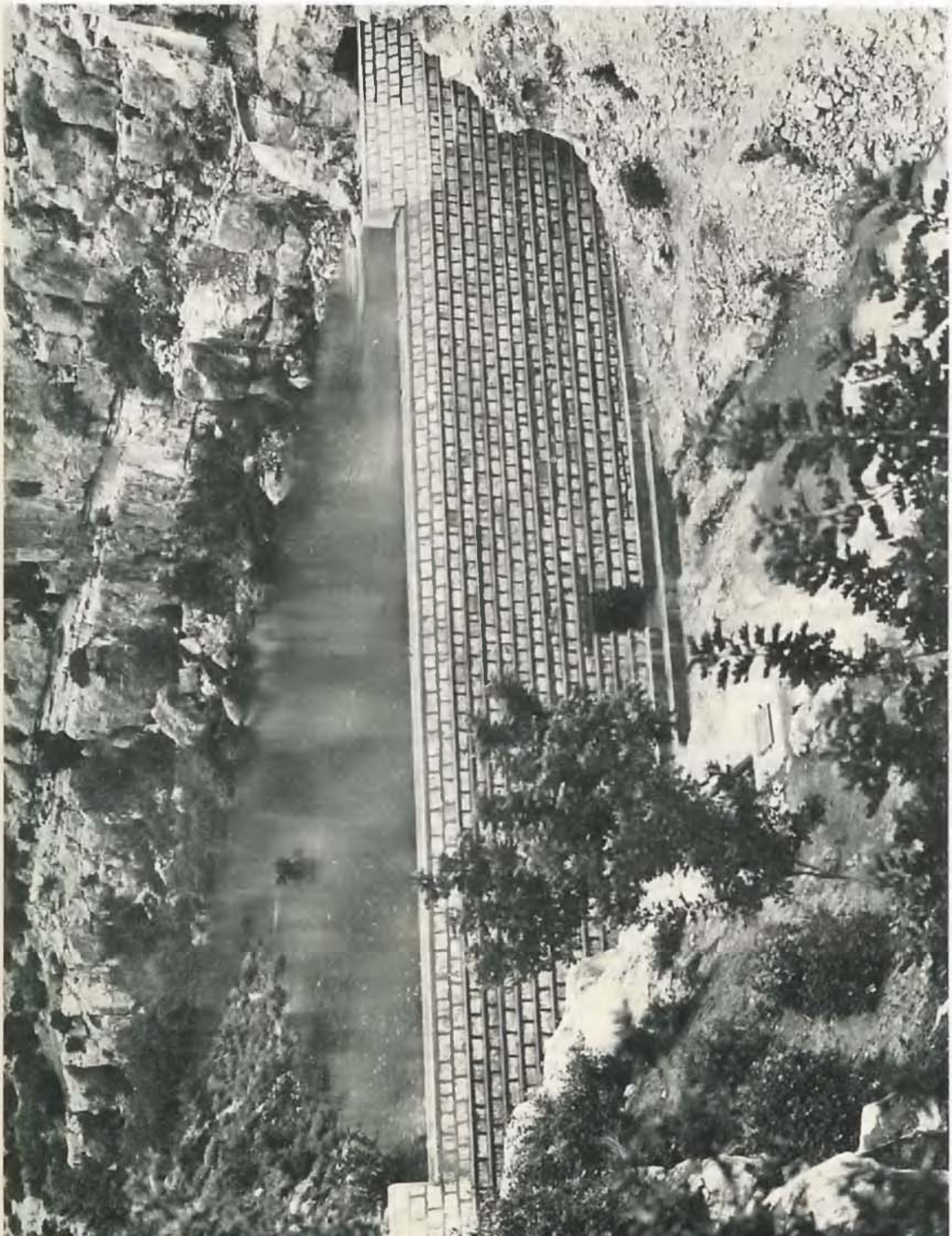
(iii) Asbestos cement pressure pipes

Size	0.20 m dia
Length	80 m
Size	0.25 m dia
Length	170 m

(iv) Masonry Main Conveyor from Mavrovouni Tank

Size	0.49x0.37 m
Capacity	0.05 m^3/s
Length	1,700 m

KANDOU



The Kandou dam.

KOURIS—KANDOU PROJECT

1. PURPOSE

This scheme was authorized for construction in order to provide additional summer water to the Batsounis Irrigation Association of Kandou village in the Limassol District. This Association is made up of 81 owners of the water of the springs issuing from the Batsounis tributary of the river Kouris and the water ownership varies between 15 min to 14 hours of the whole flow for each beneficiary over a 10 day period. The total land ownership commanded by the Batsounis irrigation system was 563 donums and as the water available, especially summer flow was not enough, the Department of Water Development made up a scheme to divert the water into the neighbouring Tapakhna stream valley where suitable storage could be produced by building a dam. This would enable the storage of the surplus winter flow of the Batsounis stream as well as the flow of Tapakhna stream.

2. LOCATION

The dam was built on the Tapakhna tributary of the river Kouris at an elevation of about 120 m asl and at a distance of about 6.5 km from the sea.

3. PLAN

a. Water and Land

The Batsounis stream was yielding 0.35 million m^3 in an average year whereas the Tapakhna stream was yielding another 0.40 million m^3 in an average year. The average rainfall over the former catchment is 540 mm/a and over the latter 550 mm/a.

The spring flow of the Batsounis is privately owned by the Irrigation Association beneficiaries but the stream flow in excess of the springs as well as the Tapakhna flow are utilized by the Association by consent of the Government.

The land commanded is mostly planted with spring and summer vegetables. Some citrus are also planted.

b. Geology

Two main rocks were found in the area; the Kandou sandstone and the marly chalks and chalks.

The Kandou sandstone is an alternating sequence of yellow white calcarenites with thin bands of marly chalks, calcareous sandstones, siltstones and conglomerate grits. This sequence overlies conformably and probably transitionally the alternating layers of marly chalks and chalks.

At the damsite area the Kandou sequence is mainly composed of hard to very hard whitish medium grained fructured calcarenite, interbedded with some very thin marls and conglomerate grits of angular to subangular gravels up to 1 cm dia. The beds are almost horizontal having an average strike 328° NW and 4° E dip.

Two sets of fractures are predominant in the area. The first one with a trend 43° NE dipping at an average angle of 88° S and the second one with a trend of 139° ES 70° W.

The leakage problem was studied before construction but it was decided to carry out grouting after construction if it would be required. The dam was also tested for leakage at a height of 5 m from the foundations during construction.

4. MAIN FEATURES

The dam and distribution system were designed by the WDD.

a. Dam

This is a masonry gravity dam.

The upstream face is vertical and the downstream has a slope of 1:0.70.

The downstream face is made up of steps along the overflow section up to river bed reducing the velocity of flow thus avoiding the construction of a stilling basin. The stepped design facilitates the placing of shuttering during concrete casting.

In the design, full uplift pressure was taken for the upstream face.

A system of drains has been incorporated in the dam and foundation to relieve hydrostatic pressure. This system is made up of 0.075 m dia perforated pipes laid horizontally at 2.74 m vertical intervals leading the water into a vertical drain which in turn discharges into the gallery. Also drains laid in dry gravel along the abutment and foundation carry the leakage out of the dam.

The gallery is provided through the dambody and its purpose was to divert the small river flow during construction. After construction its purpose is to scour the dam reservoir from the silt and to convey the drainage flow.

The penstock for the operation of the gallery is a vertical sliding gate 0.61x0.61 m operated through a spindle rod by a gearbox.

The outlet for irrigation is through a vertical perforated 0.15 m dia draw-off steel pipe which is connected to the conveyor channel.

b. Distribution System

A diversion weir on the Batsounis stream was made of rubble masonry in lime cement concrete. A conveyor channel was constructed from the Batsounis diversion weir to the dam.

The main conveyor starts from the dam outlet on the Tapakhna stream and reaches the irrigable lands near Kandou village.

The distribution network is made of masonry channels of rubble set in lime cement mortar to form a trapezoidal section.

5. CONSTRUCTION

The dam and distribution system were constructed by the WDD.

The foundation was built with a concrete mix of 1:3:6 cement sand gravel. The upstream face for 0.6 m width was made of rubble masonry set in cement mortar 1:2.5 cement sand and the central core was of rubble masonry set in lime cement mortar 1:2:9 lime cement sand. The upstream face was plastered in three coats of cement mortar to prevent seepage through the dam whereas the downstream face and wingwalls were pointed in cement mortar.

Sand and gravel for the concrete was transported from the sea 7 km away. The rock used for the masonry was calcarenite obtained from a nearby quarry.

The construction was done in two stages. The first stage, up to a height of 5 m was considered as testing height for leakage and the second stage was later on completed after tests were carried out which showed insignificant leakages.

The first stage started in November 1952 and was completed in December 1953. The second stage started in November 1954 and completed in December 1955. After completion of the dam and filling with water early in 1956, it was observed that several leakages appeared downstream through the right abutment which must have found their way through solution channels from this abutment. The total leakage from eight points varied according to the reservoir head, and at maximum level it was found to be about 5 l/s. It was therefore decided to carry out grouting works on the right abutment which was started in June 1959 and completed in October 1959, at a cost of £900. Cement grouting was injected through drilled holes on the right abutment at pressures up to 3 atmospheres. After the grouting works, tests showed that the leakage was reduced from 5 l/s to 3 l/s at maximum reservoir head.

The total cost of the dam including the grouting works was £11,400.

The distribution system is made up of the Batsounis diversion weir, the diversion channel to the dam, the main conveyor channel to the lands at Kandou and the distribution channels within the lands commanded. The scheme was carried out between 1953 and 1955 at a total cost of £6,300.

The Irrigation Association undertook to pay through a loan some £4,000 out of the total cost of the project at £17,700.

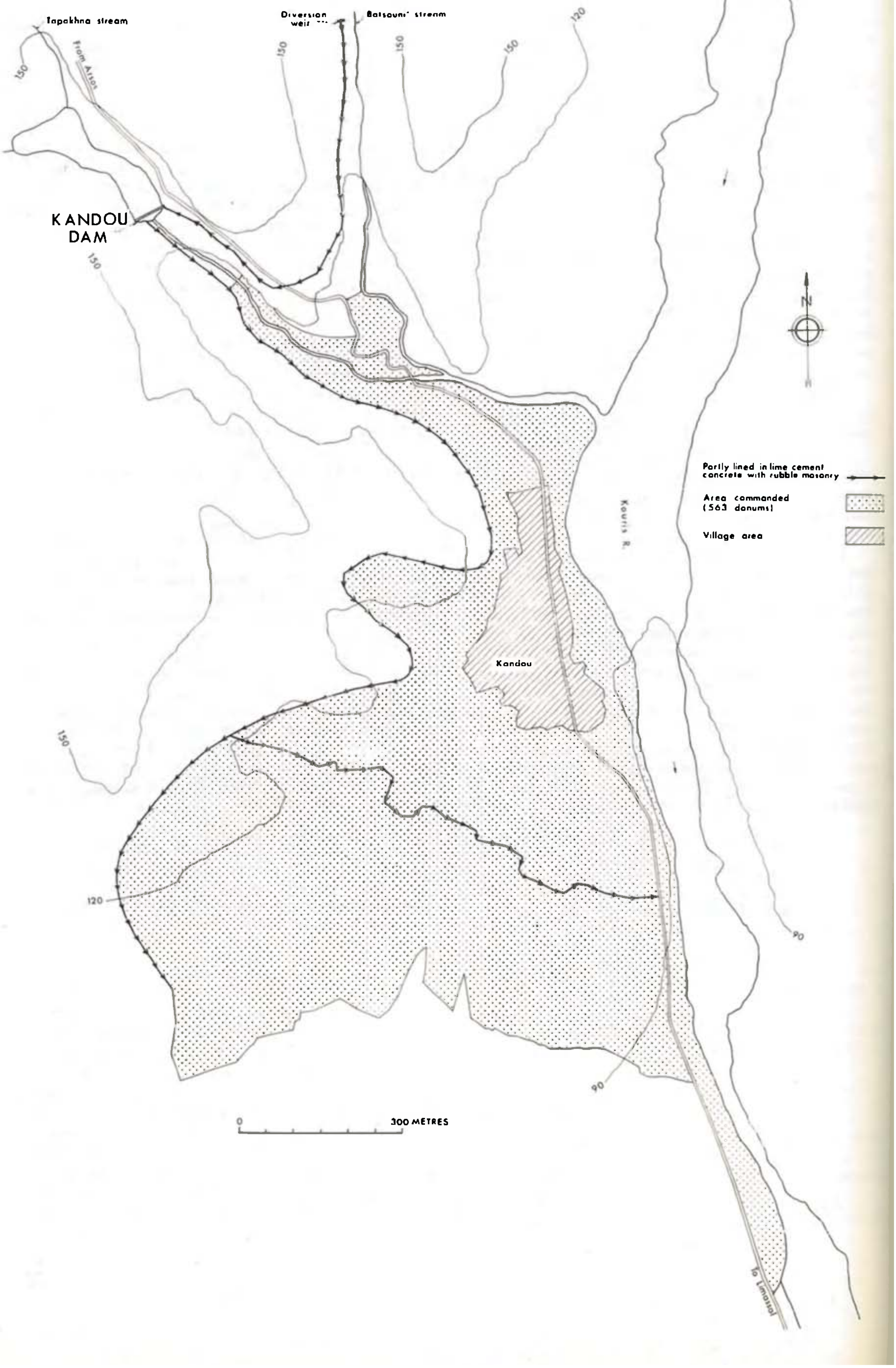
6. MANAGEMENT

This project is managed by the Kandou Irrigation Association made up of 81 owners of water. The irrigation of 420 donums of vegetables and some citrus is practiced from this project.

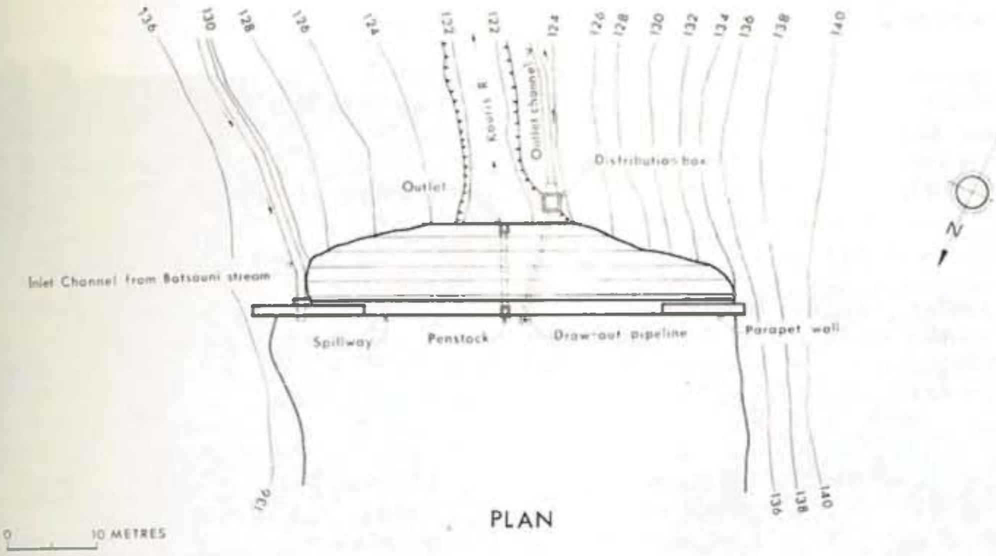
7. PROBLEMS OF SPECIAL INTEREST

The main problem of this dam was the leakage which occurred after impoundment. Although the first experimental stage at 5 m did not show any appreciable leakage, yet the final stage at 12.2 m head showed a total leakage from the nine points of about 5 l/s coming through the solution channels of the calcareous sandstones and limestones of the right abutment. The grouting which was subsequently carried out resulted in the reduction of the leakage from 5 to 3 l/s. This reduction resulted from seven out of the eight points of leakage whereas the ninth leakage point which was the largest remained unchanged. However, as the inflow in the reservoir compensates for the losses and as it was established that these losses did not affect the supply for the irrigation demand, it was not considered necessary to go ahead with further grouting works.

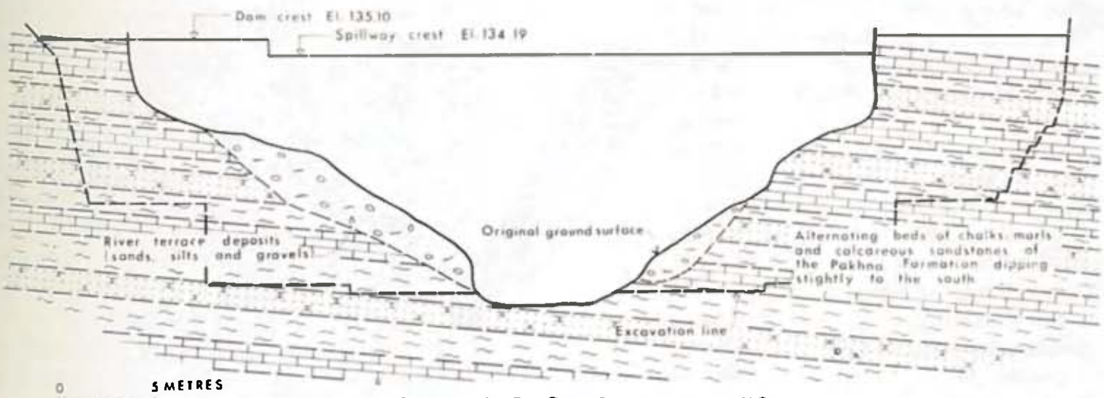
KOURIS-KANDOU-DISTRIBUTION SYSTEM



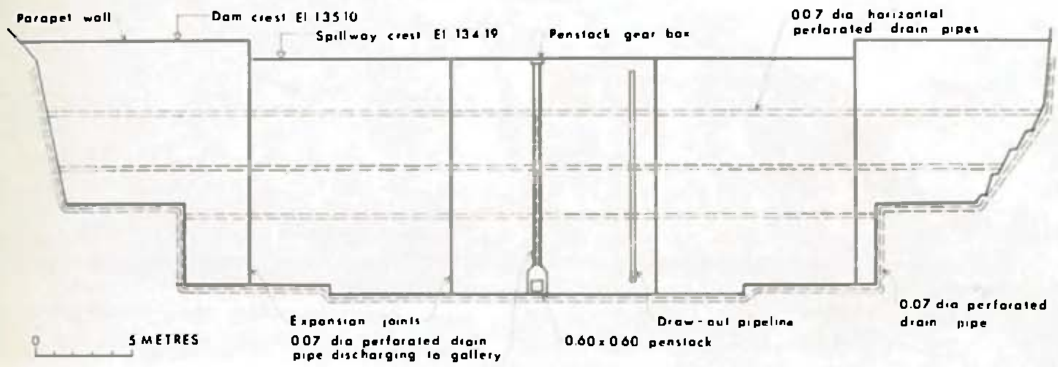
KOURIS-KANDOU DAM



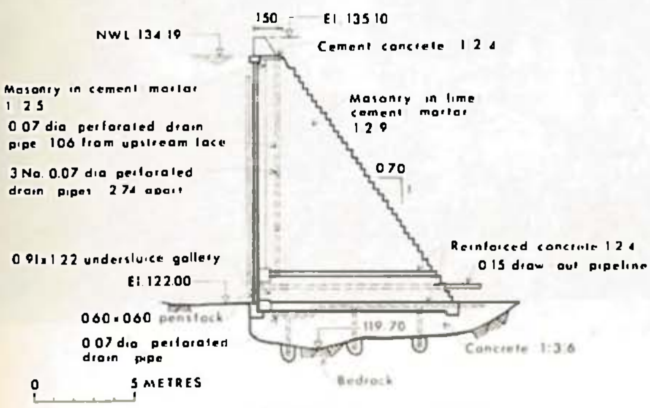
PLAN



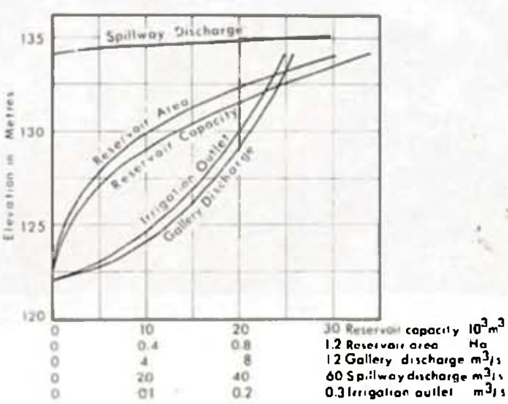
GEOLOGICAL SECTION ON DAM AXIS



SECTIONAL ELEVATION



MAXIMUM SECTION



CAPACITY-AREA-DISCHARGE CURVES

Data

i. Catchment above Dam — Tapakhna stream	
Area	7.5 km²
Average rainfall	550 mm/a
Average runoff	0.4 million m³/a
1/1000 years flood	50 m³/s
Maximum height	470 m
Maximum length	5 km
(i) Catchment above Batsounis Diversion Weir	
Area	6.4 km²
Average rainfall	540 mm/a
Average runoff	0.35 million m³/a
Maximum height	490 m
Maximum length	6.25 km
ii. Reservoir	
Area	1.2 ha
Capacity	33,900 m³
Live storage	33,900 m³
Length	140 m
iii. Dambody	
Structural height	15.40 m
Height above ground level	13.10 m
Hydraulic height	12.19 m
Depth of foundation cut-off	2.3 m
Freeboard	0.91 m
Crest length	52.5 m
Top thickness	1.50 m
Base thickness	9.66 m
Upstream slope	Vertical
Downstream slope	1:0.70
Total volume of masonry fill of dambody	2,600 m³
Total volume of concrete foundation fill	400 m³
iv. Excavations	
Total	450 m³
v. Spillway	
Size	31x0.91 m
Capacity	59 m³/s
vi. Gallery	
Size	1.21x0.91 m
Capacity	10.3 m³/s
Length	8.5 m
Operating gate size	0.61x0.61 m
vii. Outlet (steel pipes)	
Size	0.15 m dia
Capacity	0.25 m³/s
Length	20 m
viii. Batsounis Diversion Weir	
Excavations	25 m³
Rubble with lime cement concrete	50 m³
ix. Diversion Conveyor Canal from Batsounis Weir to Tapakhna dam	
Size	0.61x0.61 m
Capacity	0.11 m³/s
Length	900 m
x. Distribution System	
(i) Main conveyor	
Size	0.45x0.45 m
Capacity	0.055 m³/s
Length	2.400 m
(ii) Distribution network	
Size	0.45 + 0.22x0.22 m
Capacity	0.014 m³/s
Length	900 m

The Kandou masonry - concrete irrigation aqueduct.

PERAPEDHI

The Perapedhi dam.

KOURIS—PERAPEDHI PROJECT

1. PURPOSE

The main reason for constructing this project was to provide compensation water to the villages of Perapedhi and Kilani, for water pumped upstream from the Kryos tributary of the Kouris river to Troodos for domestic water supply requirements.

The Kryos tributary catchment at its high sources was the only possible source of domestic water to Troodos. The needs of Troodos were calculated to be of the order of 450 m^3/day during summer months. As there is plenty of surplus flow during other than summer months, it was decided to build a dam of a capacity 450 $m^3 \times 100$ days i.e. 45,000 m^3. With an allowance for evaporation, seepage losses and sedimentation, the dam was built of 54,550 m^3 capacity.

2. LOCATION

The dam lies on the Kryos tributary of the Kouris river about 2 km upstream of Perapedhi village and at an altitude of about 840 m asl.

3. PLAN

a. Water and Land

The average annual runoff based on rainfall data is estimated to be about 4 million m^3. The water use rights on the Kryos river are rather complicated and belong to Pano Platres, Perapedhi and Kilani villages on the basis of a distribution of hours during a week for each village. Before the construction of the dam Perapedhi had 75 donums of deciduous crops under irrigation and Kilani 35 donums using water from the Kryos river. The stored water in the dam aimed at satisfying at least the existing needs at the time.

The lands are mainly mountainous, but a good deal of them lie in a fairly flat area containing fertile soils.

b. Geology

The lithology of the dam site is uniform with the sheeted diabase underlying the whole area. Since the valley is very steep, talus and soil is very limited. The diabase is of uniform characteristics being hard dark grey well jointed. The sheets have a general trend of 30° NE and dip at an average angle of 70° SE. The thickness of the sheets is about 1—1.5 m.

The rock is rather fresh, nevertheless the first 3 m from the surface is weathered. Joints are open at the surface.

At the right abutment just upstream of the dam gabbro rocks are found intermixed with the diabase. In this area land sliding occurred after the first impoundment of water.

4. MAIN FEATURES

a. Dam

The design for this dam was carried out by the WDD.

The upstream face is vertical whilst the downstream face has a slope of 1:0.75. The upstream face was plastered in three coats of cement mortar to prevent seepage and the downstream face and wingwall masonry were pointed in cement mortar. The downstream sloping face is formed of steps from the crest to the river bed along the overflow section of the dam, for the purpose of breaking up the water pressure and thus avoiding the construction of a stilling basin.

This stepped design is also useful for making up the concrete lifts and for fixing the shuttering.

In the design full uplift pressure was taken for the upstream face.

A system of drains has also been provided to relieve hydrostatic pressure. Perforated pipes 7.5 cm dia along the abutment from the foundation bedrock laid horizontally and vertically in the dambody lead the water into a gallery 0.90x1.20 m. The foundation and abutment drain pipes are laid in dry gravel and extend along the abutments up to crest level. The gallery which was built in the dambody has the purpose of river flow diversion during construction, carrying away the drainage water, and for scouring the silt which would be deposited in the dam. The penstock for operating the gallery was originally 0.37 m dia and is operated from the crest manually. A new gate 0.61x0.61 m was placed in 1962 to enable better desilting of the reservoir.

b. Distribution System

The distribution system from the dam supplying water to the lands was designed by the WDD and is made up of a system of reinforced concrete cast-in-situ rectangular channels.

5. CONSTRUCTION

The dam was constructed by the WDD from May 1954 to December 1955.

Materials for the mass concrete were brought to the site from the Limassol coast about 60 km away as regards gravel and sand, and from near Saittas 10 km away as regards the stone work built in diabase rock.

The foundation up to river bed was filled with concrete mix 1:3:6 cement sand and gravel. A height of 7.5 m above riverbed was filled with rubble masonry in cement mortar. The remaining part up to the crest level was filled with rubble masonry in lime cement mortar, 1:3:9 lime cement sand. The total volume of concrete and masonry was 4,000 m³ and the total cost of the dam was £16,000.

The distribution system was also constructed by the WDD and was all in rectangular reinforced concrete cast-in-situ channels.

The total cost of the distribution system was £14,700.

6. MANAGEMENT

This project was constructed at full cost to the Government as a compensation to the Irrigation Divisions of Perapedhi and Kilani for water taken upstream for the domestic water supply of Troodos. The management was undertaken by the two Irrigation Divisions for the irrigation of 75 donums at Perapedhi and 120 donums at Kilani.

7. PROBLEM OF SPECIAL INTEREST

The landslide was a problem that developed at the right abutment on the diabase rock just upstream of the dam in January 1956. The area of movement covers about 80x30 m on the right abutment which has a slope of about 45°. A peripheral crack appeared starting from about 1 m higher than the crest of the dam embracing the slide area and reaching up to a height of about 30 m above reservoir water level and then dropping down to reservoir water level at the far end of the slide area. This peripheral crack has a maximum horizontal displacement of 0.3 m and its depth was about 3 m. Some minor cracks were also triggered below the major crack.

The landslide area is predominantly on diabase rock but some small portions of gabbro more or less parallel with the diabase jointing are also present. The face of the crack along which the landslide took place dips on the Southwestern face at a steep angle towards the displaced mass.

A number of settlement marks were established to observe later horizontal and vertical movements, but it became apparent that equilibrium had been reached and no more movement occurred ever since.

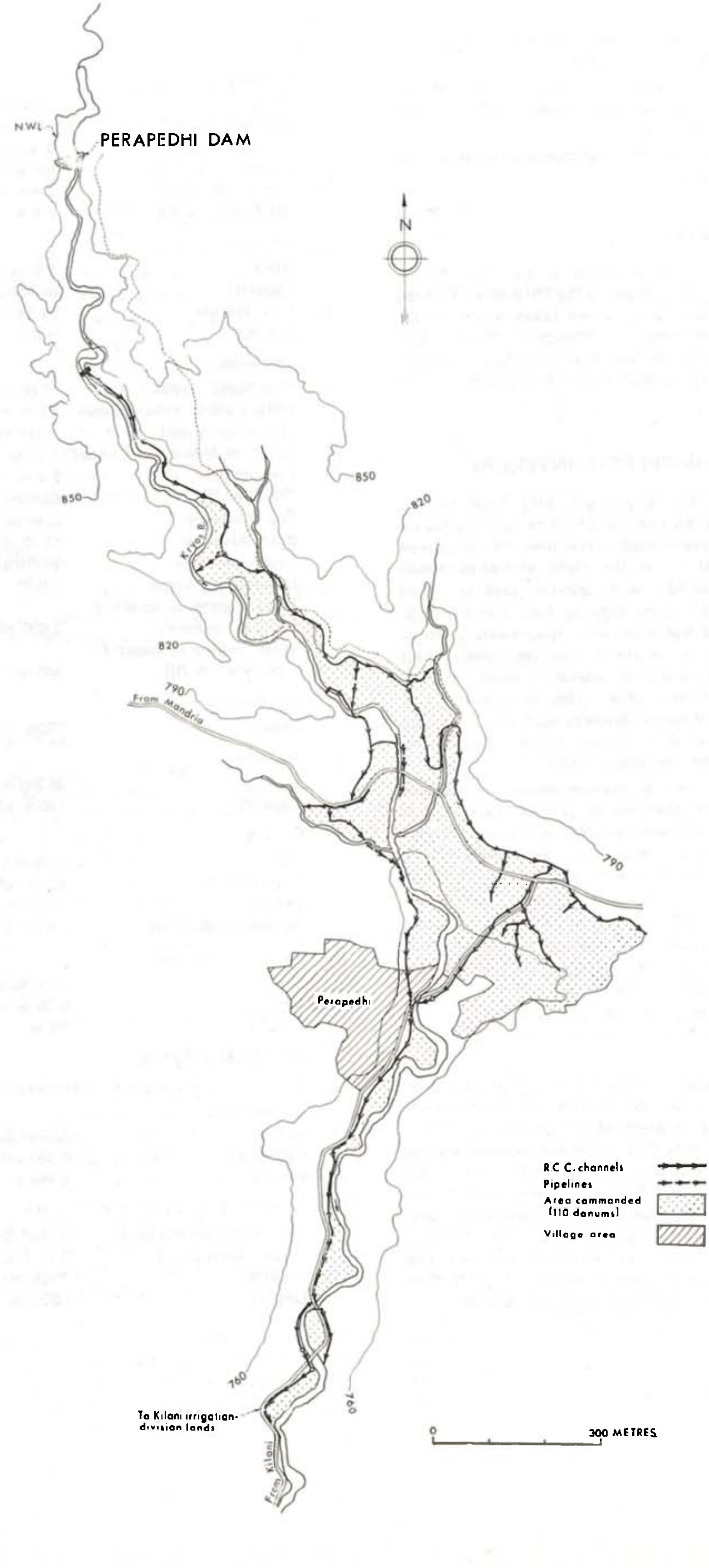
Another interesting problem after construction was the heavy sediment load and the work carried out for desilting.

The sediment load in 1959 reached 7 m height at the upstream face of the dam mainly due to the small size of the operating penstock of the gallery which was originally provided being 0.38 m dia and because during the winter months the dam was not left open. This silt was removed by agitating with water and it was emphasized that the dam should be left constantly open during the winter months for desilting. To enable a better desilting operation, the gate was changed and a larger one 0.61x0.61 m was installed. A protection grill was also fixed. The total cost was £1,200.

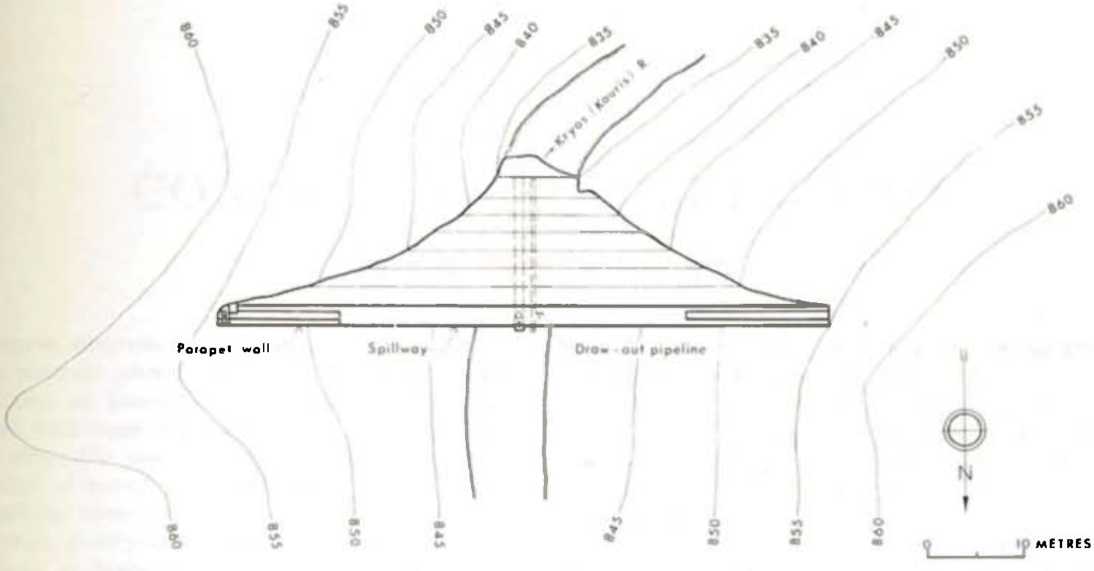
Data

i.	*Catchment*	
	Area	10 km²
	Average rainfall	950 mm/a
	Average runoff	4 million m³/a
	1/1000 years flood	60 m³/s
	Maximum height	1,880 m
	Maximum length	9 km
ii.	*Reservoir*	
	Area	1.2 ha
	Capacity	54,550 m³
	Live storage	54,550 m³
	Length	600 m
iii.	*Dambody*	
	Structural height	21.60 m
	Height above ground level	19 50 m
	Hydraulic height	18.30 m
	Depth of foundation cut-off	2.1 m
	Freeboard	1.2 m
	Crest length	65.5 m
	Top thickness	2.00 m
	Base thickness	15.25 m
	Upstream slope	Vertical
	Downstream slope	1:0.75
	Total volume of masonry fill of dambody	3,600 m³
	Total volume of concrete foundation fill	400 m³
iv.	*Excavations*	
	Total	750m ³
v.	*Spillway*	
	Size	36.5x1.2 m
	Capacity	105.6 m³/s
vi.	*Gallery*	
	Size	1.20x0.90 m
	Capacity	12.66 m³/s
	Length	15.25 m
	Operating gate size	0.61x0.61 m
vii.	*Outlet (Steel pipes)*	
	Size	0.15 m dia
	Capacity	0.20 m³/s
	Length	30 m
viii.	*Distribution System*	
(i)	*Main Conveyor (concrete rectangular channels)*	
	Size	0.33x0.23 m
	Capacity	0.056 m³/s
	Length	2,440 m
(ii)	*Distribution network*	
	Size (concrete channels) and (steel pipes)	0.25x0.20 m 0.10 and 0.15 m dia
	Capacity	0.028 m³/s
	Length	1,220 m

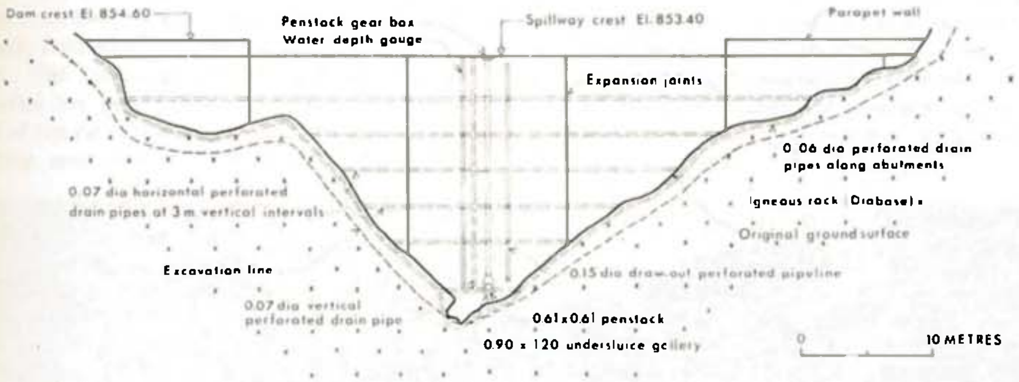
KOURIS-PERAPEDHI-DISTRIBUTION SYSTEM



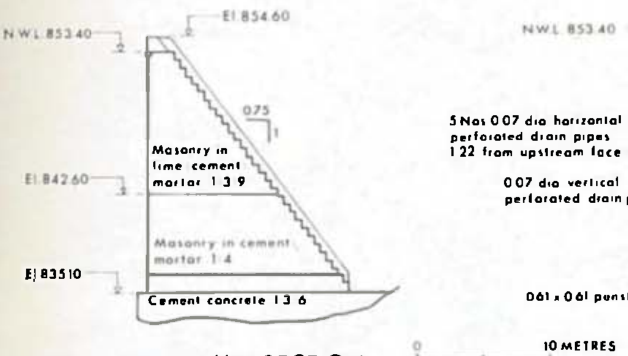
KOURIS-PERAPEDHI DAM



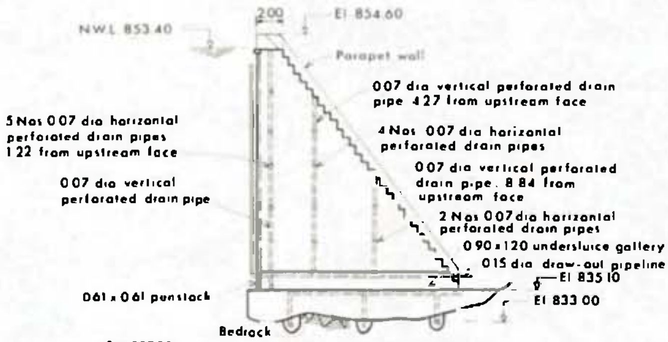
PLAN



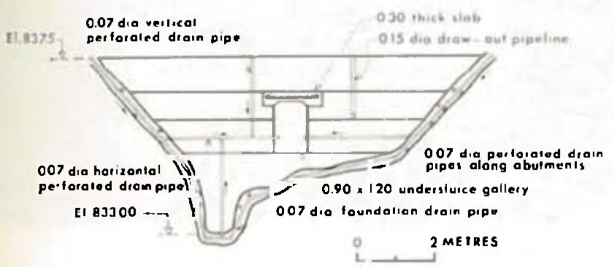
SECTIONAL ELEVATION & GEOLOGICAL SECTION



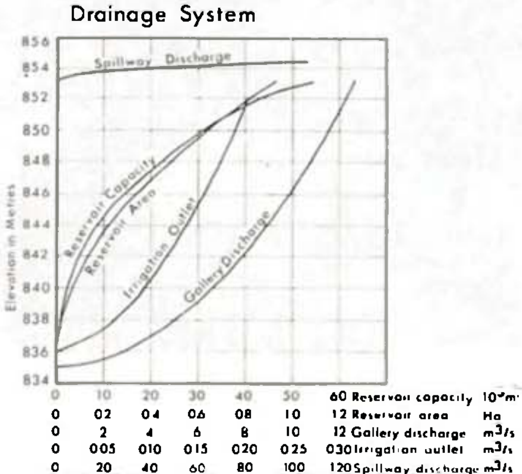
MAXIMUM SECTION



MAXIMUM SECTION
Drainage System



FOUNDATIONS



CAPACITY-AREA-DISCHARGE CURVES

CHAPTER III

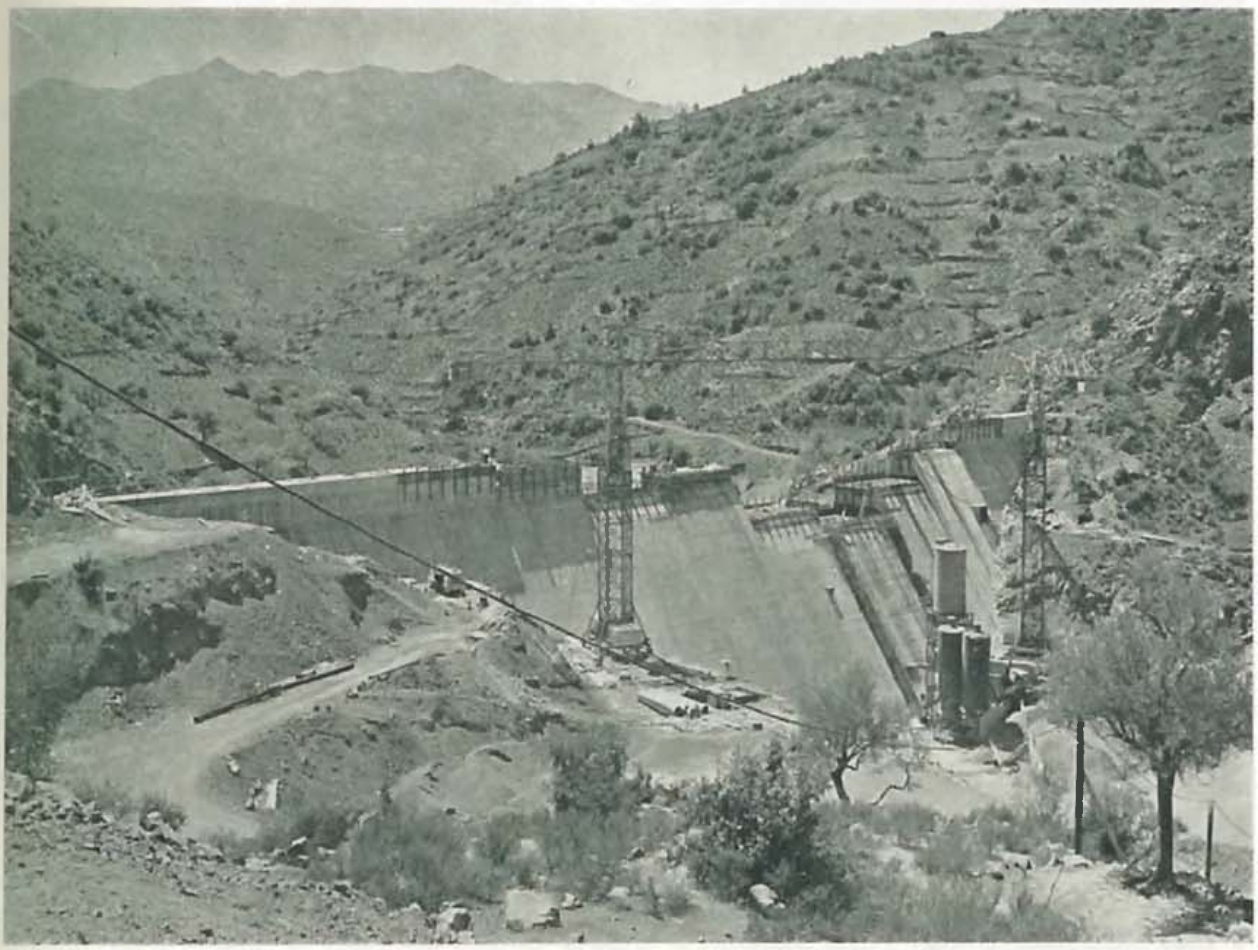
CONCRETE GRAVITY DAMS

INTRODUCTION

In Cyprus concrete dams have been built only of the mass concrete gravity type. There are few cases where it may be possible to build arch dams in the future and such cases will be for small dams on high altitudes where the possibility exists to find a narrow gorge valley of sound geological characteristics. One or two buttress dams may also be built in the future. Mass concrete gravity dams have replaced the masonry gravity dams as from 1958 when the Trimiklini dam was built. Both concrete and masonry dams have generally been built on the igneous mountainous regions of Cyprus. Sedimentary rocks such as our soft sandstone and chalk formations are not good enough for a gravity dam. The introduction of concrete as a building material for dam construction has followed a general trend of the replacement of masonry by concrete for all building construction in Cyprus as from the middle 1950's. This trend was a consequence of the increased labour costs, the production of cement locally and the boom in the building industry which necessitated quicker, larger and more versatile construction work in all fields of civil works. Four dams included on the ICOLD World Register of Dams are described in this chapter.

A comparison of the various properties of concrete and masonry are given in chapter II which shows the many advantages of concrete. The main advantage of masonry over that of concrete 20 years ago was that of cost, and as soon as this became negative, then the important properties of concrete weighed in favour of the replacement of masonry by concrete.

An analysis of costs of masonry and concrete for 1973 is given here as compared with that given in chapter II for 1953.



The Palekhori dam under construction.

Masonry cost/m³

	£ mils
Stone purchase cost 7.5 Nos.	1.500
Cost of dressing the stones	0.750
Rubble filling	1.000
Lime cement mortar	
Sea sand 0.35 m³	0.500
Cement 120 kg	0.850
Lime 60 kg	0.500
Building Labour	7.500
Miscellaneous	0.650
Plastering of masonry walls 1:3	
Quantities and cost per m²	
Sea sand 0.06 m³	0.050
Cement 27 kg	0.200
Labour	1.250
Miscellaneous	0.250
Total cost/m³	15.000

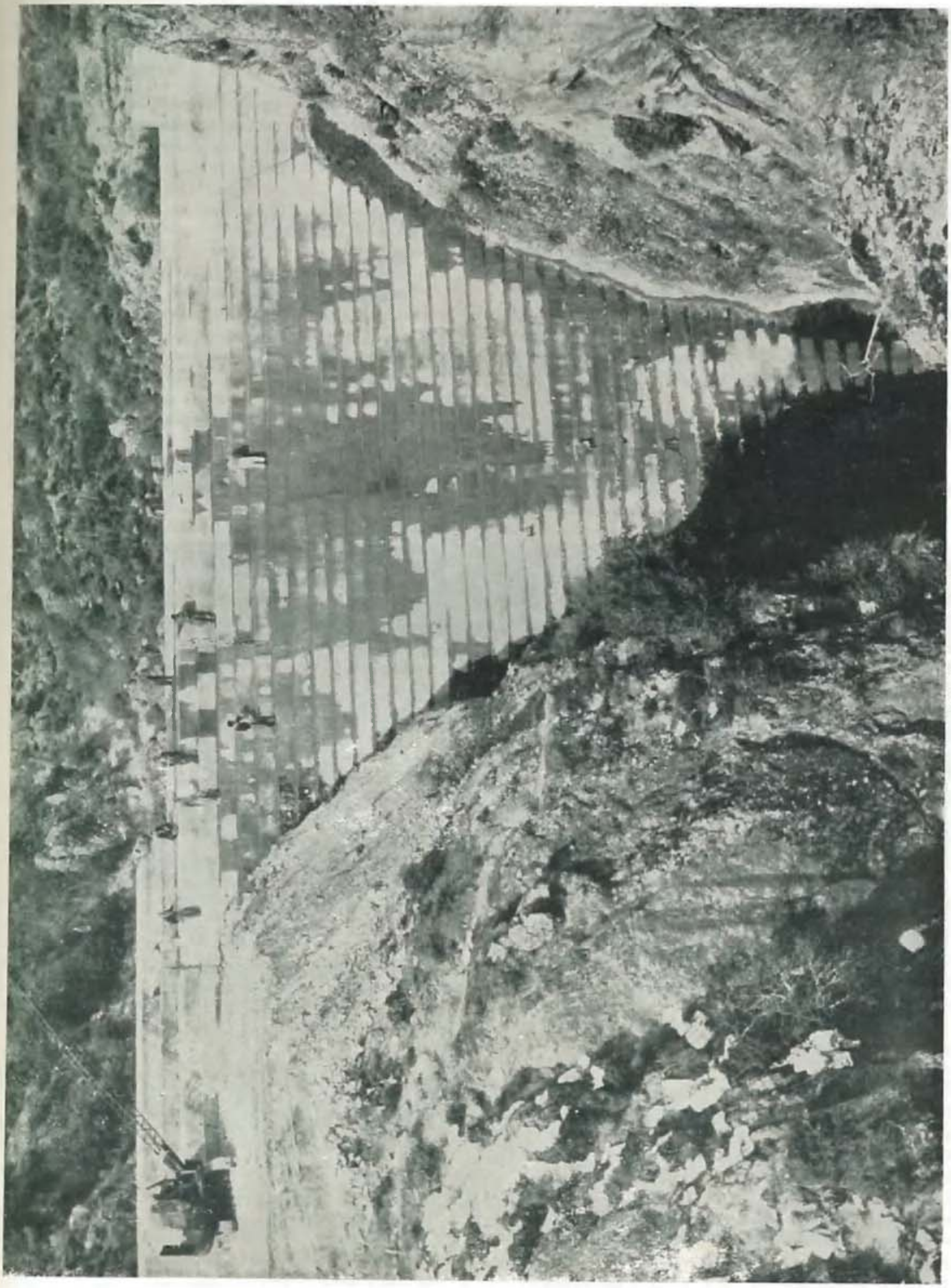
Concrete cost (1:2:4)

	£ mils
Sea Gravel 0.97 m³	1.250
Sea sand 0.50 m³	0.500
Cement	1.500
Labour	3.000
Miscellaneous including mixer, formwork etc.	2.000
Total Cost/m³	8.250

It can be seen from the above analysis that whereas in 1953 the cost of masonry was just about cheaper than that of concrete, in 1973, 20 years later, the costs were reversed to such an extent that the cost of masonry is nearly twice that of concrete. Therefore, the use of masonry has ever since been entirely discontinued from all water works.

Table 8 gives the main design characteristics of the four larger concrete gravity dams built.

TRIMIKLINI



The Trimiklini dam.

KOURIS—TRIMIKLINI PROJECT

1. PURPOSE

This Project was constructed in order to provide summer water to the Irrigation Division at Trimiklini for the purpose of growing deciduous plantations of apples, pears, peaches and plums. The village of Trimiklini is predominantly vine growing and the introduction of deciduous crops would add considerably to the benefit of the village. It is the general plan for the area to replace part of the vines through more remunerative crops such as deciduous trees. However, this conversion requires the supply of summer water which is not available and which can only be done through the construction of storage schemes.

2. LOCATION

The dam is situated on the Amiandos tributary of the Kouris river about 1.5 km upstream of the Trimiklini village and at an elevation of about 600 m asl.

3. PLAN

a. Water and Land

The average annual runoff yields about 19.4 million m^3. Water rights are claimed on this river from the various downstream proprietors who have title deeds on the Kouris river. Therefore a compensation pipeline to supply 0.18 m^3 of water from the upstream of the dam to the downstream side was placed through the reservoir and through the dam in order to satisfy these water rights.

The land commanded is 300 donums and most of it was quite steep so that extensive land levelling works had to be carried out for the plantations to be grown and for efficient irrigation to be practiced.

b. Geology

The dam is built on a massive fault breccia. This tectonic breccia is hard, polygenic and contains mostly lava and diabase fragments with some gabbro fragments as well.

Near the tail of the reservoir, is the contact between the tectonic breccia and the sheeted diabase. This diabase is greenish, irregularly fractured and altered. Two types of alteration were found. The first type is due to weathering and the second is due to late stage solutions along small fractures. The general trend of the diabase sheets is roughly NS dipping at an angle of 58° towards SE. No geological problems as regards the foundations and abutments were experienced, the dam site being a very narrow gorge. However, the reservoir area is steep and the storage capacity is relatively small in comparison with the height of the dam. The main geological problem is that of a heavy silt transportation in the reservoir which is largely attributed to the Asbestos Mines upstream of the reservoir from where heavy erosion of the dumps takes place under wet conditions.

4. MAIN FEATURES

a. Dam

The design of this dam was undertaken by the WDD. It is the first mass concrete gravity dam built in Cyprus. The upstream face is vertical whereas the downstream side is of a stepped design up to river bed level to reduce the velocity of the overflow water. Its slope is 1:0.61. A system of drains has been provided to relieve hydrostatic pressure, with perforated pipes 7.5 cm dia laid along the abutments from the foundation bedrock and horizontally and vertically in the dambody itself conveying the drainage water into the outlet gallery. The penstock for the desilting gate was originally 0.9x0.9 m and was operated from low level whereas the smaller penstock of 0.43 m dia could be operated from the top. This penstock was changed at a later time by another type of mechanism as explained later.

b. Distribution System

This is a system of cast-in-situ concrete channels and steel pipes designed by the WDD to supply water as far as every individual holding.

5. CONSTRUCTION

The dam was constructed by the WDD, work having started in June 1956 and completed in September 1958. The total volume of excavations was about 1,000 m^3 and that of concrete including the foundation was 6,115 m^3. Sea sand and gravel were transported from Limassol 50 km downstream. For the foundations, the upstream and downstream faces, a concrete mix of 1:6 was used, whereas 1:8 mix was used for the central core. The total cost of the dam including certain changes in the gates and subsequent repairs until 1970 was £38,700. Another £5,500 was spent on the compensation pipeline.

The main distribution network was constructed at the same time but later extensions were added in 1961, 1966 and 1970.

The total cost of the distribution system including later extensions was £20,300.

The total cost of both dam and distribution system was £64,500.

The Trimiklini Irrigation Division signed a loan to pay 0.33% of the total cost of both dam distribution system and compensation pipeline over a period of 20 years at a rate of interest of 6%.

6. MANAGEMENT

This project is operated by the Irrigation Division of Trimiklini made up of 154 members who own a total of 650 donums of land.

7. PROBLEMS OF SPECIAL INTEREST

The main problem regarding this dam is that of the heavy silt sediment which has its sources from the Asbestos Mines about 9 km upstream. The Mine has been in operation for over 50 years now and has been dumping its wastes along the river valleys thus becoming a potential source for silt erosion and transportation in the river downstream. The Mines operations result in some 2 million tons of waste every year which has to be dumped somewhere. It was first found by the Mines that the easiest dumping place was the river valley the water of which would gradually transport these materials downstream. However, in time, the interest for irrigation increased and downstream irrigators got concerned about the amount of silt, the fine particles of which resulted in an impermeable thin layer laid over the land when this water was used for irrigation. This silt clogs the pores of the soil and therefore the percolation of water to the root zone is prevented. With the erection of this dam, the problem of silting up the reservoir became very serious. During construction, when the dam had reached a level of about half its height (17 m), one flood was enough to silt up the reservoir completely. Underscouring was then applied through the desilting gallery and with the help of incoming water most of the silt was removed.

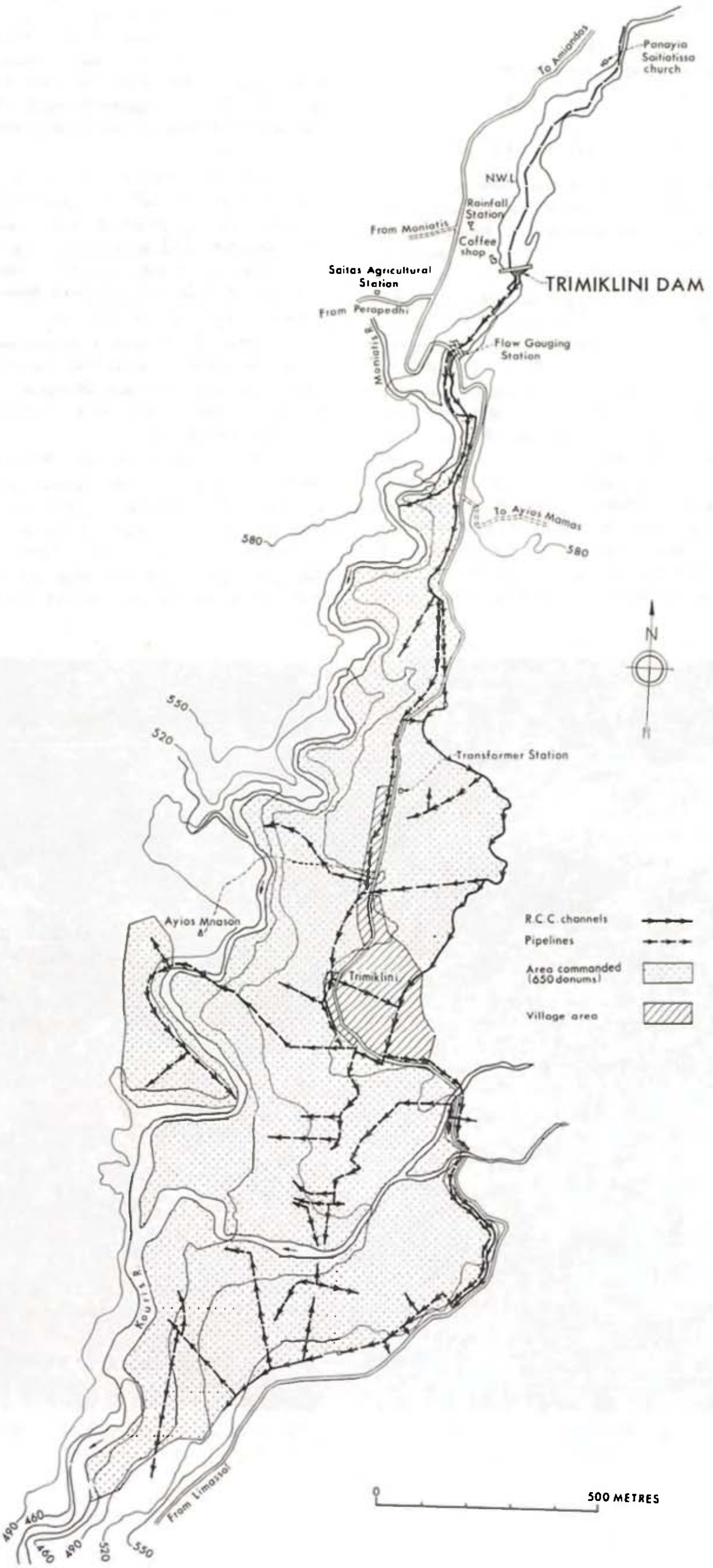
From later observations it became obvious that with normal rainfall, the accumulation of asbestos sediment in the reservoir was occurring every year. This sediment had to be scoured through the gallery. In October every year the scour gate of the dam had to be opened and left open until sometime in April when it had to be closed to store water.

In 1964—65 the Mines undertook some remedial works including measures to stabilize the old dumps and the creation of a new dumping place in such a way as to prevent them being eroded and reaching the river valley.

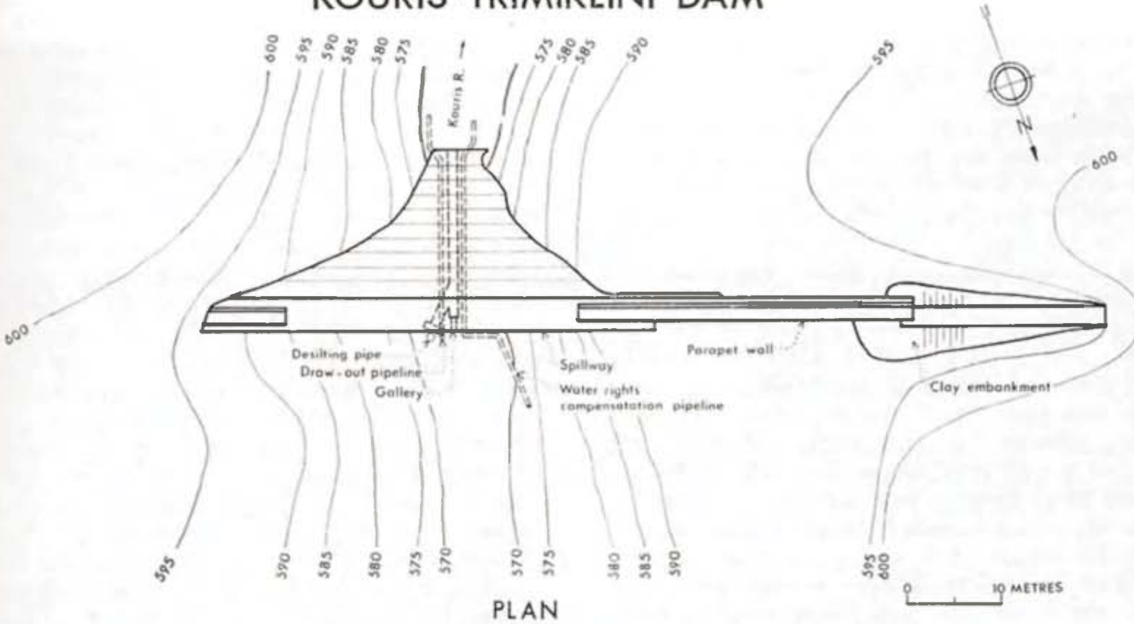
However, in spite of important improvement measures taken by the Mines, the worse situation developed in 1968—69. In October 1968 the pressure from the sediment caused a breakage to the operation mechanism of the gates. Immediately after, six attempts were made by skin divers who eventually managed to lift the gate by the use of a crane so that

Trimiklini dam in May 1969 showing silt deposited in the reservoir from erosion of the Asbestos Mines dumps upstream. The depth of the silt at the lowest point is 8 m.

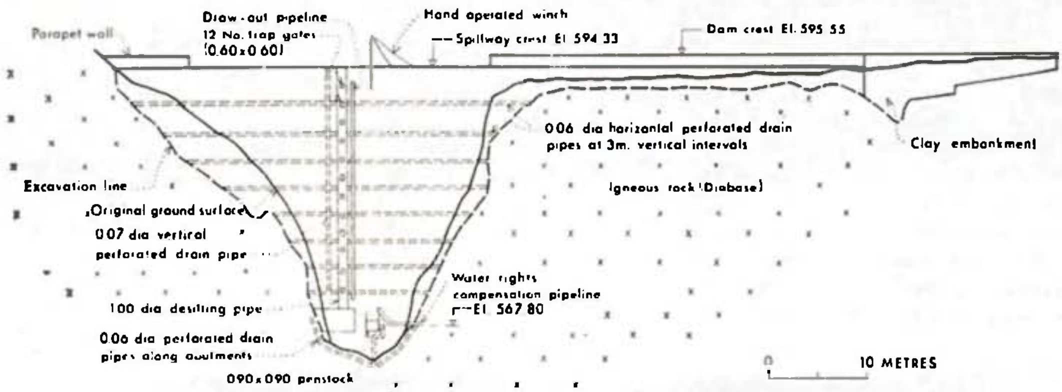
KOURIS - TRIMIKLINI - DISTRIBUTION SYSTEM



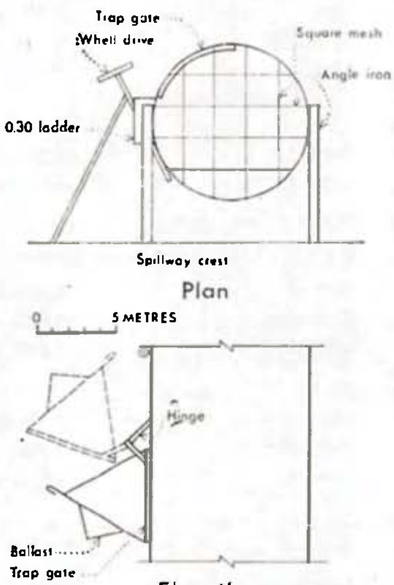
KOURIS-TRIMIKLINI DAM



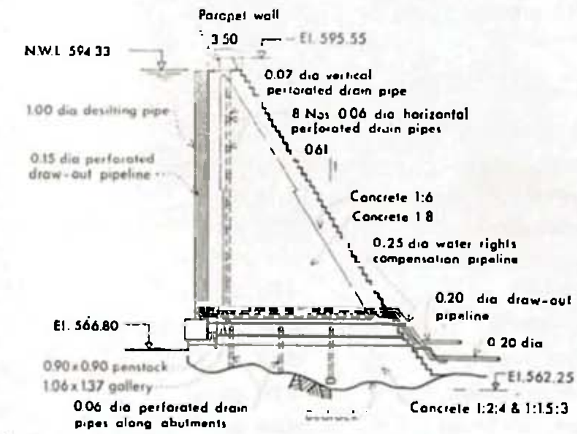
PLAN



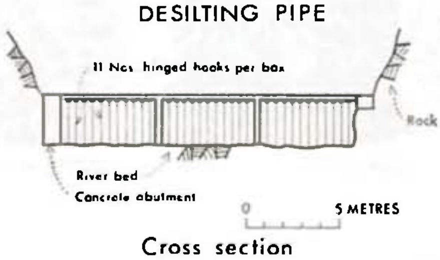
SECTIONAL ELEVATION & GEOLOGICAL SECTION



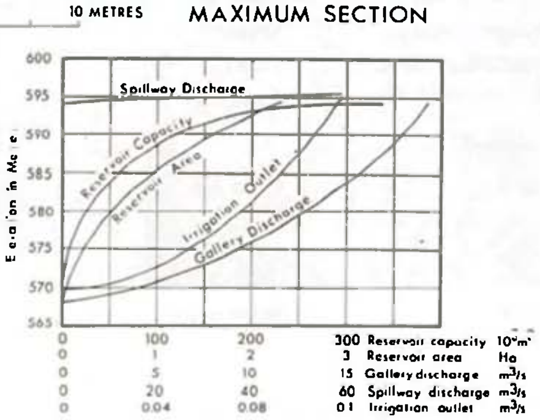
DESILTING PIPE



MAXIMUM SECTION



TRASH RACK UPSTREAM OF RESERVOIR



CAPACITY-AREA-DISCHARGE CURVES

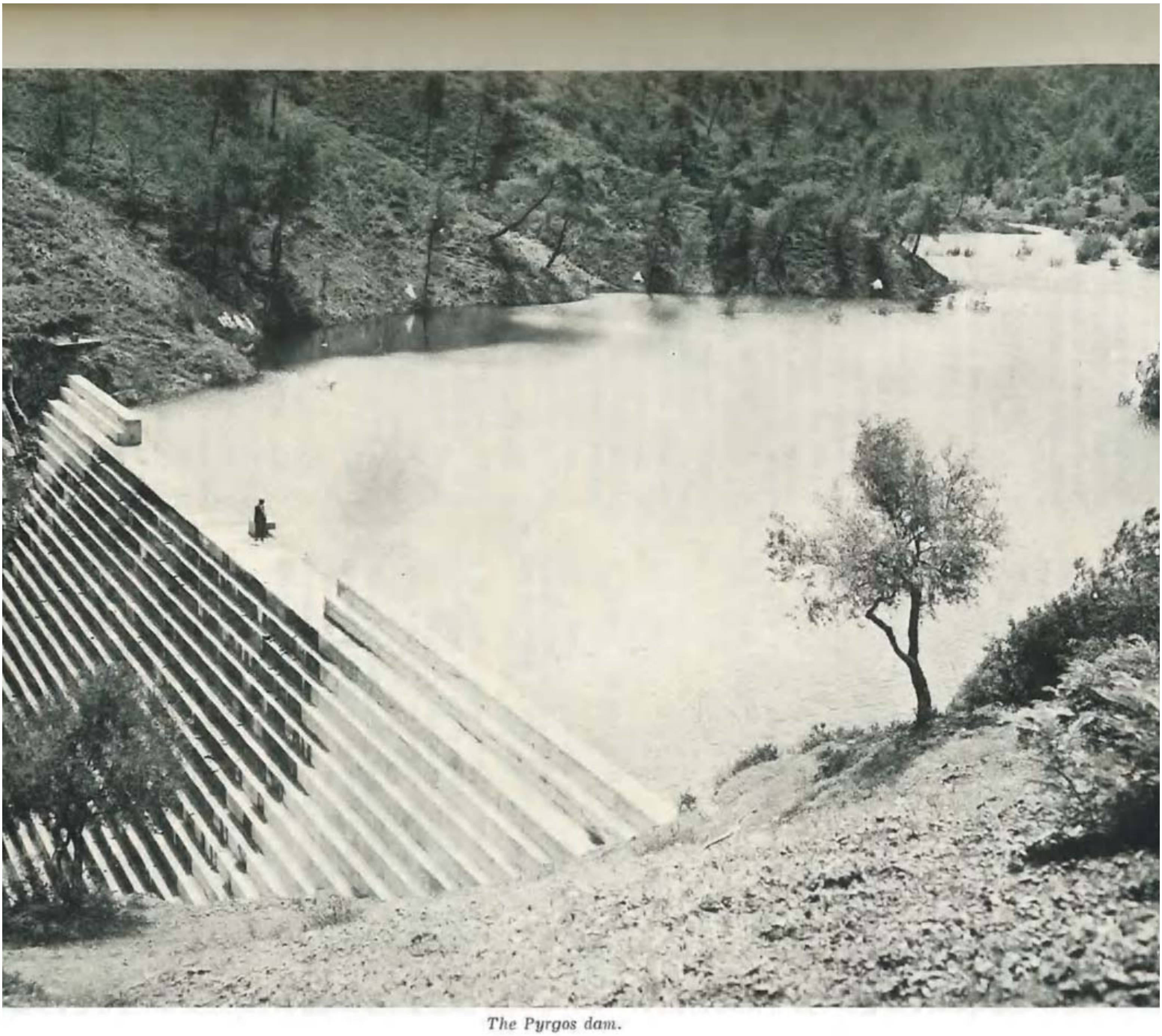
the dam was emptied in 2.5 hours revealing some 13 m of asbestos sediment. Before the desilting of the dam had been completed a very heavy rain occurred in December 1968. This rain, which was one of the highest intensities recorded over the past 50 years, conveyed a huge quantity of sediment, boulders, tree trunks and rocks which broke to pieces all metal structures and resulted in the collapse of the 30 m high protective tower grill which blocked the tunnel completely and silted up the reservoir up to 25 m from the riverbed, that is to say only 5 m below the crest. In May 1969 operations were started again to clear the reservoir which included a specialized light blasting operation to break away the trunks which blocked the tunnel entrance. After 2 blasting operations, the tunnel opened and the silt gave way so that the dam emptied partly but filled up again with sediment the next day. A series of operations using compressed air pumped into the tunnel and the main outlet pipe was then started from the upstream side in order to stir the sediment but it met only with partial success. Lastly an attempt was made to blast away the junk that was in the tunnel. This blasting was successful and the tunnel was cleared and emptied in 2.5 hours. Immediately all metal structures were removed from the riverbed to avoid blocking of the tunnel again. All this operation for clearing the dam lasted until the end of November 1969 and the total cost was £2,100. After these desilting works were completed, the Mines undertook to introduce a different type of desilting arrangement which consisted of a vertical outlet tube of 1 m dia connected to the tunnel at the small gate entrance and fitted with 12 trap gates. A hand operated winch was installed on the spillway crest to operate the penstock and trap gates. One iron penstock 1.37x1.2 m was installed on the big tunnel entrance held in position by the pressure of the water. Finally, upstream of the reservoir a protecting grill 16 m wide and 2.5 m high was installed consisting of 3 sections each having 11 pendulum type iron hooks (see photo). This grill serves the purpose of holding all tree trunks and drift wood back. In the case of the large rocks the pendulums help to retard their movement thus preventing the breakage of the grills.

Data

i. Catchment	
Area	51.5 km²
Average rainfal	972 mm/a
Average runoff	19.4 million m³/a
1/1000 years flood	242 m³/s
Maximum height	1,760 m
Maximum length	14.6 km
ii. Reservoir	
Area	2.3 ha
Capacity	340,000 m³
Live storage	300,000 m³
Length	850 m
iii. Dambody	
Structural height	33.30 m
Height above ground level	28.75 m
Hydraulic height	27.53 m
Depth of foundation cut-off	4.55 m
Freeboard	1.22 m
Crest length	76.19 m
Top thickness	3.50 m
Base thickness	24.38 m
Upstream slope	Vertical
Downstream slope	1:0.61
Total concrete fill	6,115 m³
iv. Excavations	
Total	1,000 m³
v. Spillway	
Size	30.47x1.22 m
Capacity	58.70 m³/s
vi. Gallery	
Size	1.37x1.07 m
Capacity	19 m³/s
Length	18.28 m
Large operating gate size	1.37x1.2 m
Small operating gate size	0.43 m dia
vii. Outlet (steel pipes)	
Irrigation	
Size	0.15 m dia
Capacity	0.117 m³/s
Length	60 m
Compensation	
Size	0.25 m dia
Capacity	0.18 m³/s
Length	855 m
viii. Distribution System	
(i) Main Conveyor (concrete channels)	
Size	0.43x021 m
Capacity	0.042 m³/s
Length	1,870 m
(ii) Steel pipes	
Size	0.20 & 0.15 dia
Capacity	0.056 m³/s
Length	1,325 m
(iii) Distribution Network (concrete channels)	
Size	0.20x0.20 & 0.25x0.15 m
Capacity	0.021 m³/s
Length	745 m
(iv) Steel pipes	
Size	0.15 dia
Capacity	0.021 m³/s
Length	7,220 m

Trimiklini dam. — Upstream silt trap.

Trimiklini dam. — Desilting pipe with trap gates.



The Pyrgos dam.

KATOURIS—PYRGOS PROJECT

1. PURPOSE

Pyrgos is the largest village of Tylliria and the majority of its inhabitants are farmers. Before the construction of the dam, they were mainly occupied with dry farming, their chief crop being figs. Their demand for the supply of summer water through the construction of a dam would not only supply them water to satisfy the limited water requirements of the fig trees, but furthermore it would enable them to plant more remunerative crops which grow very well in this particular region. This dam was the first to be built in Tylliria and was the starting point for the execution of other dams in this region.

2. LOCATION

The dam is built on the Katouris river at an elevation of about 85 m asl and at a distance of about 5 km from the coast.

3. PLAN

a. Water and Land

The average annual runoff at the damsite is estimated to be 2 million m^3.

The Katouris river flow had never been used prior to the construction of the dam and a new Irrigation Division had to be formed in 1953 to take over the scheme.

The scheme commands 1,600 donums of land out of which 350 donums are planted with trees such as fig and almond trees which are rainfed and could grow without irrigation. Irrigated trees such as peach, pear, plum, orange and bananas were planted after the construction of the dam. The remaining land is planted with vegetables such as potatoes, tomatoes, cucumbers, cabbages as well as melons, watermelons and strawberries. Also during winter and early spring when no irrigation is required cereals and broadbeans are grown.

b. Geology

The damsite is a narrow V-shaped gorge and the valley runs in a NS direction.

Sheeted diabase covers the damsite area with minor outcrops of intrusive dykes of Gabbro. The diabase is medium grained, vesicular and of light greenish-grey colour. It is altered, highly fractured and ironstained. Vesicles are filled with epitode as well as the weather fractures and joints.
The trend of the sheets is roughly NS dipping at an angle of 60° towards the West.

Two sets of joints are predominant in the area. Their trend and dip being 42°E dipping 30°SE and N 30°W vertical. In the river bed about 1 m gravel overlies sound diabase rock.

On the West side of the valley the diabase is more solid than on the East where rain water percolation along the joint planes and temperature variations have caused weathering of the rock both along the major joint planes and at right angles.

4. MAIN FEATURES

The dam and distribution system were designed by the WDD.

a. Dam

The dam is of mass concrete gravity design.

The upstream face is vertical and the downstream face has a slope of 1:0.67 with a stepped design for breaking up the velocity of flow.

The dambody contains a drainage system to relieve hydrostatic pressure, made up of 3 No. 7.5 cm dia horizontal perforated pipes laid at the lower half of the dam, leading the drainage water to a 7.5 cm dia perfortated vertical drain which discharges into the gallery. Also 5 cm dia perforated pipes laid along the abutments and foundations drain into the gallery.

There is a gallery through the dambody which after construction is used for desilting the reservoir and for carrying away the drainage flow.

The penstock operating the gallery gate is operated by a gear box through a spindle rod.

b. Distribution System

The steel outlet pipe is connected to the main conveyor reinforced concrete cast-in-situ rectangular channel, 180 m downstream of the dam. Thereafter, the main conveyor channel supplies water to the distribution steel pipes conveying the water to each private holding for irrigation, covering a total length of 11,950 m.

5. CONSTRUCTION

Both the dam and distribution system were constructed by the WDD.

The construction started in January 1957 and was completed in June 1960. The foundation was excavated up to about 2 m at the river bed including the removal of some gravels overlying the bed rock. The filling of the foundation was done with 1:2:4 concrete.

The central portion of the dambody was filled in 1:8 cement-aggregate concrete including also 15% plumbs. The outside part of the dam was filled with 1:2:4 concrete. The casting of the concrete of the dambody was done from either abutment towards the centre of the river bed which was left open to a width of 5 m up to the top crest in order to allow passage of the water during construction. The central part was cast last.

All materials were obtained from the river bed and from the nearby coast 5 km downstream.

The total fill with concrete was 7,650 m^3 and the cost of the dam reached £26,400.

The distribution system was constructed between 1957 and 1969 in four phases, and is made up of a system of reinforced concrete rectangular cast in-situ channels for the main conveyor and steel pipes for the secondary and tertiary distribution. The 1957 phase was the main conveyor reinforced concrete channels, at a cost of £18,500. The second phase in 1961 was mostly secondary steel pipes with some concrete channels at a cost of £5,500. The third phase in 1964 at a cost of £3,200 was secondary steel pipes with some concrete channels. The fourth phase in 1969 was secondary and tertiary steel pipes to serve all holdings at a cost of £10,000.

The total cost of the distribution system was £37,200.

The funds for the construction were provided by the Government. The Irrigation Division undertook to pay a low interest loan of 10—20 years repayment period for £16,510. The percentage contribution on the total investment was 25% except for the last phase of the distribution system for which it was 33%

6. MANAGEMENT

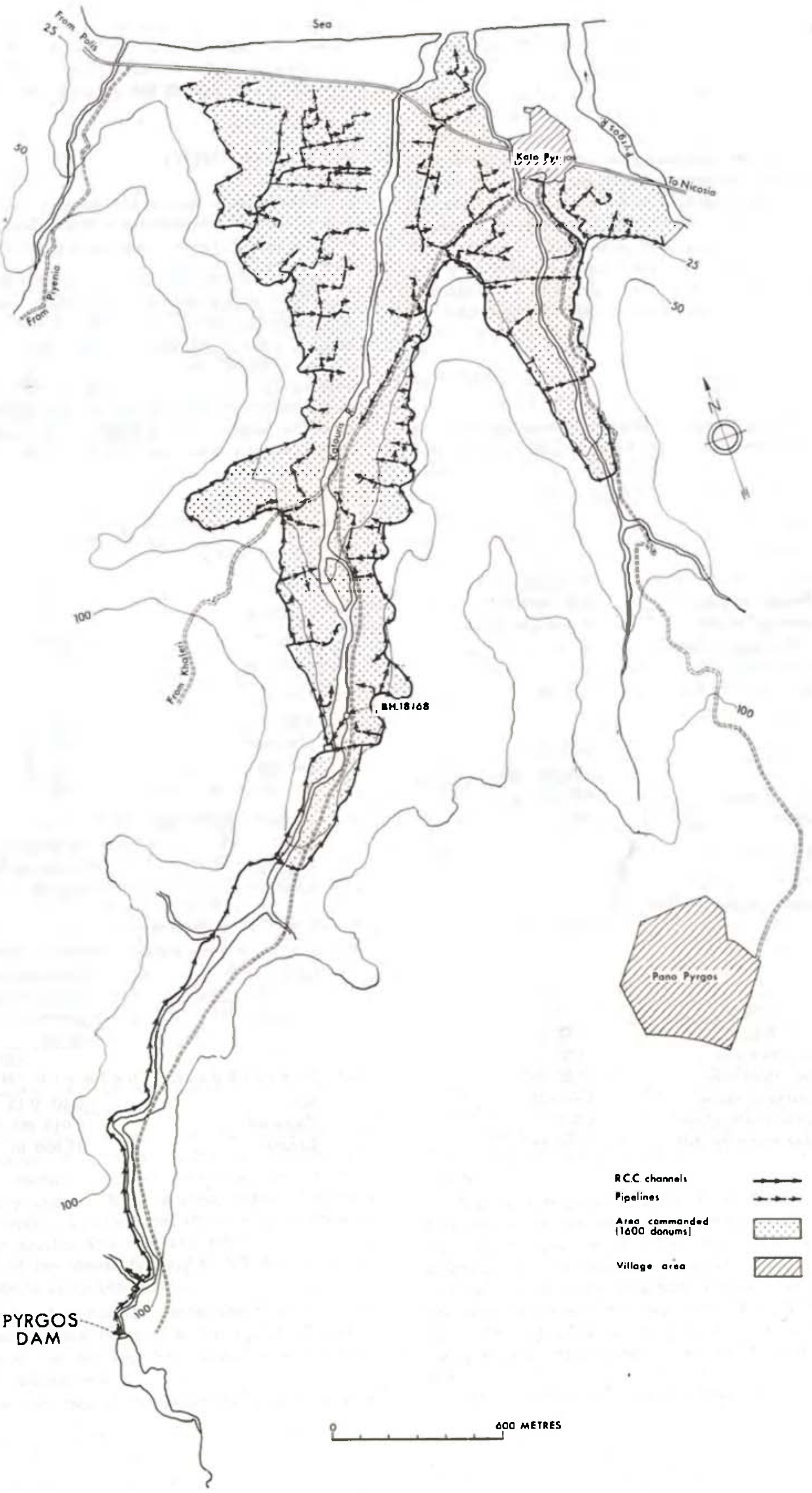
The Pyrgos Irrigation Division made up of 360 members who own 1,600 donums of land within the Division are responsible for the management of the Project.

As the distribution system now covers the whole area up to each individual land holding, additional water is required to satisfy the 1600 donums and for this purpose private boreholes in the river bed are used to supplement the dam water. Furthermore the Government has drilled a borehole which was given to the Division and incorporated in their system to supplement the supply. The possibility for raising the dam in the future is also under consideration.

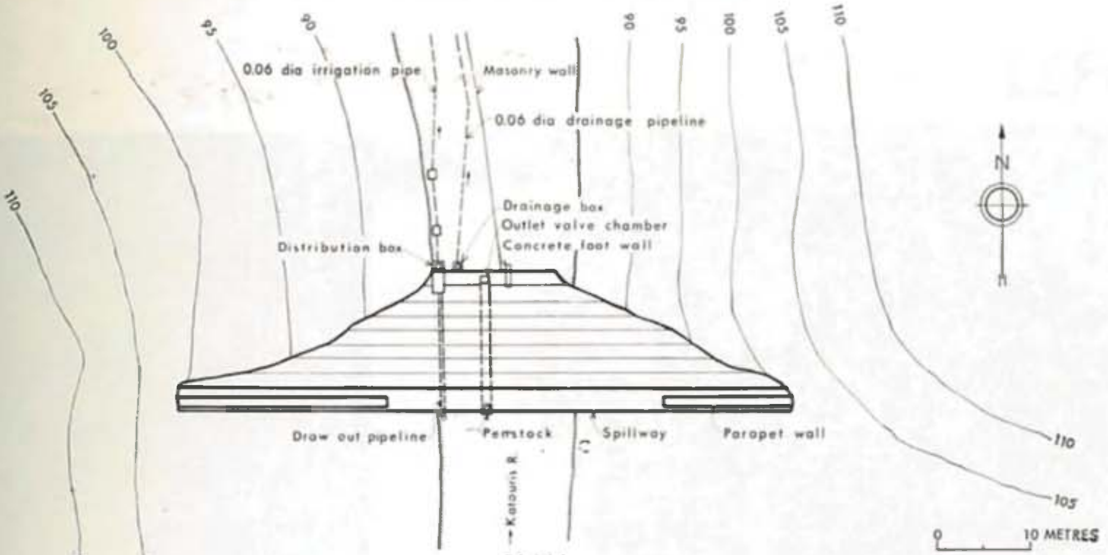
Data

i.	*Catchment*	
	Area	13.5 km^2
	Average rainfall	620 mm/a
	Average runoff	2 million m^3/a
	1/1000 years flood	73 m^3/s
	Maximum height	840 m
	Maximum length	9.5 km
ii.	*Reservoir*	
	Area	3.0 ha
	Capacity	283,000 m^3
	Live storage	283,000 m^3
	Length	480 m
iii.	*Dambody*	
	Structural height	22.25 m
	Height above ground level	20.40 m
	Hydraulic height	18.90 m
	Depth of foundation cut-off	1.85 m
	Freeboard	1.50 m
	Crest length	65.50 m
	Top thickness	2.50 m
	Base thickness	17.30 m
	Upstream slope	Vertical
	Downstream slope	1:0.67
	Total concrete fill	7,650 m^3
iv.	*Excavations*	
	Total	1,000 m^3
v.	*Spillway*	
	Size	30x1.50 m
	Capacity	120 m^3/s
vi.	*Gallery*	
	Size	1.90x1.20 m
	Capacity	12.5 m^3/s
	Length	13.5 m
	Operating gate size	0.38 m dia
vii.	*Outlet (steel pipes)*	
	Size	0.15 m dia
	Capacity	0.057 m^3/s
	Length	220 m
viii.	*Distribution System*	
(i)	*Main conveyor (concrete channels)*	
	Size	0.35x0.22 & 0.30x0.22 0.30x0.30 & 0.30x0.20m
	Capacity	0.057 m^3/s
	Length	15.130 m
(ii)	*Distribution network (steel pipes)*	
	Size	0.10, 0.15 & 0.20 dia
	Capacity	0.014 m^3/s
	Length	11,950 m

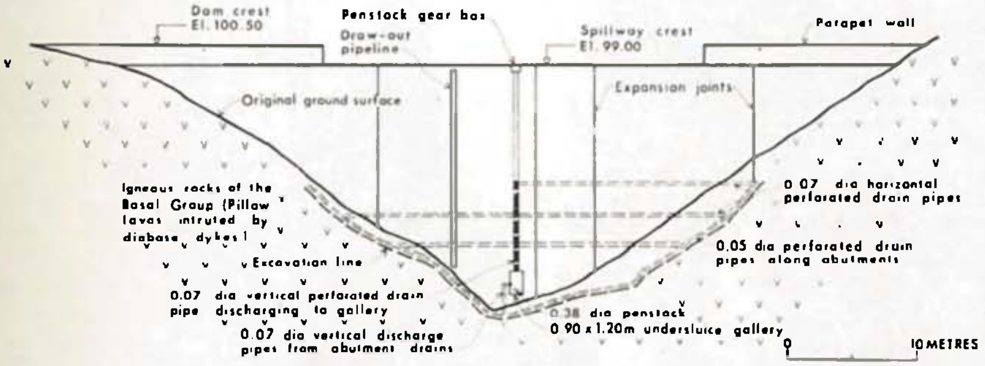
KATOURIS-PYRGOS-DISTRIBUTION SYSTEM



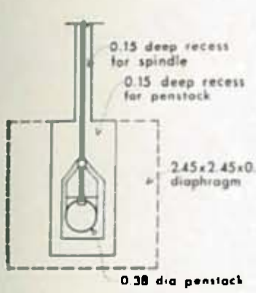
KATOURIS-PYRGOS DAM



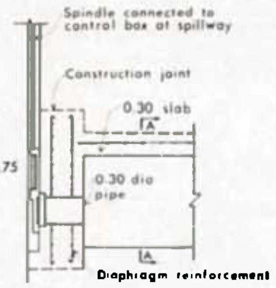
PLAN



SECTIONAL ELEVATION & GEOLOGICAL SECTION

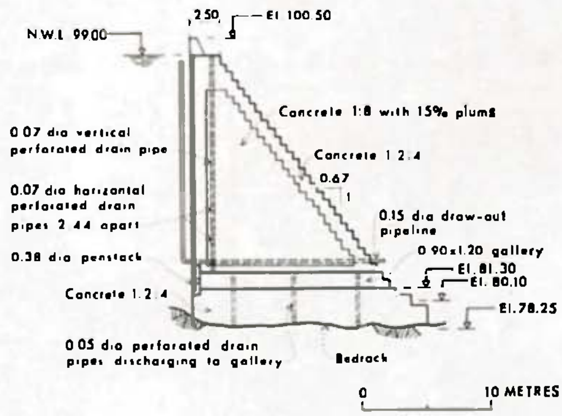


Elevation

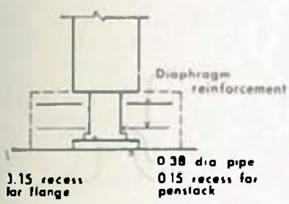


Section

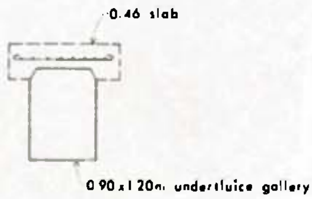
0 1 METRES



MAXIMUM SECTION

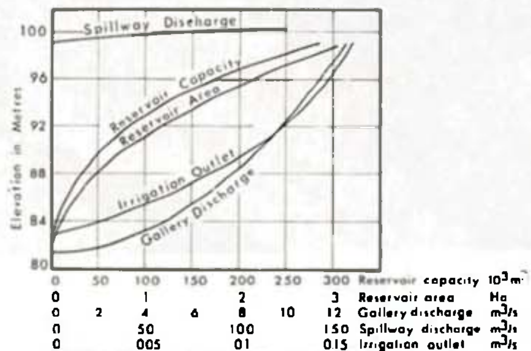


Plan



Section A-A

PENSTOCK DETAILS



CAPACITY-AREA-DISCHARGE CURVES

The Lefka dam.

MARATHASA—LEFKA PROJECT

1. PURPOSE

The Marathasa dam is an extension of the Kafizes Project and is meant to provide supplementary water to satisfy the total water requirements of the Lefka citrus gardens.

2. LOCATION

The dam is built on the Marathasa river at an elevation of about 250 m asl and at a distance of about 10.5 km from the sea.

3. PLAN

a. Water and Land

The average runoff is estimated to be 15.6 millions m^3.

Lefka farmers had ab-antiquo water use rights on the Marathasa river before the construction of the dam for irrigating their lands extending to about 2070 donums of which about 560 of citrus, mainly orange groves, are the most important crop.

b. Geology

The Basal group series covers the area of the dam. This group consists mainly of sheeted diabase with some inclusions of pillow lavas.

The sheeted diabase is vesicular highly fractured and jointed, highly to very highly weathered. The vesicles are filled with secondary silica, epitode, calcite and desseminated pyrite. Mineralization in the fractured zones was also found. In some cases this fills and cements the fracture spacings.

The sheets trend N 20°E and dip towards the NE at angles varying from 40° to 70°.

The main joints have a strike of N 118°S and NS with dips varying from 75°W to vertical.

On the Western abutment the bedrock is covered with an old river terrace which in places reaches 15 m in thickness. The bedrock itself is badly shattered. In the main river channel, soft rock impregnated with pyrite was exposed and extends into the lower portion of the Eastern face slightly upstream from the dam axis.

In general, the whole of the Marathasa river presents adverse geological problems. It is probable that the main river course was determined originally by a fault along which erosion could more easily form a channel. Minor deviations of the river course have happened at various places cutting through rock new channels whilst terraces cover the older higher river bed.

4. MAIN FEATURES

This dam and its distribution system were designed by the WDD.

a. Dam

The dam is of mass concrete gravity design, with the upstream face vertical and a stepped downstream face sloping at 1:0.58 in the upper part and 1:0.84 in the lower part.

A horizontal and vertical drainage pipe system is provided in the dambody and in the foundations to relieve the hydrostatic pressure.

A desilting and drainage gallery through the dambody, and a sluicing penstock 0.61x0.61 m operating from full reservoir level are provided on the upstream face. On the left abutment formed of the old river valley, a concrete cut-off was provided with earthfill zones on either side.

A steel outlet pipeline with a vertical upstream perforated inlet passes through the gallery having its exit in a settling tank just downstream.

b. Distribution System

The main conveyor is a steel pipeline whilst various sizes of earth distribution channels within the area of the gardens were lined in concrete.

For the distribution system layout map see Xeros—Kafizes project.

5. CONSTRUCTION

The whole project including the dam and the distribution system were constructed by the WDD.

Construction on the dam started in September 1959 and was completed in May 1962.

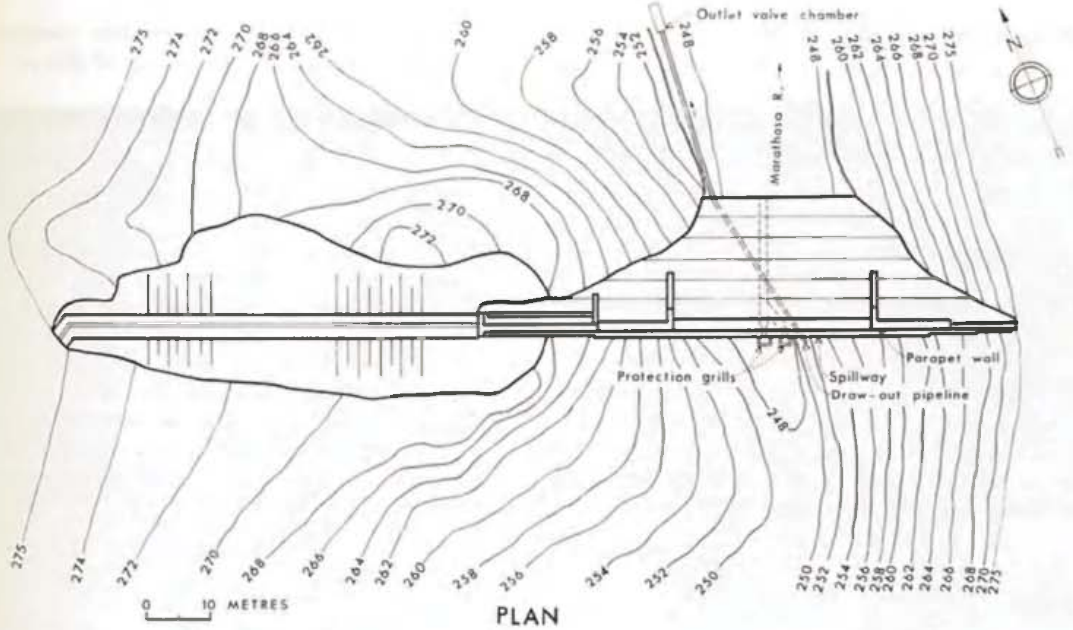
The foundations are in cement concrete 1:2:4 and the dambody in cement concrete 1:9 whilst the upstream and downstream faces are in 1:2:4 mix.

The excavation in the main river bed had to reach about 8 m depth whilst on the left abutment in the old river valley, It reached about 16 m depth before good sound rock was found. On the old river valley side most of the excavation was done by hand in a narrow trench 3 m wide, using scaffolding to support the faces. This trench was first filled with mass concrete whilst the upper part was cast as a thinner reinforced concrete wall with earthfill zones on either side. Hand placed rip-rap was used on the upstream side. The rock was obtained by blasting from a nearby quarry upstream of the dam, whilst gravel and sand were brought to the site from the coast downstream. The total cost of the dam was £110,700. The steel conveyor pipeline from the dam to the gardens was started in August 1958 and was completed in November 1958 at a total cost of £7,700.

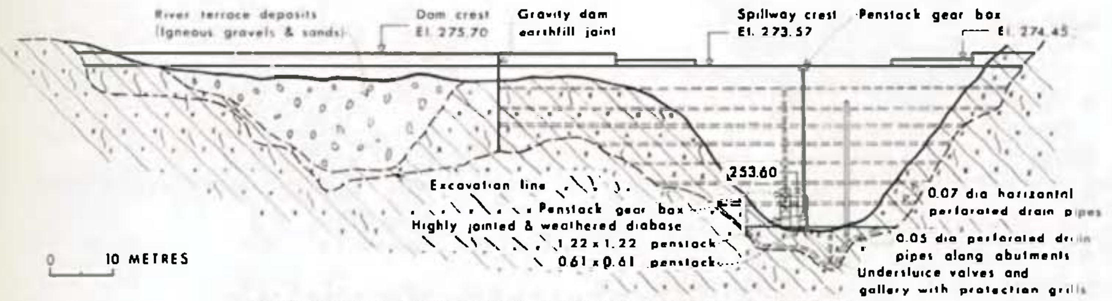
The lining of some of the earth distribution channels in cast-in-situ rectangular concrete section was done between January and December 1963 at a cost of £8,500.

Thus the total cost of the main conveyor and the

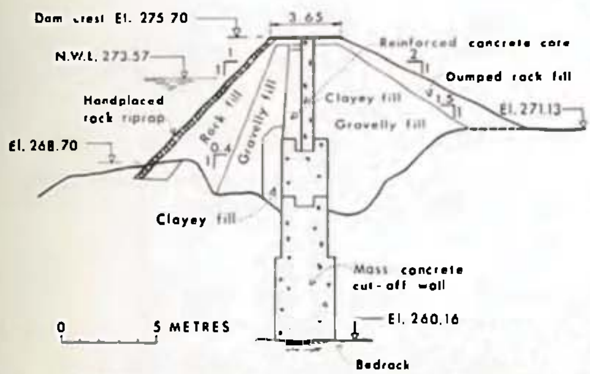
MARATHASA-LEFKA DAM



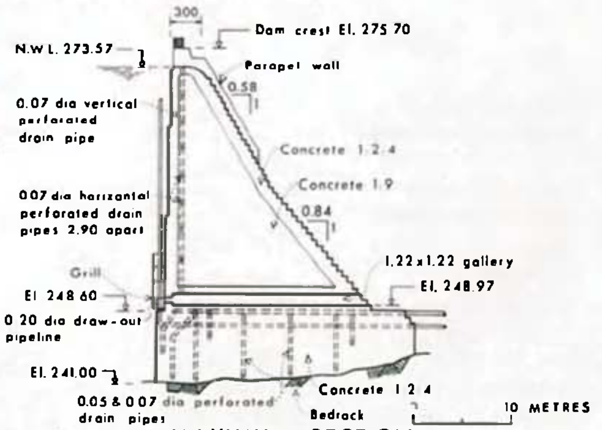
PLAN



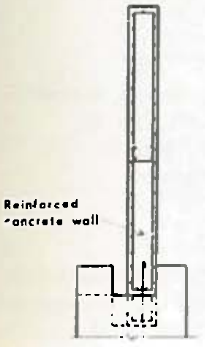
SECTIONAL ELEVATION & GEOLOGICAL SECTION



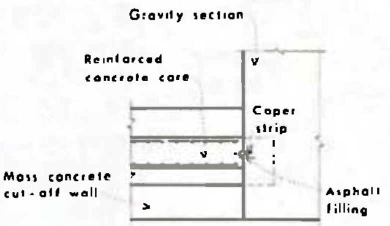
MAXIMUM SECTION THROUGH EARTHFILL



MAXIMUM SECTION



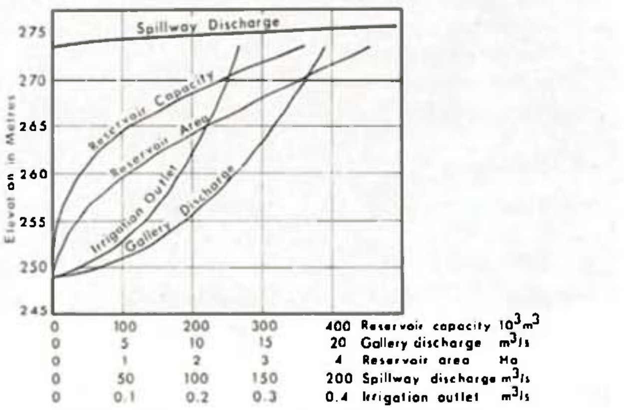
R.C. Wall details



Plan at gravity dam-earthfill connection

0 5 METRES

CONCRETE CUT-OFF WALL



CAPACITY-AREA-DISCHARGE CURVES

distribution system reached £16,200.

For the steel conveyor, the Irrigation Association signed a loan to contribute 55% of the total cos over 20 years period. For the dam and distribution channels, the loan signed was for 38% of the cost payable over 20 years.

6. MANAGEMENT

The project is managed by the Lefka Marathasa Irrigation Association which was made up of 97 proprietors, at the time when the dam was built.

They own shares of the water, their ownership varying from a few minutes to up to two days per month. The number of proprietors and their shares are a little different than those mentioned for Kafizes project which was the first dam built for the same Association, because the situation changes with time through inheritance and selling of shares.

7. PROBLEMS OF SPECIAL INTEREST

An interesting aspect regarding this dam is the old riverbed terrace on the left abutment and its treatment.

The dam was first built to a height of 18 m before excavating and filling the pervious abutment. After impounding with water and testing at this height for leakage, it was observed that the seepages amounted to 2.5 l/s which would obviously increase at the full height of 27 m. It was therefore decided to excavate the alluvium down to sound bedrock, fill in with concrete and finish off the dam to its full height. After completion and water testing it was found that the leakage had stopped.

Data

i. Catchment

Area	54.6 km²
Average rainfall	850 mm/a
Average runoff	15.6 million m³/a
1/1000 years flood	320 m³/s
Maximum height	1,760 m
Maximum length	14.2 km

ii. Reservoir

Area	4.5 ha
Capacity	368,450 m³
Live storage	365,000 m³
Length	630 m

iii. Dambody

Structural height	34.70 m
Height above ground level	27.10 m
Hydraulic height	24.97 m
Depth of foundation cut-off	7.60 m
Freeboard	2.13 m
Crest length	149 m
Top thickness	3.00 m
Base thickness	26.82 m
Upstream slope	Vertical
Downstream slope	1:0.58, 1:0.84
Total concrete fill	15,000 m³
Earth embankment upstream slope	1:1
Earth embankment downstream slope	1:2
Total earth fill	5,000 m³

iv. Excavations

Total	5,760 m³

v. Spillway

Size	30.48x2.13 m
Capacity	246 m³/s

vi. Gallery

Size	1.22x1.22 m
Capacity	19.4 m³/s
Length	21.33 m
Operating gate size	0.61x0.61 m

vii. Outlet (steel pipes)

Size	0.20 m dia
Capacity	0.27 m³/s
Length	52 m

viii. Distribution System

(i) Main Conveyor Steel Pipes

Size	0.20 m dia
Capacity	0.084 m³/s
Length	1,620 m

(ii) Concrete channels

Size	0.30x0.23 m
Capacity	0.113 m³/s
Length	3,000 m

(iii) Distribution network (steel pipes)

Size	0.25, 0.15 m dia
Capacity	0.113 m³/s
Length	1,250 m

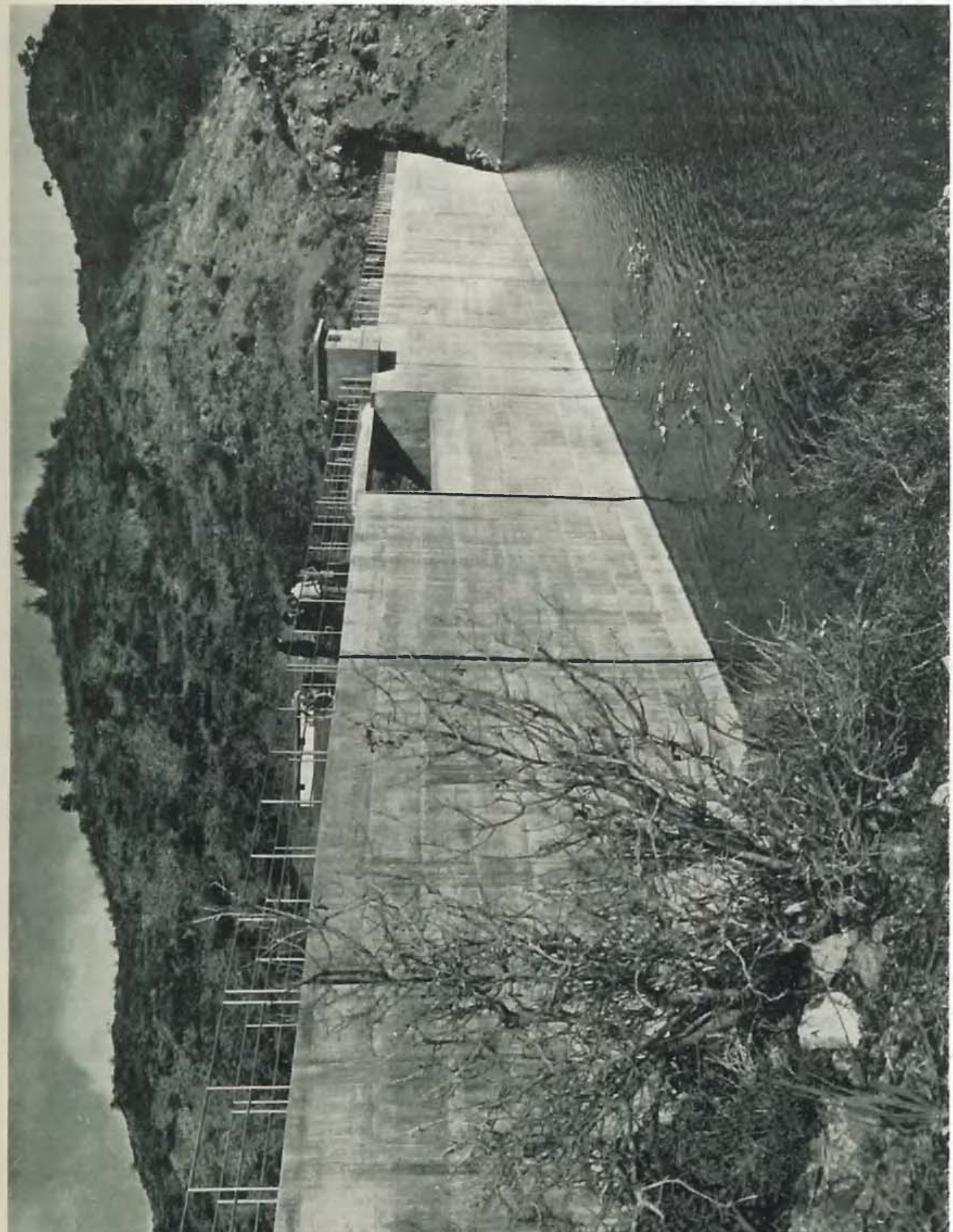
(iv) Concrete channels

Size	0.30x0.23, 0.40x0.23, 0.50x0.23 m
Capacity	0.113 m³/s
Length	1,500 m

(v) Earth channels

Size	0.30x0.18, 0.35x0.18, 0.38x0.18, 0.41x0.20 m
Capacity	0.113 m³/s
Length	4,900 m

PALEKHORI



The Palekhori dam.

PALEKHORI—KAMBI PROJECT

1. PURPOSE

The intake and earth channel built in 1949 on the Maroullena tributary of the Serakhis river, and the subsequent concrete lining in 1961 could provide water for the irrigation of winter and spring vegetables only, since the river is dry during the summer months. Also rainfed fruit trees such as almonds, olives and vines were extensively cultivated on the 1166 donums commanded.

The income from these crops is very low and therefore the farmers constantly pressed for a storage dam to enable summer and perennial irrigation.

2. LOCATION

The dam is built on the Kambi tributary of the Serakhis river at an elevation of about 700 m asl and at a distance of about 50 km from the coast.

3. PLAN

a. Water and Land

The average runoff is estimated to be 2 million m^3/a. The Irrigation Division had an intake and a canalization system from the river taking water prior to the construction of the dam.

The net irrigable area after land levelling of the 1166 donums commanded is about 530 donums, and the annual water requirements for the irrigation of this area is about 500,000 m^3.

Water use rights on this river are claimed by the downstream villages at Akaki and Meniko.

b. Geology

The damsite lies approximately 1 km SE and upstream of the confluent with the Maroullena river.

The Apliki region is occupied by the diabase consisting of parallel multiple dykes of andesitic or basaltic composition. The diabase is sheeted and is well pronounced by a set of well developed joints running parallel to the strike of the dykes.

The diabase is well exposed along the river channel and at the steeper parts of the slopes. The flatter slopes are covered by a thin mantle of soil or talus. The upper portions of the slopes are usually open jointed and were subjected to more intense weathering than the lower part, which is generally fresh, hard and closely jointed. Minor river terrace deposits consisting of boulders, gravels and sands are met in the river channel.

On the upper part of the left hand slope looking downstream there is a fault which is filled with breccia and clays. It appears that its location and direction is such that no important leakage problems would be encountered.

4. MAIN FEATURES

a. Dam

The Dam and distribution system were designed by the WDD and is of mass concrete gravity type.

It is founded on sound rock and consists ot 13 vertical blocks extending over the entire height of the dam. A continuous grout curtain extending over the whole length of the dam was provided in the dam foundations, injections being done through a concrete cap.

The fifth block from the left abutment of the dam, is serving as a spillway, 10.36 m wide discharging overflow water into a flip bucket.

A drainage and inspection gallery extends through the whole length of the dam. A valve chamber formed partly within the gallery accomodates the sluice valves. A series of drains 10 cm dia formed behind the contraction joints at 11 m spacing with a second row of drains also 10 cm dia formed in front of the upstream face of the gallery at 3.5 m spacing, drain into the gallery. Similarly, from the foundations, downstream of the grout curtain 10 cm dia relief drains at 5 m spacing were installed discharging into the gallery.

An outlet gallery serving also for desilting is electrically operated by a gate.

Two steel pipes situated in a recess along the sluiceway are serving as irrigation and compensation pipes respectively.

A concrete foot bridge 12x1.3 m spans the spillway.

b. Distribution System

A scheme for lining in concrete the earth channels of the Division was carried out in 1961. The irrigation conveyor diverted water from a masonry intake built on the Maroullena river in 1949.

Extensions of the distribution network are scheduled to be constructed soon.

5. CONSTRUCTION

The construction of the dam was carried out by the Contractors Joannou and Paraskevaides Ltd, who started work in October 1971 and completed in October 1973.

The excavation of the dam foundation, approach channel to sluiceway, flip bucket and stilling basin foundation was completed by July 1972.

The concreting started in May 1972 and finished in October 1973. The Contractor used a crushing plant at Malounda village in the main river downstream for the supply of aggregates. An automatic batching plant for mixing the concrete with two tower cranes for conveying and placing of a total output of 40 m^3/hr were used on the site.

Drilling and grouting works commenced in May 1972 and were finished in November 1972. The whole work has been carried out by the subcontractor Energoexpostroy of Bulgaria, a specialized firm on drilling and grouting.

The treatment of the rock consisted of two rows of holes 2 m apart beneath the upstream part of the concrete dam. The main grout curtain line was of about 18 m depth and the second of about 10 m depth with hole spacings 2.4 m apart. From the results obtained of the main curtain, it was observed that some sections were tight enough so a considerable number of holes provided in the design for the second grout curtain were omitted.

The inclination of the boreholes was variable. On the left flank the inclination was 15° to the vertical with direction towards the left abutment. On the right flank the inclination was 30° to the vertical with direction towards the right abutment. In the riverbed the boreholes had radial direction, more or less perpendicular to the excavation level.

After the completion of the grouting works, control holes were drilled in various zones and water tests were carried out in order to check the effectiveness of the grouting. The results obtained were satisfactory.

The drilling of the grout holes was carried out with rotary percussion rigs with hole size 50 mm using water flushing through the concrete cap of about 1.5 m thick. The control holes were drilled with diamond core drills with core size 62 mm. The holes were drilled in one set up to the full designed depth except when there was collapsing of the rock in which case the collapsed part was grouted first and after setting of the cement grout, the remaining depth was drilled and grouted.

The grouting was carried out mostly in stages of 5 m in ascending order using rubber packers.

The grout mixture was composed of water and ordinary portland cement with the addition of 3% bentonite.

The grout pressures applied were 0.46 kg/cm²/m depth. The required pressure for each stage was regulated with the return line value and was maintained till the initial set of the grout, the minimum of which was 10 minutes.

The grouting of any stage was considered finished when the consumption fell to 1.5 l/m for pressures up to 3.5 kg/cm- and 2 l/m for higher pressures.

All grout holes were filled up with cement mortar after completion.

The average permeability of the control holes water tested was 1.6 lugeons.

The total cost of the grouting works reached £17,000. Table 9 gives the main data for the grouting works carried out:

The total cost of the dam including the foundation treatment was about £330,000.

The main conveyor system was constructed prior to the dam in the year 1961, at a cost of £18,500. Certain extensions of the distribution network have still to be constructed for efficient water utilization.

The Irrigation Division signed a loan to contribute 25% of the total cost of the dam and 33.3% of the cost of distribution system, over 25 years period.

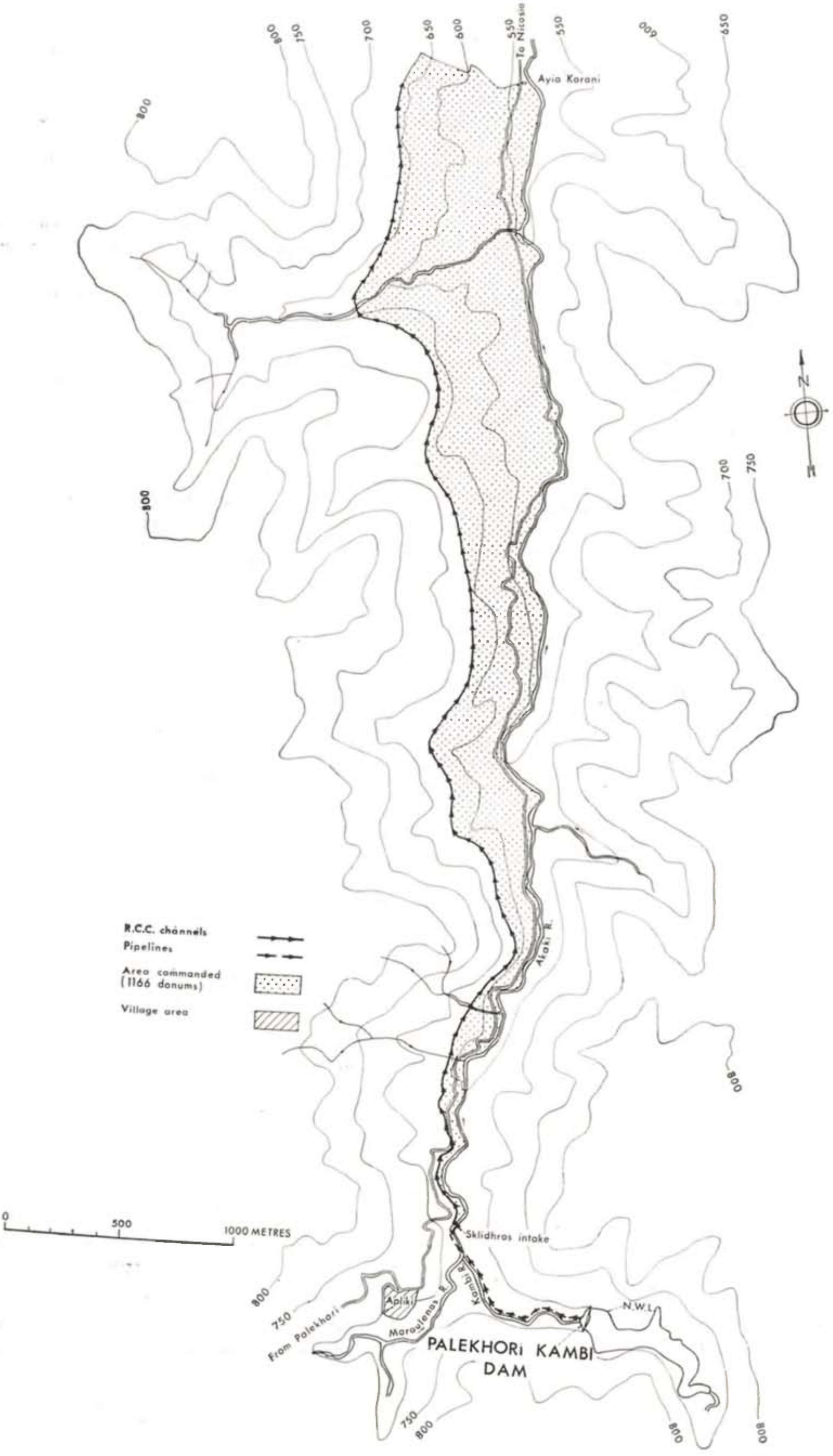
6. MANAGEMENT

The project is managed by the Palekhori Irrigation Division which covers 1,166 donums of land commanded by the main conveyor.

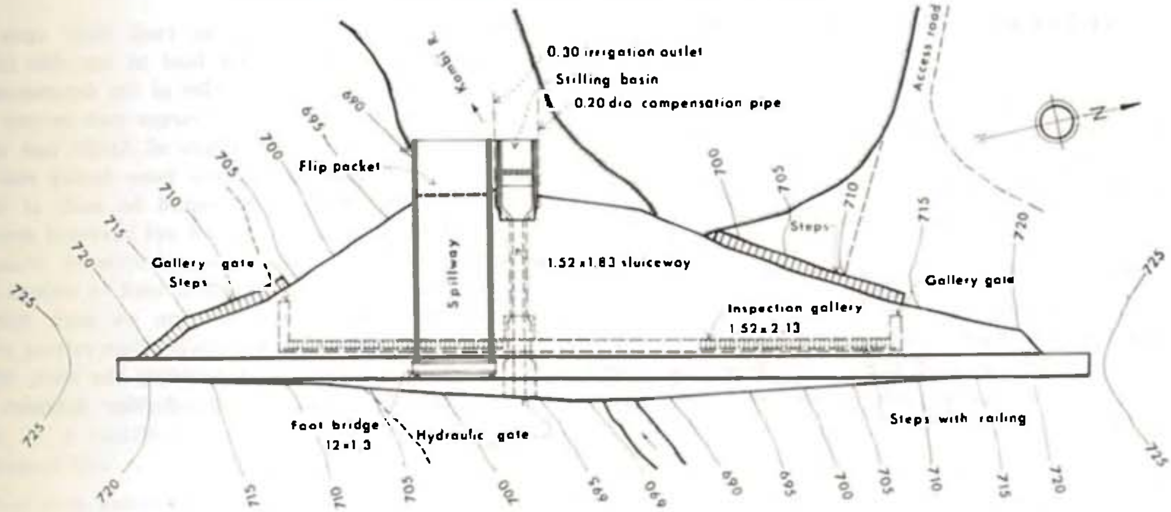
TABLE 9

Main Grout Curtain	Number of holes	Number of stages	Total drilling for grouting	Consumption of cement kg	Consumption of bentonite kg	Total dry materials	Average grout take kg/m
Section 1	28	73	325	4416	137	4553	14.0
" 2	18	39	285	1310	40	1350	4.7
" 3	26	84	384	4497	139	4636	12.1
Totals	72	196	994	10223	316	10539	10.6
Second line of grouting							
Section 1	6	7	35	644	20	664	18.9
" 2	4	4	10	16	—	16	1.6
" 3	14	15	114	882	27	909	7.9
Totals	24	26	159	1542	47	1589	10.0
Grand Totals	96	222	1153	11765	363	12128	10.6

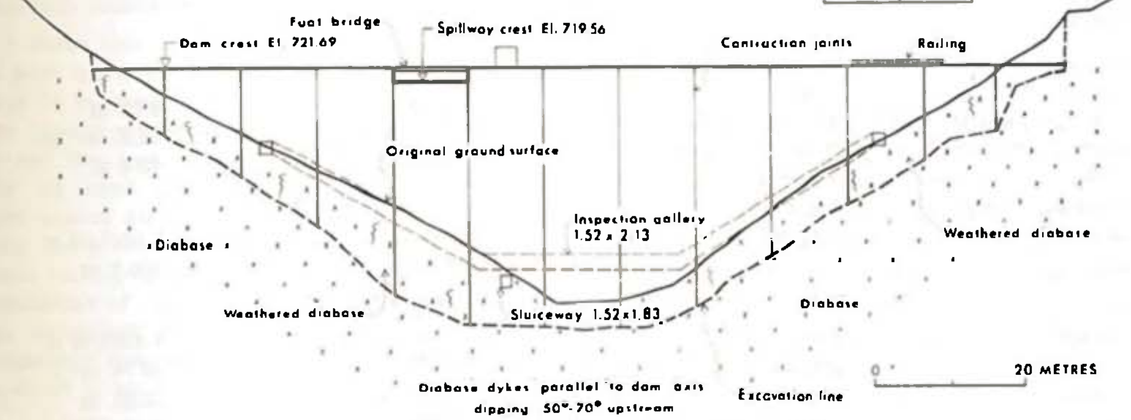
AKAKI-PALEKHORI KAMBI-DISTRIBUTION SYSTEM



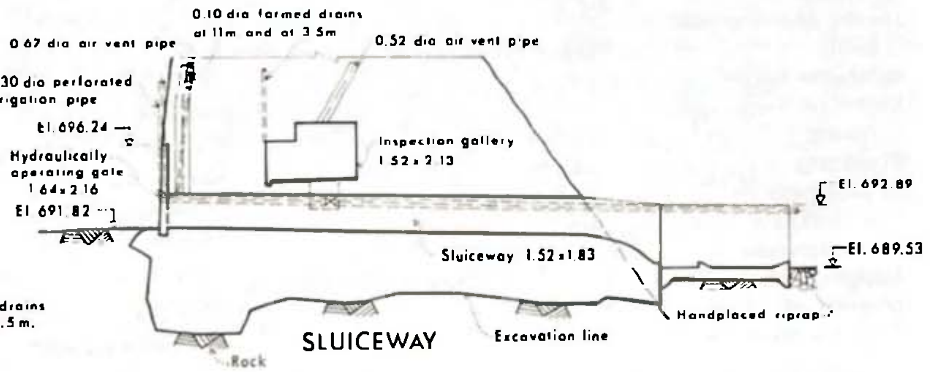
AKAKI-PALEKHORI KAMBI DAM



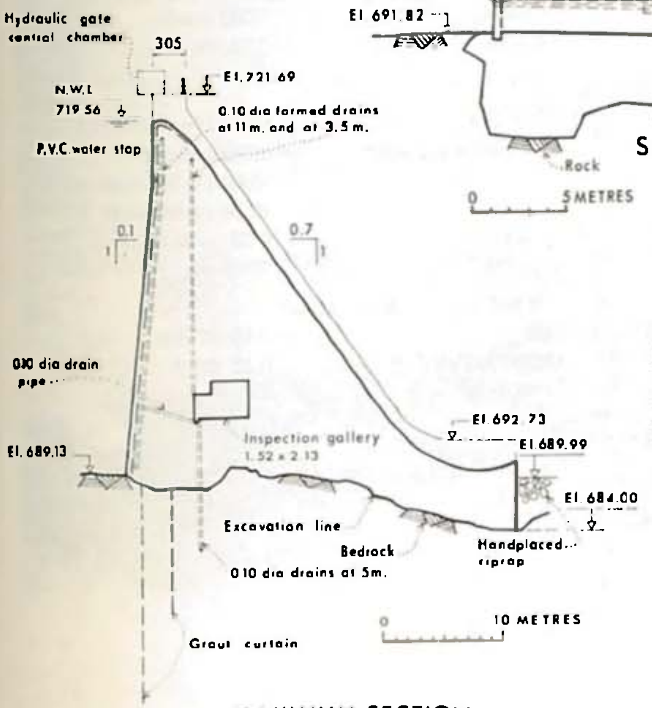
PLAN



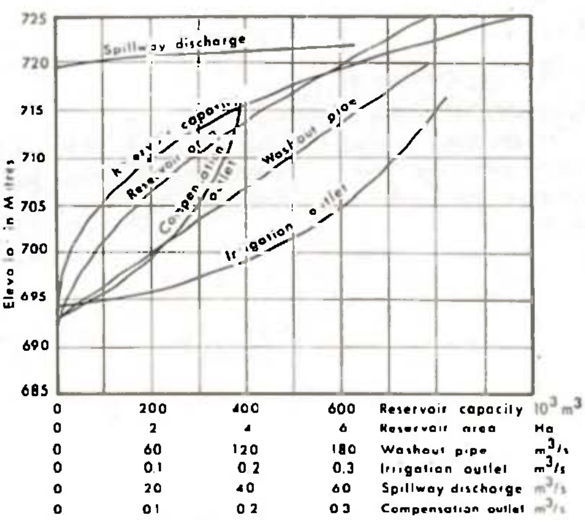
GEOLOGICAL SECTION ON DAM AXIS



SLUICEWAY



MAXIMUM SECTION



CAPACITY-AREA-DISCHARGE CURVES

7. PROBLEMS OF SPECIAL INTEREST

The original damsite chosen, where extensive investigations were carried out. was at Sklidhros, at the confluent of the Kambi and the Maroullena tributaries of the Akaki river. However, important weaknesses were proved at this site which made it necessary to abandon it in favour of an upstream site in the Kambi tributary. The main weaknesses could be described as follows:-

A major fault zone of the Troodos massif following the Maroullena river channel. Parallel to this major fault zone other smaller faults were observed. These faults have an Eastern inclination towards the river bed. The inclination of the rock, their open jointed and broken characteristics lead to unstable conditions which could trigger landslides of the decomposed rocks as well as the movement of larger rock masses towards the valley. The small village of Apliki just upstream could be considered to have been facing real danger from sliding if the dam would be built at this site. The observation wells carried out revealed decomposed rock to big depths covering extensive areas of the proposed damsite which would lead to serious leakage. Even the stability of the dam on such foundations could not be ensured, especially when raising the water level in the valley after building the dam, when the conditions for stability would further diminish.

Data

i.	*Catchment*	
	Area	8 km^2
	Average rainfall	750 mm/a
	Average runoff	2 million m^3/a
	1/1000 years flood	23.17 m^3/s
	Maximum height	1,234 m
	Maximum length	5.18 km
ii.	*Reservoir*	
	Area	11 ha
	Capacity	620,000 m^3
	Live storage	600,000 m^3
	Length	960 m
iii.	*Dambody*	
	Structural height	37.69 m
	Height above ground level	32.56 m
	Hydraulic height	30.43 m
	Depth of foundation cut-off	5.13 m
	Freeboard	2.13 m
	Crest length	132 m
	Top thickness	3.05 m
	Base thickness	27.4 m
	Upstream slope	1:0.1
	Downstream slope	1:0.7
	Total concrete fill	39,000 m^3
iv.	*Excavations*	
	Total	13,625 m^3
v.	*Foundation Treatment*	
	Total drilling depth for grouting	1,153 m
	Total grout take	12,128 kg
	Average grout take	10.6 kg/m
vi.	*Spillway*	
	Size	9.75 m
	Capacity	62.87 m^3/s
	Length	45.4 m
vii.	*Inspection Gallery*	
	Size	2.13x1.52 m
	Length	100.6 m
viii.	*Outlet Gallery*	
	Size	1.83x1.52 m
	Capacity	12.74 m^3/s
	Length	22.55 m
	Operating gate size	2.18x1.75 m
ix.	*Outlets (steel pipes)*	
(i)	*Irrigation*	
	Size	0.3 m dia
	Capacity	0.12 m^3/s
	Length	36.6 m
(ii)	*Compensation*	
	Size	0.2 m dia
	Capacity	0.085 m^3/s
	Length	23.6 m
x.	*Distrubution System*	
(i)	*Main Conveyor (concrete channels)*	
	Size (Rectangular)	0.56x0.40 m
	(Trapezoidal)	(0.63+0.30)x0.40 m
		(0.56+0.20)x0.40 m
	Capacity	0.08 — 0.12 m^3/s
	Length	7,320 m
(ii)	*Syphon (steel pipes)*	
	Size	0.15 m dia
	Capacity	0.12 m^3/s
	Length	300 m

CHAPTER IV

ROCKFILL DAMS

INTRODUCTION

Rockfill dams are designed and their selection is based on similar criteria as for earthfill dams. The choice for a rockfill dam instead of an earthfill dam is governed by:-

(i) Sound rock availability as a fill material.

(ii) Good rock foundation conditions.

Table 1 shows that up to now four rockfill dams have been built including the highest dam in Cyprus at Lefkara of 74 m. height. All these dams have been built in the igneous region of Cyprus on the Diabase Formation and the rock used for the fill was also diabase. In all cases the possibility for concrete gravity dams existed and a comparison was made between rockfill and a concrete gravity dam construction, which showed rockfill to be cheaper. It was fortunate that the availability of clay near the site enabled a clay core to be introduced with economic advantage, for otherwise upstream concrete or other type of upstream protection would in some of the cases weigh in favour of concrete gravity dams. Looking to the future, a small number of rockfill dams are expected to be built in addition to what we have. All such dams will be at high altitudes, over 500 m asl, where sound diabase rock exists. They will all be around the Troodos Igneous massif and will have to prove cheaper than concrete dams. On the North Range of the Island where the Hilarion Limestone exists, no suitable rivers or sites are available for rockfill dams. The only other rock in Cyprus which would be suitable for rockfill work is the Gabbro again on the Troodos massif. Gabbro is not extensively found in Cyprus and its application would be very limited.

Due to the non availability of clay in some areas, the introduction of upstream concrete, asphalt or other impermeable blanket should not be excluded for some of the future rockfill dams.

Table 10 gives the main properties of the materials used for the construction of the existing Rockfill Dams and design data.

Lefkara rockfill dam under construction.

ENGINEERING PROPERTIES OF FILL MATERIAL USED IN EARTH AND ROCKFILL DAMS

TABLE 10

Type of dam	Name of Dam	Slopes of dam: Upstream	Slopes of dam: Downstream	Clay core: Liquid limit %	Plastic limit %	Plasticity index %	U.S.C.S.	Compaction characteristics: Max. dry density t/m3	Optimum moisture content %	Coefficient of permeability cm/sec	Triaxial shear strength character: Cu Kg/cm2	φu	Filter material: Type of fill	Compaction characteristics: Max. dry density t/m3	Optimum moisture content %	Shell of rockfill material: Type of fill	Compaction characteristics: Max. dry density t/m3	Optimum moisture content %
Zoned rockfill	Argaka-Magounda	1:1.5	1:1.75	85	46	39	MH	1.15	43.0	2.0×10^{-8}	1.51	6.0	Sand & gravel	2.20	8.9	Diabase rockfill	—	—
	Ayia Marina	1:1.5	1:1.6	41	27	14	ML	1.57	23.0	7.9×10^{-8}	1.68	18.0	Sand & gravel	2.20	8.9	Diabase rockfill	—	—
	Pomos	1:1.3	1:2.0	41	27	14	ML &	1.57	23.0	7.9×10^{-8}	1.68	18.0	Sandy gravel	2.23	8.8	Diabase rockfill U/S Weathered diabase and terrace material D/S	2.24	7.5
				untestable			SC	1.94	17.6	7.3×10^{-6}	0.77	27.0					—	—
	Lefkara	1:1.5	1:1.6	38	22	16	CL	1.87	15.0	6.6×10^{-8}	1.51	13.0	Terrace Material: silt, sand, cobbles	2.03	11.5	Diabase Rockfill	—	—
Homogeneous rolled clay fill	Geunyeli	1:3.0	1:2.0	44	28	16	ML	1.63	19.3	0.8×10^{-8}	1.48	8.0	Sand & gravel	2.32	6.6	—	—	—
	Kanli Keuy	1:3.0 & 1:2.75	1:2.0	46	28	18	ML	1.64	19.3	1.7×10^{-7}	1.48	8.0	Sandy gravel	2.12	8.4	—	—	—
	Liopetri	1:3.0	1:2.0	39	28	11	ML	1.62	20.9	3.8×10^{-7}	1.68	17.5	Sandy gravel	2.10	7.0	—	—	—
	Syngrasi	1:2.5	1:2.0	50	29	21	ML	1.67	19.8	2.9×10^{-7}	1.05	3.7	Sandy gravel	2.10	7.0	—	—	—
Zoned earthfill	Athalassa	1:2.5	1:2.0	71	31	40	CH	1.45	29.8	5.7×10^{-8}	1.16	10.0	—	—	—	Sandy gravel	2.24	8.3
	Morphou	1:2.5 & 1:2.25	1:2.0 & 1:1.75	40	20	20	CL	1.49	19.8	6.5×10^{-8}	1.41	18.4	—	—	—	Sand & gravel	2.29	8.2
	Prodhromos (Reservoir)	1:2.0	1:2.0	52	31	21	MH	1.67	21.8	6.1×10^{-8}	1.70	11.0	—	—	—	Weathered diabase	--	—
	Mia Milia	1:2.5	1:2.0	44	28	16	ML	1.43	19.6	0.2×10^{-8}	1.48	8.0	Sandy gravel	2.10	6.0	Silty sand, gravel	1.96	13.8
	Ovgos	1:2.5	1:2.0	43	27	16	ML	1.68	17.2	3.6×10^{-7}	1.58	8.0	Sand & gravel	2.29	8.2	Silty sand, gravel	2.01	10.7
	Agios	1:2.0	1:1.6	50	26	24	CL	1.59	21.8	1.1×10^{-7}	1.62	13.4	Gravelly sand	2.21	10.2	Sandy gravel U/S Gabbro rockfill D/S	2.21 --	10.2 —
	Kiti	1:2.5	1:2.0	46	28	18	ML	1.60	25.1	1.4×10^{-7}	0.52	25.0	Sand	—	—	Silty sand, gravel	1.68	20.9
	Polemidhia	1:2.0	1:2.0	38	24	14	CL	1.62	18.1	6.2×10^{-8}	1.16	5.0	Sandy gravel	2.03	6.3	Sandy gravel with cobbles Sandstones, clay, silty sand	1.95 1.81	10.6 16.1
	Kalopanayiotis	1:2.5 & 1:2.2	1:2.0 & 1:1.75	34	23	11	CL	1.89	14.8	2.0×10^{-7}	1.20	16.5	Sand & gravel	2.2	8.0	Silty sand, gravel	1.91	11.2
	Mavrokolymbos	1:3.0 & 1:2.0	1:1.75	36	20	16	CL	1.77	16.2	1.2×10^{-7}	1.27	10.0	Sandy gravel	1.94	5.0	Silty sand to sandy gravel	1.92	12.5
	Yermasoyia	1:3.8 & 1:2.0	1:1.8	41	22	19	CL	1.77	17.0	1.5×10^{-7}	1.41	20.5	Silty sand Sand & gravel	1.92 2.13	10.3 8.1	Gravel U/S Sandstones, clay, silty sand D/S	2.17 1.83	8.0 14.5
	Masari	1:2.5 & 1:2.25	1:2.0 & 1:1.8	44	22	22	CL	1.70	19.3	2.3×10^{-7}	1.65	13.0	Sand & gravel	2.19	7.5	Sandy gravel & cobbles	2.28	6.0

ARGAKA

The Argaka dam.

MAGOUNDA—ARGAKA PROJECT

1. PURPOSE

The need for additional irrigation that would result to a higher income of the Tylliria region, which is one of the poorest in Cyprus, was the main consideration for the decision taken to build the dam. The people of the two villages which would benefit, Argaka and Magounda, were mainly occupied in mining works at the nearby Limni Copper Mines. Some of these people, especially villagers from Argaka, who own the fertile coastal region and where limited groundwater could be cheaply extracted, were practicing irrigation of early spring vegetables such as tomatoes, cucumbers and melons under cover of polythene or glass. The advantage of early crops which have high returns as well as the interest of the farmers to extend this practice and the fact that no more water could be extracted from the gravels, were additional reasons for constructing the dam. Lastly, the interest of the Mines to maintain their groundwater supply from the coastal aquifer for their own mining works would not allow any more water to be extracted for irrigation. Thus the construction of the dam which would impound a considerable portion of the surplus to the sea, would be used for the extension of irrigation in the area, and would also help in maintaining and strengthening the water supply requirements of the Mines.

Soil and climatological investigations have shown, that apart from the early vegetables which would be grown at the coastal strip, citrus and bananas could be grown inland and especially in the weather protected river valley from the dam going downstream.

2. LOCATION

The dam is built on the Magounda river at a height of about 65 m asl and at a distance of about 3.7 km upstream from the coast.

3. PLAN

a. Water and Land

The average rainfall on the catchment 670 mm/a produces an annual average runoff of about 8.6 million m^3. The maximum flood estimated at the damsite at a frequency of 1/1000 years is about 280 m^3/s. Certain water right uses were exercised by an Irrigation Division through an intake and a system of concrete and earth canals from the Magounda river. These channels were used for the spate irrigation of about 2,400 donums of cereals, cultivated on lands belonging to the two villages. With perennial irrigation practiced, about 1,200 donums could be irrigated with the available water from the dam cultivating mainly citrus, cucumbers melons and tomatoes. Important water right use for the water supply of the Mines claimed by the Limni Mines had to be secured by agreement with the Government for safeguarding a supply from the groundwater of 1.5 million m^3/a. As the groundwater is extracted from the coastal alluvial aquifer which is directly recharged from the river, a spreading ground was built downstream of the dam to facilitate infiltration. Also relocation of the boreholes of the Mines was done so that those very near the coast which extracted water with high salt content, would be replaced by other boreholes further inland.

To enable this quantity to be satisfied in dry years a 0.30 m dia steel pipe outlet has been provided through the dam, and spreading grounds downstream of the dam to recharge the coastal aquifer were constructed.

b. Geology

The whole reservoir and catchment lie on igneous diabase rocks. Talus scree and whethered rock to a depth of up to 5 m was found on either abutment. Investigation holes were drilled before construction to determine the depth of this talus. The rock on the left abutment was found to be tighter than on the right whilst the base rock itself was found to be quite impermeable.

The depth of the sand and gravel alluvium was about 8 m which was wholly excavated. The very sound diabase rock on the spillway site allowed for a conservative concrete lining made up of two deflecting buckets only with the necessary retaining walls.

One important aspect was a recent forest fire on the catchment which had left considerable remnants of timber logs and debris in the catchment just above the dam which would result to heavy erosion and a reduction of the reservoir capacity.

4. MAIN FEATURES

a. Dam

The detailed design of this dam was made by Howard Humphreys and Sons of London in 1963. The embankment is formed of a clay core in the centre with a gravel and sand filter zone on either side and diabase rockfill zones on the upstream and downstream sides.

A spillway built on the right abutment with two concrete lined deflector buckets was provided which could accommodate a flood of 280 m^3/s.

An outlet tunnel 2.15 m dia was excavated through the left abutment and lined in concrete throughout. This has been provided with an upstream hydraulically operated panstock. In the tunnel the irrigation pipeline and the Limni Mines compensation pipeline have been housed. The irrigation outlet pipe is connected to an upstream hydraulically operated valve.

b. Distribution System

The distribution system designed by the WDD consists of the main pipe conveyor and the secondary and tertiary pipe network. The whole area is divided into 26 irrigation blocks each one provided with a sluice valve and a water meter at the higher elevation of the block.

5. CONSTRUCTION

The construction was undertaken through bilateral agreement with the Government of Cyprus and a consortium of British Contractors Mowlem and Ridgeways between April 1963 and February 1964. An extension of the spillway by constructing a second deflector bucket was constructed by the WDD later in 1964.

All materials except the concrete aggregates were obtained from the site.

The compaction of the clay core was done either by sheepfoot rollers or by dump trucks followed by vibrating rollers with similar results. Limited grouting was carried out both along the dam axis, under the spillway foundation and in the tunnel.

A total of 1,950 m^2 were grouted in the rock under the embankment foundation. The grout injection was carried out through 50 mm dia holes drilled at 3.3 m intervals along the axis. The maximum depth of these holes was 18 m. The grout pressure applied was twice the depth of the stage being grouted. Injection was continued until the pressure drop in two minutes after closing the cock on top of the hole, did not exceed 0.33 of the injection pressure. The total cement injected was about 100 tons.

The tunnel was grouted from three rings of eight holes each, one at the centre line, one 3.3 m upstream and the other 3.3 m downstream of the centre line. The grouting was made through 37 mm dia holes to depths of 6 m into the rock and at pressures not exceeding 4.22 kg/cm^2. Contact grouting throughout the tunnel was also applied. The total cement consumption in the tunnel grouting was only 3 tons. The total area grouted was 980 m^2 at a total cost of £1,800.

The spillway foundation was not found to require grouting. The total cost of the drilling and grouting works was about £10,000.

The total cost of the dam was £270,000 including £20,000 works carried out by the WDD.

The spreading grounds built downstream of the dam as well as the 0.30 m dia compensation pipeline placed through the dam for the Limni Mines groundwater requirements were constructed by the WDD in 1965 at a cost of £3,700.

Work on the piped distribution system started in April 1972 and completed by the end of 1973, at total cost of £128,000. Before 1972 the water was utilised through the existing earth channels.

6. MANAGEMENT

This project has been constructed as a Government Project to be operated by the Government and water is sold to the members of the Argaka — Magounda Irrigation Division at 13 mils/m^3 which represents 50% of the investment cost paid over 50 years at 7% rate of interest and including operation and maintenance costs.

7. PROBLEMS OF SPECIAL INTEREST

One important problem which cropped up was the water rights which were claimed by the Irrigation Division. Many fruitless investigaions, discussions and legal proceedings followed up and the matter was pending before the Court for about 5 years. This problem, with the fact that some groundwater for irrigation is available in the region and alternative employment is many times available at the Limni Mines, resulted to a very inadequate utilization of the reservoir water in the early years.

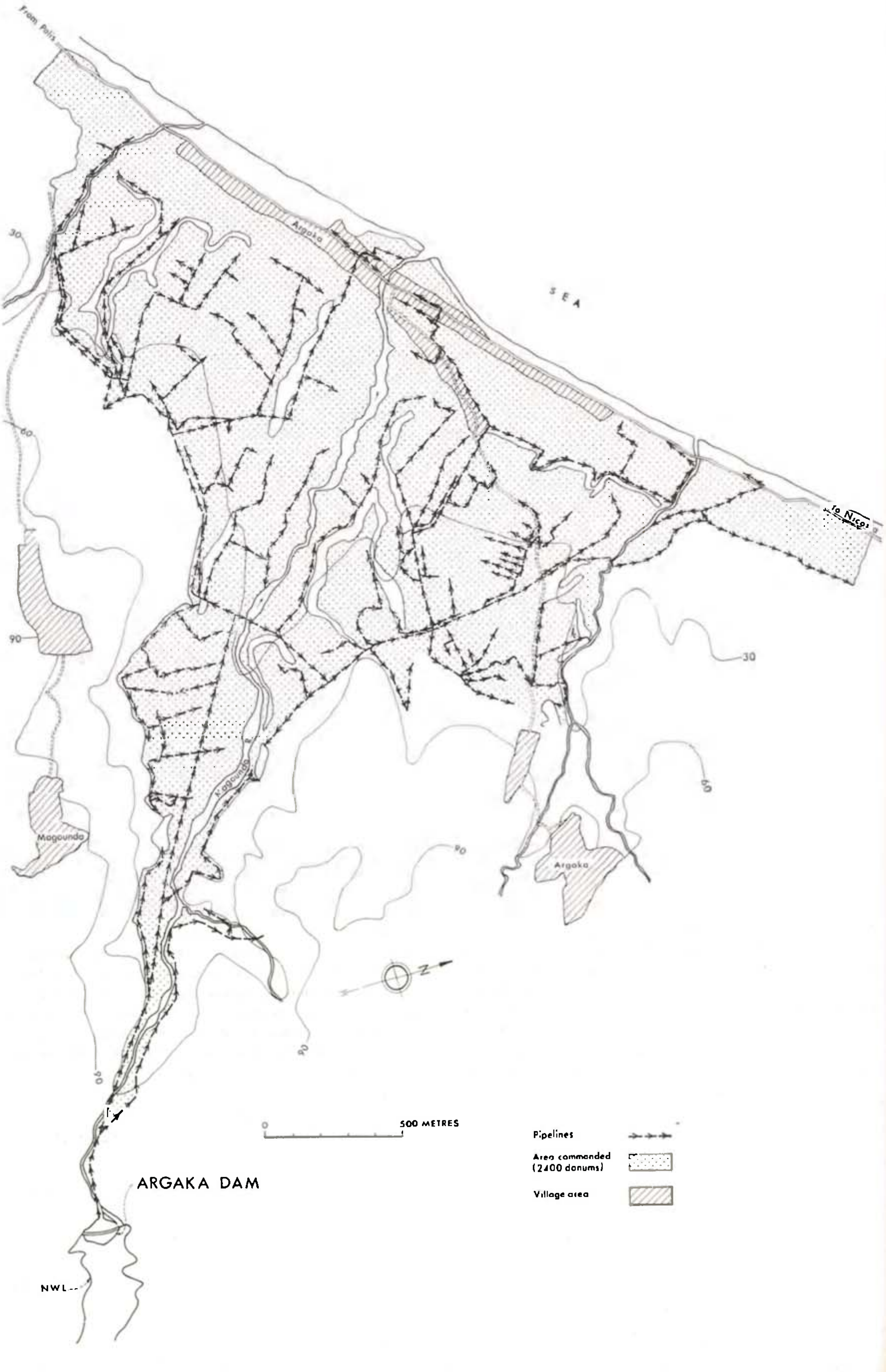
Finally, agreement was reached and with the construction of the whole distribution system in 1973 it is believed that the Dam water will be properly utilized for irrigation.

Another matter which received special attention was the water supply requirements of the Limni Mines and the groundwater recharge undertaken.

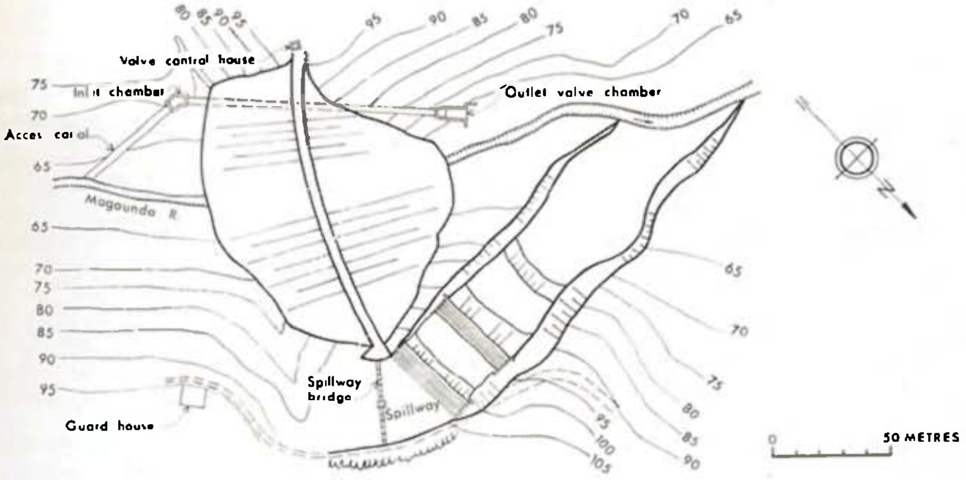
Observations taken in 1965 on the water supply boreholes of the Mines after the 1964 first filling of the reservoir showed that out of the seven boreholes available the three best ones mostly used by the Mines showed increased salinity of 117 p.p.m., 121 p.p.m. and 104 p.p.m. of NaCl as compared with previous years' records which did not give salinities exceeding 90 p.p.m. NaCl. Those nearest to the coast showed salinities up to 1,000 p.p.m. and water levels up to 17.5 m below sea level.

Following these results a decision was taken to construct the spreading ground downstream which was done in 1965 and to replace three of the coastal boreholes of the Mines with three other which were drilled in 1966 at suitable inland sites selected by the WDD. The result of the recharge through the spreading grounds was satisfactory eversince their construction in 1965 so that the new inland boreholes are left standing by to be used if required. Because of the satisfactory groundwater situation, it was not roquired to use the compensation pipeline to take water from the dam for the Mines.

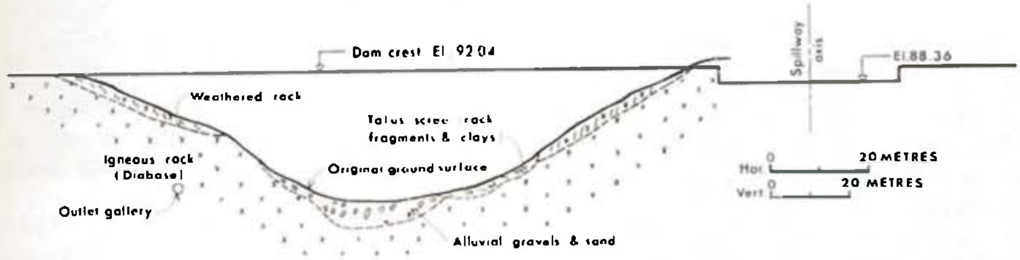
MAGOUNDA-ARGAKA-DISTRIBUTION SYSTEM



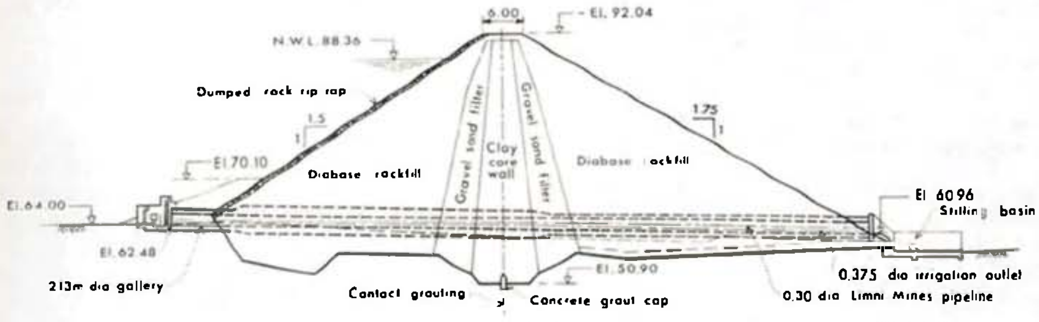
MAGOUNDA-ARGAKA DAM



PLAN

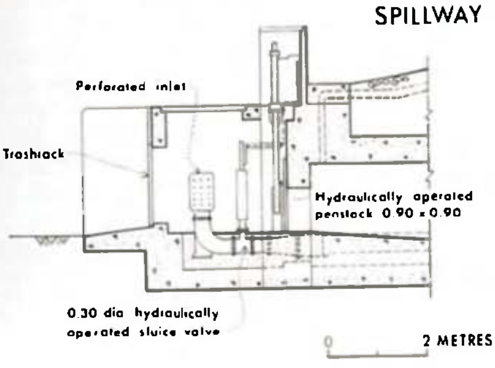


GEOLOGICAL SECTION ON DAM AXIS

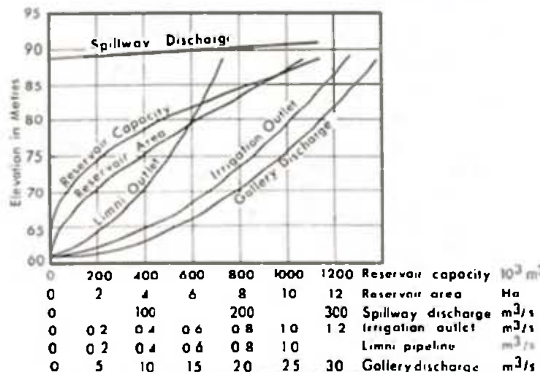


MAXIMUM SECTION

SPILLWAY SECTION



VALVE CHAMBER



CAPACITY-AREA-DISCHARGE CURVES

Data

i. Catchment

Area	50 km^2
Average rainfall	670 mm/a
Average runoff	8 million m^3/a
1/1000 years flood	280 m^3/s
Maximum height asl	1,080 m
Maximum length	13.8 km

ii. Reservoir

Area	10.7 ha
Capacity	1.15 million m^3
Live storage	1.10 million m^3
Length	960 m

iii. Embankment

Structural height	41.14 m
Height above ground level	28.04 m
Hydraulic height	24.36 m
Depth of foundation cut-off	13.10 m
Freeboard	3.68 m
Crest length	137 m
Top thickness	6.00 m
Base thickness	103 m
Upstream slope	1:1.5
Downstream slope	1:1:1.75
Upstream core slope	1:0.1
Downstream core slope	1:0.1
Minimum core thickness	2.7 m
Maximum core thickness	10.3 m
Rockfill	84,000 m^3
Clay core	32,440 m^3
Gravel, sand filter	17,310 m^3
Total earth fill	133,750 m^3

iv. Excavations

Total	120,500 m^3

v. Spillway

Size	37x3.68 m
Capacity	280 m^3/s
Length	146 m
Concrete	1,200 m^3

vi. Tunnel

Size	2.15 m dia
Length	0.35 m^3/s
Capacity	106 m
Concrete	1,090 m^3
Steel reinforcement	18 tons
Operating gate size	0.90x0.90 m

vii. Outlets (steel pipes)

(i) For irrigation

Size	0.375 m dia
Capacity	1.3 m^3/s
Length	107 m

(ii) Compensation pipeline

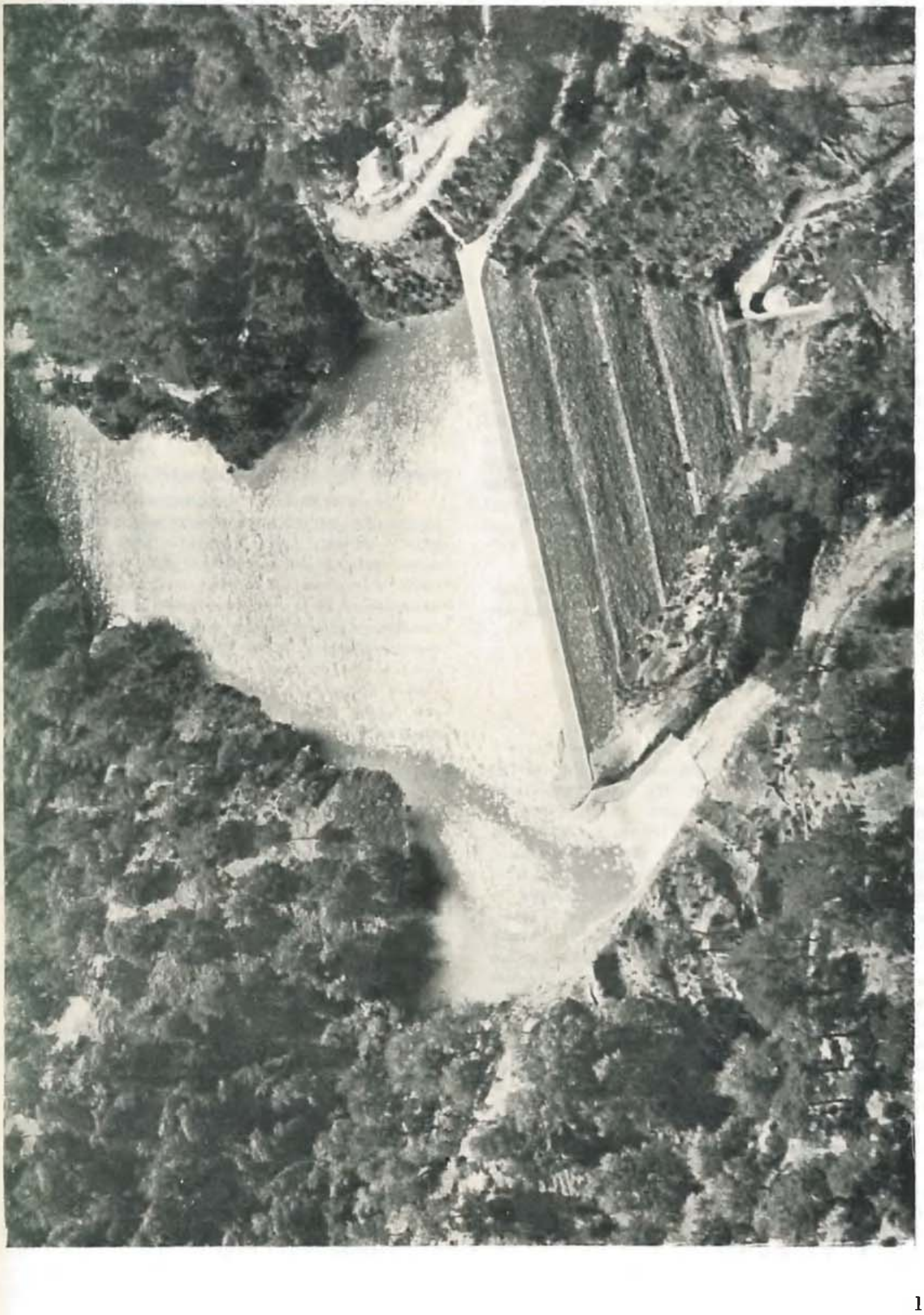
Size	0.305 m dia
Capacity	0.7 m^3/s
Length	130 m

viii. Distribution System

(i) Main Conveyor (A.C. Pipes)

Size	0.500 m dia
Capacity	0.38 m^3/s
Length	2,050 m

'AYIA MARINA



The Ayia Marina dam.

XEROS—AYIA MARINA PROJECT

1. PURPOSE

The Ayia Marina dam was built as part of the plan to develop the Tylliria region by supplying additional water supplies and enable the extension of irrigation.

The Xeros river itself is one of the smallest in Tylliria with little alluvial gravels, but with a relatively good coastal pleiocene aquifer which supplies water through shallow boreholes to a number of farmers of Ayia Marina village mainly for growing early summer vegetables and other crops such as tomatoes, cucumbers, melons and water melons. Due to overpumping, however, the aquifer started depleting and more water was required. This village has also a combined irrigation system from the nearby Yialia river with the neighbouring village of Yialia, irrigating mainly cereals and giving some irrigation water to their summer crops in the early season.

However, as important profits from early crops were being realised, and more people got interested, the need for a dam on the Xeros river was justified.

2. LOCATION

The dam is built on the Xeros river at an elevation of about 120 m asl and at a distance of about 2.5 km upstream of the sea.

3. PLAN

a. Water and Land

The average rainfall over the catchment is 475 mm/a with an estimated yield based on the rainfall data of 0.35 million m^3/a. The maximum flood estimated at the damsite of 1/1000 year frequency is estimated to be of the order of 160 m^3/s. Water was utilized from the Xeros river by the local Irrigation Division from a point just downstream of the dam from where a steel pipe conveyor was supplying water to the lands belonging to the beneficiaries. As, however, the flow was available during short periods in winter and spring time, no reliable crops could be grown based on this water only. The construction of the dam was therefore necessary to safeguard the supply of water during spring and summer months. The land which was selected was 1,500 donums that was already within the existing Irrigation Division. The available water from the dam can only irrigate 220 donums of perennial crops. As the irrigation practiced in the region, however, is mainly for early vegetables, the extent of cultivation annually can reach 500 donums. A few hundred donums are in addition irrigated from groundwater and a high percentage of rotation of the land is practiced. Winter crops requiring very little irrigation, such as cereals and broad beans are cultivated on the remaining land.

b. Geology

The whole catchment lies on igneous diabase rocks. The coastal region is made up of river and marine deposits which contain some ground water mainly recharged from the river and from direct rainfall.

The damsite itself is predominantly formed of well jointed sound diabase. The rock in the cut-off is mainly microgabbro which underwent chalcopyritic mineralization. The investigations carried out at the damsite included 9 test pits up to 6 m depth, 8 trenches on either side up to 4 m depth and one gallery 7 m long.

4. MAIN FEATURES

a. Dam

The detailed designs for this dam were made by Energoprojekt of Yugoslavia in 1963.

The embankment is formed of slightly inclined upstream clay core on either side of which filter zones of gravel and sand were provided. Next to the filter zones, upstream and downstream, there have been placed zones of random fill, whilst the main upstream and downstream zones were of good diabase rock fill. Handplaced quarry rip-rap was provided both upstream and downstream. A two floor access gallery was used, concrete cast, on excavated rock throughout. The lower gallery 1.52x(1.6—2.75 m) provided access to the gate and to the 0.30 m dia irrigation steel outlet pipe, whereas the upper gallery 2.13x1.52 m with a semicircular roof provides access to the mannually operating mechanism. The twin gallery reaches just upstream of the clay core from where the gate is operated. The lower gallery reduced in size to 1 m horse shoe shape gallery, and housing the irrigation outlet pipe extends to the upstream side. A wheel gate 1.3x0.9 m operated manually through a mechanism has been installed on the upper gallery. The total weight of the gate and its lifting mechanism is 1.5 tons and can operate under full hydrostatic head.

A steel transition lining 18 mm thick 2 m upstream and 1 downstream of the gate was fitted to protect the concrete walls of the gallery.

The spillway is on the right abutment with a mass concrete free overfall weir and a short concrete lined chute left well above river bed, as the rock has been considered very resistant to erosion and because overflows would be limited.

b. Distribution System

The design of the distribution system which is made up of a reinforced concrete main conveyor channel with secondary and tertiary network of steel pipes has been designed by the WDD.

5. CONSTRUCTION

This dam together with Pomos were the first to be internationally tendered and the successful constructor for both dams was Mediterranean-Zachariades a Greco-Cypriot Contractor. The work started for both dams in May 1963 and continued with the contracted agreement until February 1964. Thereafter construction was taken over and completed by the WDD. For Ayia Marina dam the Contractor did most of the work valued at £68,000 whereas the WDD did finishing works, including the spillway and rip-rap valued at £32,000. Materials for the fill were obtained from structural excavations or from quarries at the site, but the clay was brought to the site from Polis area 20 km away. The clay core was compacted in 0.15 m layers at about 22.5% moisture content and 1,500 kg/m^3 of dry density using sheepfoot rollers. The filter zones were compacted by vibrating rollers in 0.30 m layers at a moisture content of about 9% giving dry density of 2,080 kg/m^3.

The random fill was compacted by vibrating rollers in 0.45 m layers at a moisture content of about 6.5% with a dry density of 2,240 kg/m^3.

The wheel gate, hoist and operating mechanism were subcontracted to the Greek manufacturer Polemis-Drakos.

The total cost of the dam only including both contracted and Departmental work was £100,000.

The distribution system made up of the main channel conveyor and pipe network was constructed by the WDD between 1965 and 1968 at a total cost of £28,000. Some more tertiary system has still to be built for the water to reach all farms. To enable more efficient water application 400 donums were levelled by the Department of Agriculture at a cost of £6,000.

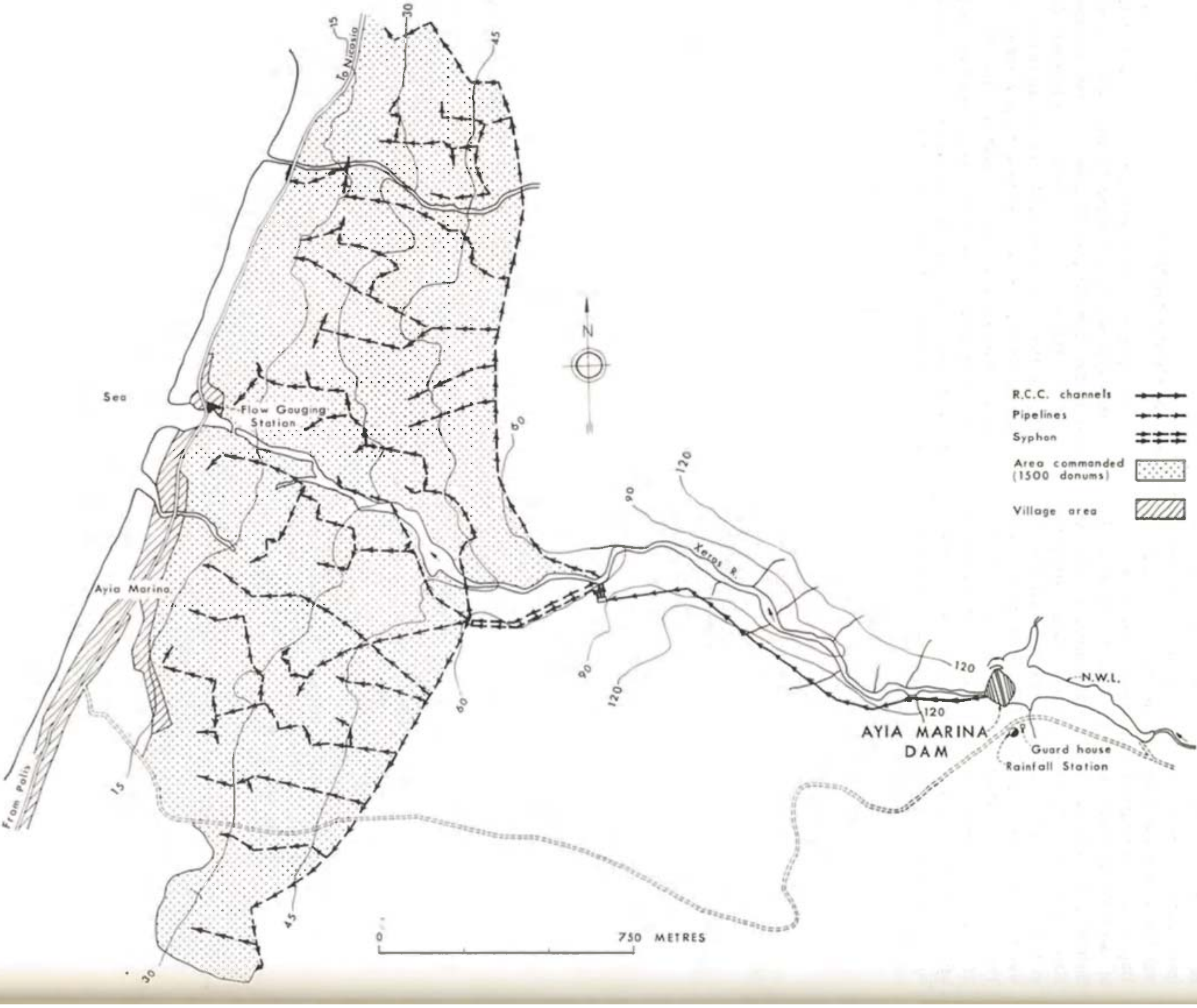
6. MANAGEMENT

This project was constructed by the Government for the Ayia Marina Irrigation Division which signed a loan to pay £36,667 over 20 years at a rate of interest of 5.5%. The beneficiaries later on preferred not to abide by this agreement, and another agreement was reached by which they undertook to buy water at 10 mils/m^3 from the dam and at 5 mils/m^3 from the overflow. With this arrangement the management of the Dam is taken over by the Government and operated through a District Water Committee.

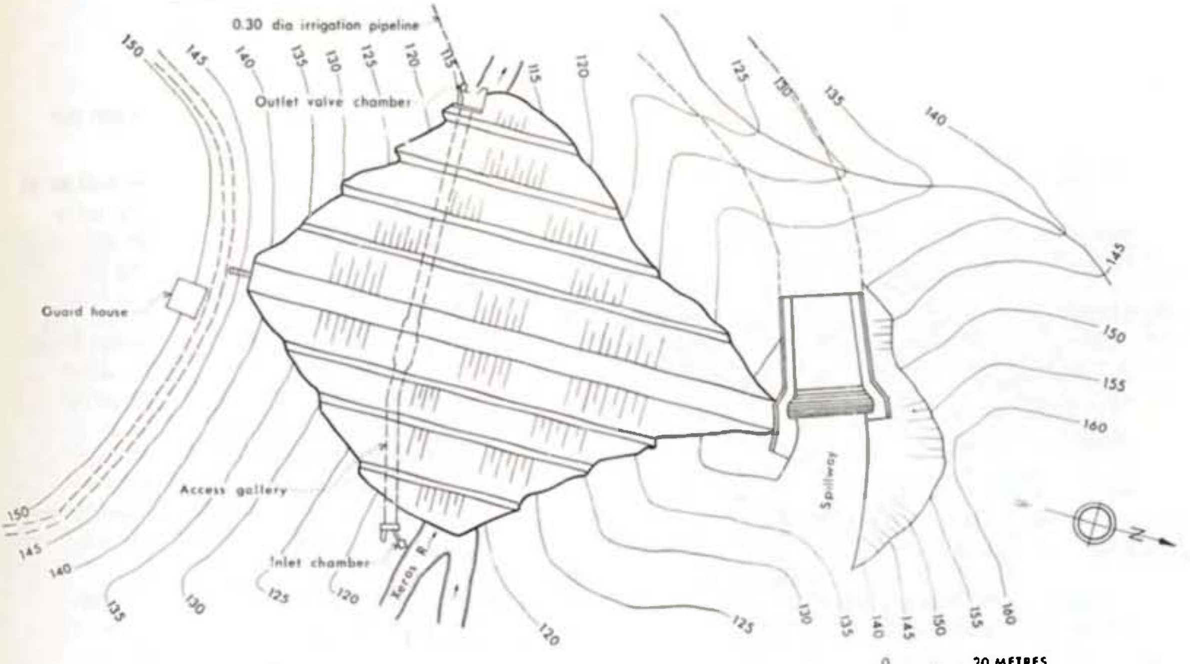
7. PROBLEMS OF SPECIAL INTEREST

No serious leakages or other defects were encountered. The only problem was that in dry years there was insufficient water to fill this dam. So a diversion scheme has been constructed at a cost of £23,000 which now conveys 0.02 m^3/s of water from a high tributary of the Livadhi (Pomos) river into the Ayia Marina Dam. The conveyor is of a composite steel pipeline 0.20, 0.15 and 0.10 m dia.

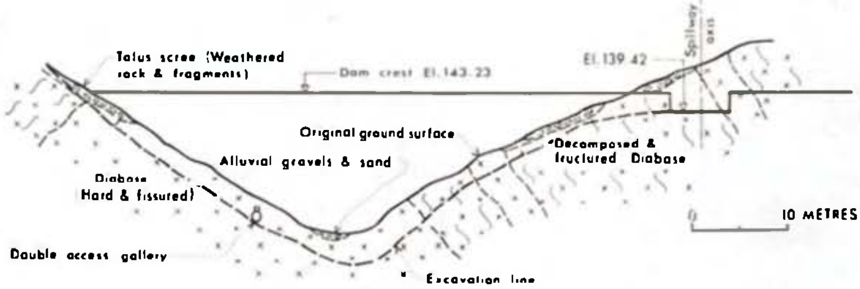
XEROS-TYLLIRIA-AYIA MARINA-DISTRIBUTION SYSTEM



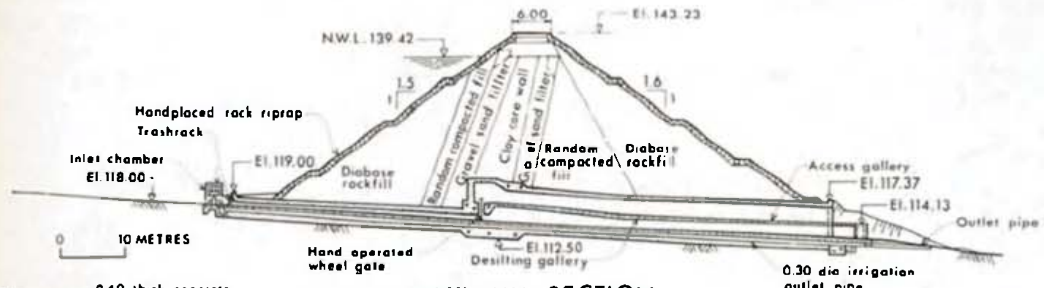
XEROS-TYLLIRIA-AYIA MARINA DAM



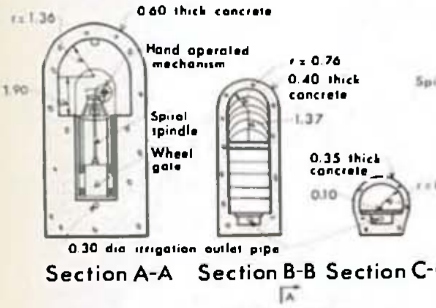
PLAN



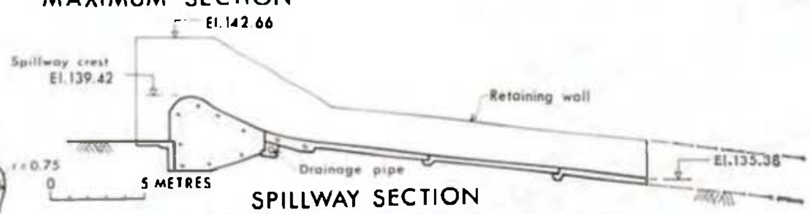
GEOLOGICAL SECTION ON DAM AXIS



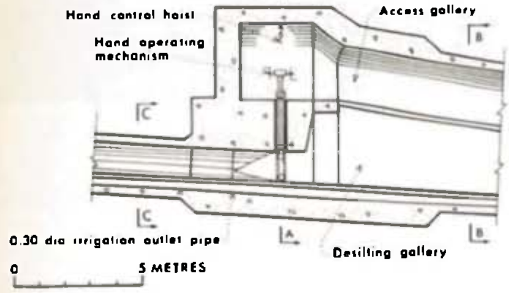
MAXIMUM SECTION



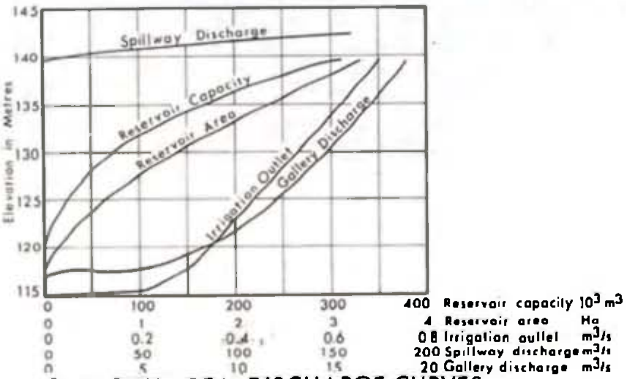
Section A-A Section B-B Section C-C



SPILLWAY SECTION



BOTTOM OUTLET VALVE CHAMBER



CAPACITY-AREA-DISCHARGE CURVES

Data

i. ***Catchment***

Area	8.4 km^2
Average rainfall	475 mm/a
Average runoff	0.35 million m^3/a
1/1000 years flood	160 m^3/s
Maximum height	580 m
Maximum length	2.4 km

ii. ***Reservoir***

Area	3.3 ha
Capacity	0.32 million m^3
Live storage	0.[illegible]0 million m^3
Length	518 m

iii. ***Embankment***

Structural height	30.73 m
Height above ground level	25.23 m
Hydraulic height	21.42 m
Depth of foundation cut-off	5.50 m
Freeboard	3.81 m
Crest length	116 m
Top thickness	6.00 m
Base thickness	93 m
Upstream slope	1:1.5
Downstream slope	1:1.6
Upstream core slope	1:0.4
Downstream core slope	1:0.24
Minimum core thickness	3 m
Maximum core thickness	7.16 m
Rock fill	29,000 m^3
Random fill	13,000 m^3
Clay core	8,180 m^3
Gravel, sand filter	10,000 m^3
Rock rip rap	990 m^3
Total earth fill	61,170 m^3

iv. ***Excavations***

Total	19,300 m^3

v. ***Spillway***

Size	18.30x3.80 m
Capacity	160 m^3/s
Length	26 m
Concrete	430 m^3

vi. ***Gallery***

Upper (Access)	2.13x1.52 m
Lower	3x1.52 m
Capacity (lower)	19 m^3/s
Length	90 m
Concrete	780 m^3
Steel reinforcement	33 ton
Wheel gate size	1.3x0.90 m
Weight	2.8 ton
Transition lining	
Thickness	18 mm
Length	3 m
Weight	3.4 ton

vii. ***Outlet*** *(Steel pipes)*

Size	0.30 m dia
Capacity	0.70 m^3/s
Length	90 m
Operating value	0.30 m dia

viii. ***Distribution System***

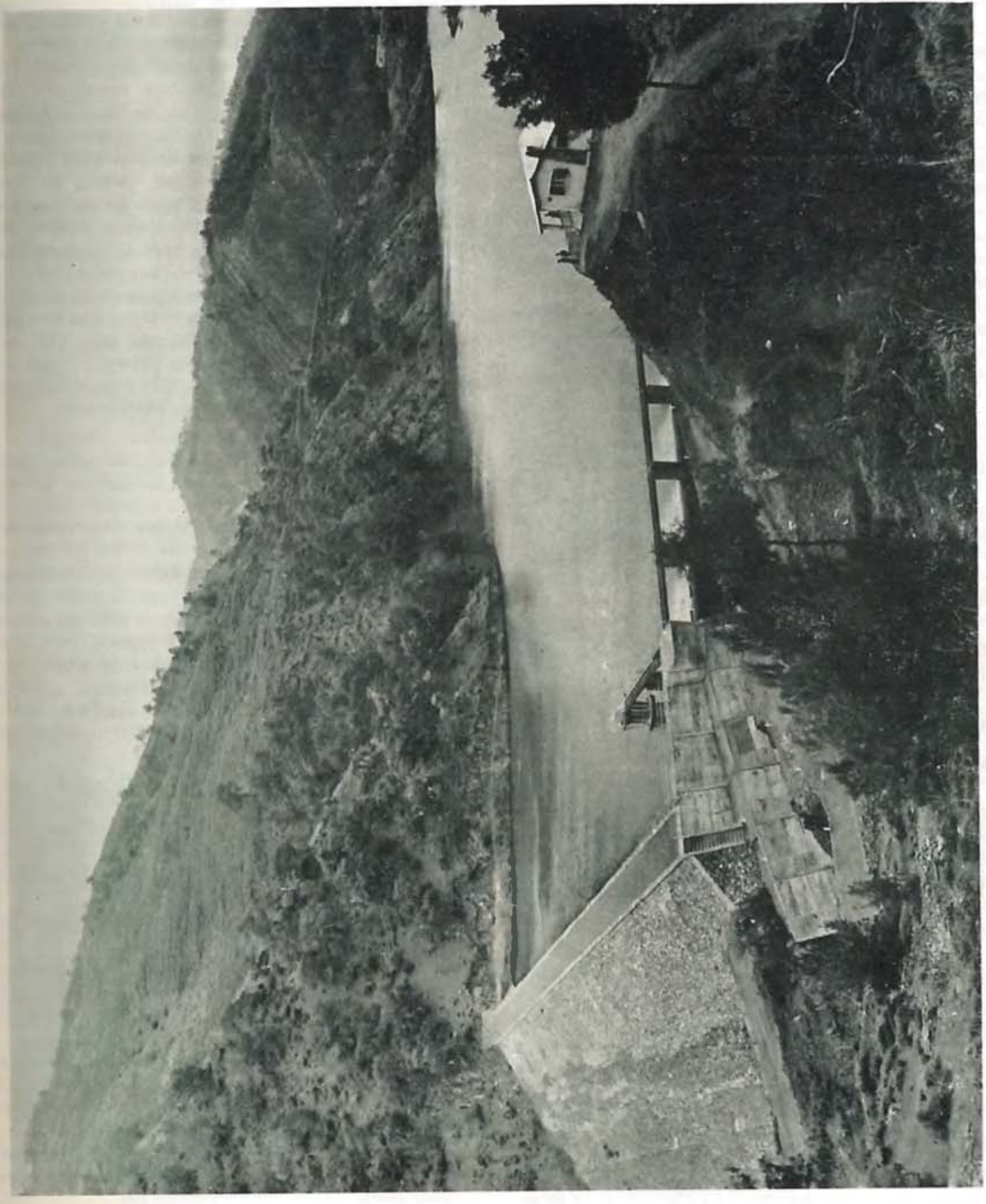
(i) ***Main Conveyor*** *(concrete channels)*

Size	0.28x0.23 m
Capacity	0.042 m^3/s
Length	1,330 m

(ii) ***Distribution Network*** *(Steel pipes)*

Size	0.06, 0.075, 0.10, 0.15, 0.20 m dia
Capacity	0.042 m^3/s
Length	14.5 km

POMOS



The Pomos dam.

LIVADHI—POMOS PROJECT

1. PURPOSE

Very little irrigation had been practiced in this Dam region, the main agriculture being the cultivation of dry farming crops such as almonds, olive and fig trees. A small diversion intake on the Livadhi could occassionally supply water for early spring vegetables to a few donums of land alongside the river valley. The groundwater availability at the region was insignificant and the irrigation practiced therefrom negligible.

The income from the dry farming crops is very little and many villagers had to travel long distances away from the village to work as labourers to supplement their income.

Similarly the two other small neighbouring villages, Paliambela village upstream and Nea Dhimmata village to the East of Pomos were facing the same problems of resources and low income.

Therefore the construction of a dam on the Livadhi river which discharged a lot of water to the sea annually, would contribute to a major extent to the increase of the income of these people.

This dam also forms part of the water development plan of Tylliria which provides for the development of the water resources of the region for irrigation.

2. LOCATION

The dam is built on the Livadhi river at an elevation of about 75 m asl and at a distance of about 3.5 km from the coast.

3. PLAN

a. Water and Land

The average rainfall over the catchment is about 550 mm/a yielding an annual average runoff of 5.8 million m^3. The figure for the runoff has been mainly based on rainfall data as the available flow recorder had insufficient data. The maximum flood estimated at the damsite of a frequency of 1/1000 years is of the order of 215 m^3/s.

The water utilized from this river prior to the construction of the dam was very limited and was confined to a narrow strip alongside the river downstream of the dam. The existing Irrigation Division, before the construction of the dam could not operate properly as the water available was insufficient. The new Irrigation Division formed, covers 2,850 donums to be irrigated from the dam. The available land had been soil surveyed to select the best land and the 2,850 donums selected has been land levelled as most of the terrain was quite steep. With the available water, however, only 1,415 donums are estimated to be irrigated permanently so a lot of the land will be irrigated by rotation which will help more people to be benefited. The main crops which can be grown in this region are citrus in the river valley and early summer vegetables, tomatoes, cucumbers, melons and water melons at the coastal plain.

b. Geology

The whole catchment lies on the Igneous Diabase Rock Formation. Only the narrow coastal region is formed of marine and alluvial deposits. The alluvial deposits at the damsite reach only up to 3 m depth lying on diabase rock. Several boreholes and trenches revealed the bed rock foundation at shallow depth. In the region of the tunnel the rock is predominantly quartz granophyre with several diabase dykes cutting it. The abutments on either side are made up in many places of decomposed and fissured rock with joints and minor faults associated with zones of brecciation and fracturing development to varying degrees. A fault on the bottom of the river on the right hand side is perpendicular to the striking of the rock fissures.

4. MAIN FEATURES

a. Dam

The detailed designs for this dam were carried out by Energoprojekt of Yogoslavia in 1963.

The embankment was designed to have an inclined upstream clay core which has a central part made of fine clay with one upstream and one downstream portion made up of boulder clay. Filter zones of gravel and sand were provided on either side of the clay.

Good diabase rockfill formed the main upstream fill and random fill the main downstream fill. Hand-placed diabase rock rip-rap was used both upstream and downstream.

A horse-shoe shape tunnel which was used for flood diversion during construction was provided through the rock on the left abutment. This tunnel was lined throughout and has an upstream access shaft for operating the tunnel for emergency purposes and for desilting during winter time.

A wheel gate manually operated was installed in the tunnel under the shaft for opening or closing the tunnel. The total weight of the gate and mechanism is 6 tons and is capable of lifting the gate under full hydrostatic head but it can close only under a head of water of about 10 m.

A transition zone lined with 16mm thick steel was provided 4 m upstream and downstream of the gate. Also a 0.30 m dia aeration pipe through the shaft was incorporated in the tunnel.

The irrigation outlet steel pipe placed in the tunnel was 0.40 m dia and is operated by a 0.40 m valve downstream.

The spillway was incorporated on the left abutment as a side spillway, with an access bridge over it resting on three piers and on either of the side walls.

b. Distribution System

The distribution system was designed by the WDD and is made up of a long main reinforced concrete rectangular channel cast-in-situ with pipe syphons at places. The distribution network is of steel pipes.

5. CONSTRUCTION

Pomos and Ayia Marina dams were constructed by the Greek-Cyprus Constractor Mediterranean-Zachariades who started work in May 1963. The contracted work was continued until February 1964 when the remaining works were taken over and completed by the WDD. The contractor executed work to the value of £132,000 whereas the WDD carried out completion works on the embankment, rip-rap and spillway lining to the value of £85,000.

The central part of the clay core fill was formed of good quality clay which was brought to the site from the Polis area 20 km away whereas on either side of it zones of clay with coarse material which was obtained from the site at the tail end of the reservoir was added. The clay core was compacted in 0.20 m layers by sheepfoot rollers with a moisture content of 22.5% and at an optimum dry density of 500 kg/m^3.

The filter zone material was brought from the coast, whilst all other materials, for the random fill, rockfill and rip-rap was excavated either from the structural excavations or from site quarries. The spillway was lined to a minimum length in the first instance in 1964 but subsequent fillings showed that the erosion was significant and this necessitated the extension of the concrete lining of the spillway which was carried out by the WDD in 1967. The hydromechanical equipment made of the wheel gate, hoist and operating mechanism were subcontracted to Polemis-Drakos of Greece. The excavations of the cut-off reached up to 6.5 m depth. On the right abutment the inflow of water from the disintegrated zone around the fault necessited some grouting before earth filling to the value of £2,250. The original contract provided for substantial grouting, but as the construction was late to start, it was decided to go on and fill in after minimum grouting in order to reach a safe height of the embankment before the onset of winter. The filling of the reservoir would then prove if grouting was required As a matter of fact the first filling proved this. Throughout 1965 regular measurements were taken of the leakage through the drains at the toe of the embankment and through the joints of the tunnel which was also not grouted. The leakage varied according to the water level in the reservoir. The maximum leakage at full reservoir head was estimated at 18 l/s from the embankment drains and toe and 1 l/s from the tunnel joints. These leakages were considered excessive and a decision was taken to go ahead with grouting of the tunnel and of foundations by drilling from the top of the embankment into the rock. A contract with Foundation Engineering of London in a joint venture with Joannou and Paraskevaides, Contractors of Nicosia started in September 1965 and completed in May 1966. The cost incurred on the grouting works of the grout curtain reached about £19,000 whilst that of the tunnel was £6,250. A single row of 47 boreholes 97 mm dia uncased and inclined at 20% to the vertical were scheduled to be drilled by percussion rigs from the crest of the dam through the clay core at 1.5 m centres to grout the rock. Finally only 31 holes were drilled some of them at 1.5 m and some at 3 m centres in 133 stages. Casing of 50 mm dia was used in the rock giving holes of 47 mm in nominal dia reduced where necessary to 44 mm dia. The total depth of drilling was 708 m though the clay core and 380 m through the rock. The grout mix used was cement 56% clay 38% and bentonite 6%. The total weight of solids used in the curtain was 81,500 kg made up of 69,800 kg of cement, 3,500 kg of bentonite and 8,200 kg of clay. The average consumption of solids was 214 kg/m.

The water: solids ratio varied between 1.1 and 5.1 starting with the lean mix. The depth of the grouted rock zone was between 9 and 18 m whilst a total area of about 1,230 m^2 is estimated to have been grouted.

The interface between the clay core and the rock foundation was treated carefully. Following the drilling of the clay core to rock, a 75 mm dia casing was installed at rock surface. A small quantity of quick setting but low strength grout of about 6% bentonite suspension in cement, was used for this work. The water pressures to detect leakage were applied to the top of the hole and were equivalent to the depth being between 3.16 kg/cm^2 and 9.14 kg/cm^2. Grouting was done if the loss after 5 minutes was greater than 45 l/m, otherwise the next stage of 3 m was drilled, but not more than two stages were drilled without grouting. After the grout was set in each stage, the hole was cleaned out and the grouting carried to the next stage. The process was repeated until the water loss in each hole was below the maximum permissible for a grouted hole. For the stage grouting the connection was always made to the hole on top of the rock. Control boreholes were drilled and tested at a pressure of 3.16 kg/cm^2. Where the consumption was not more than 0.2 l/m the grouting was considered successful.

Subsequent records of the leakage of the drains and toe after grouting showed that the leakage was reduced from 18 to 11 l/s at full reservoir head.

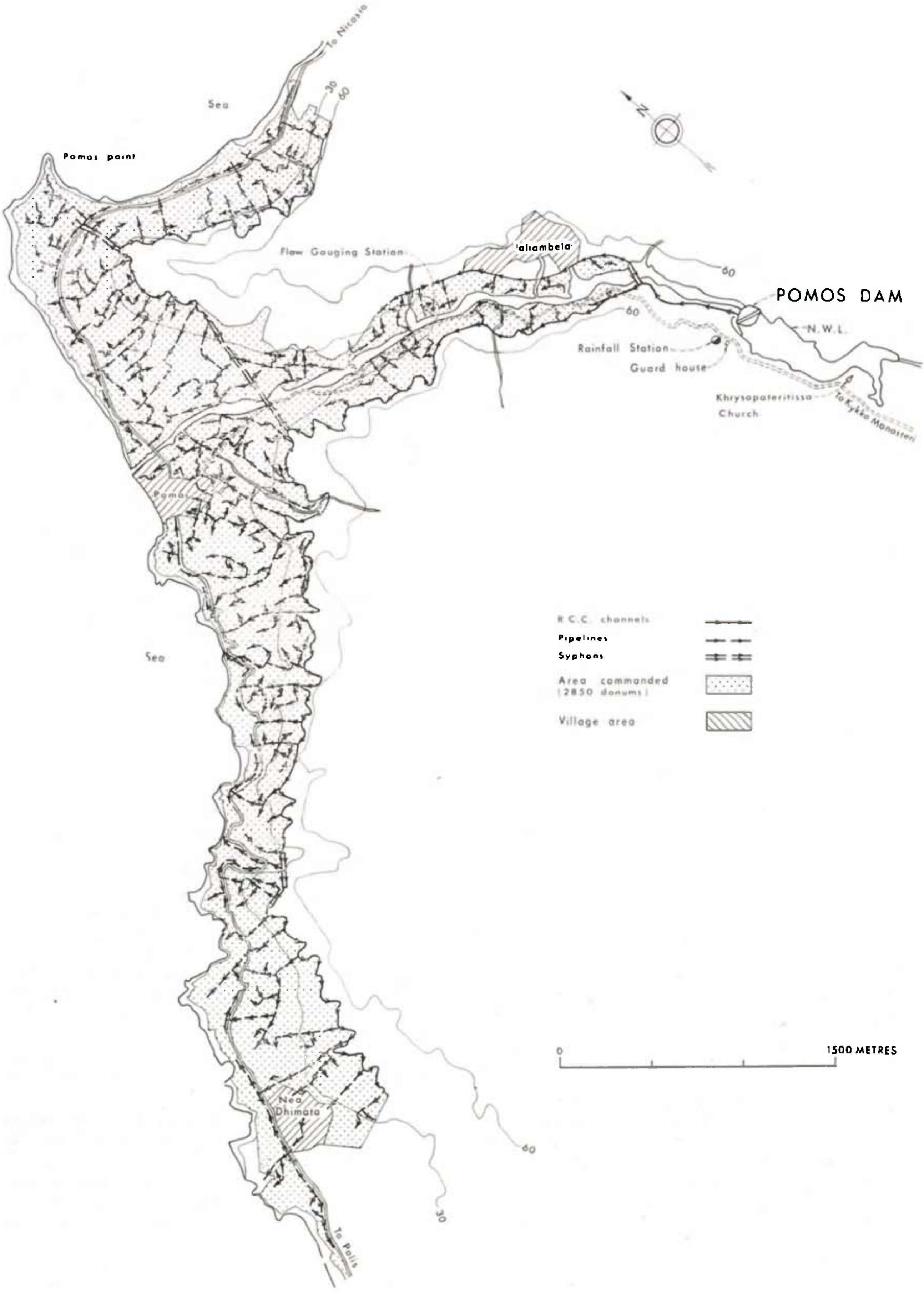
The grouting of the tunnel was done from many profiles along the tunnel through 3 drill holes 37 mm dia in each profile. The depth of the drill holes varied from 1.8 to 4.5 m and the mixture used was 1:3 to 1:0.7 cement water ratio beginning with the thinner mix. The pressure applied varied from 1 to 2 kg/cm^2. The grouting at each hole was considered satisfactory when the consumption under maximum pressure was less than 0.2 l/m for 15 minutes using 1:3 cement water mixture. Control holes were also drilled and the test was made using the same criteria.

The total drilling was 165 m and the total consumption was 22 tons through a total number of 120 holes.

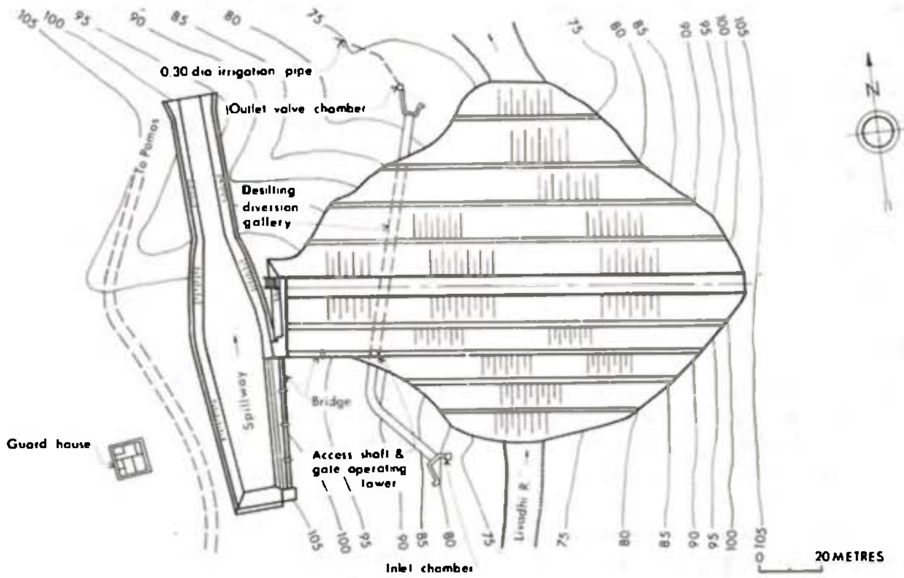
The access shaft was similarly grouted through 16 holes. The total depth drilled was 30 m and the total mixture consumed was 4 tons.

The result of the tunnel grouting was such that the leakage was reduced from 1 l/s before grouting to 0.2 l/s after grouting, in both cases at full reservoir

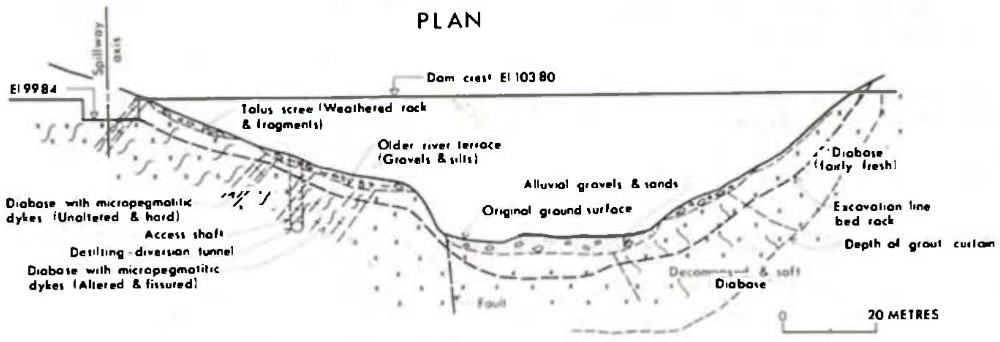
LIVADHI - POMOS - DISTRIBUTION SYSTEM



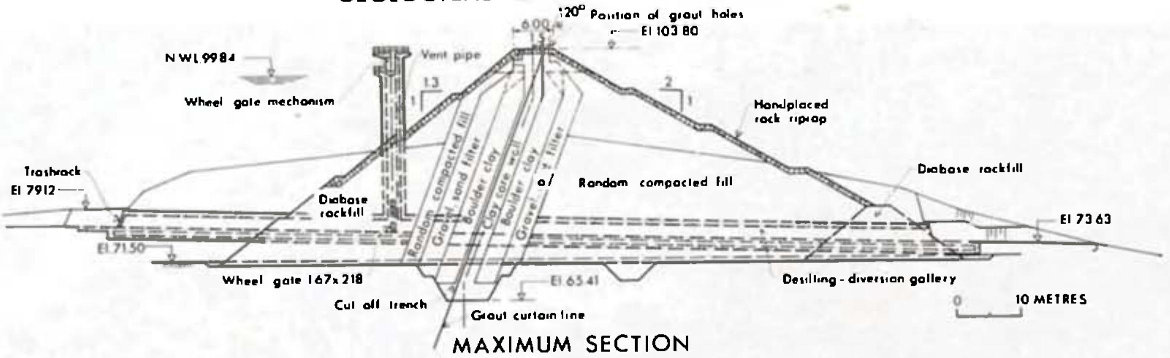
LIVADHI-POMOS DAM



PLAN



GEOLOGICAL SECTION ON DAM AXIS



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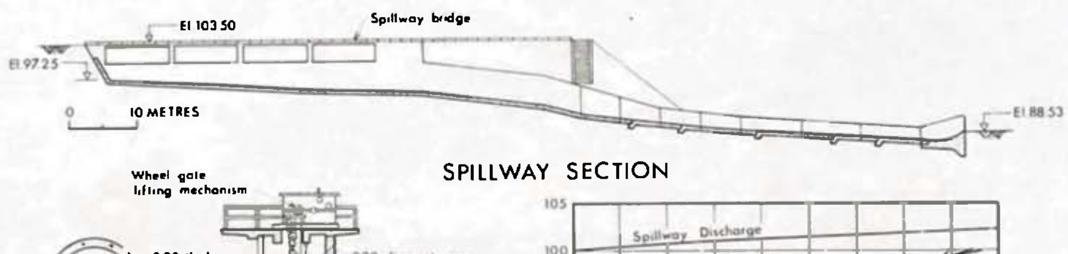
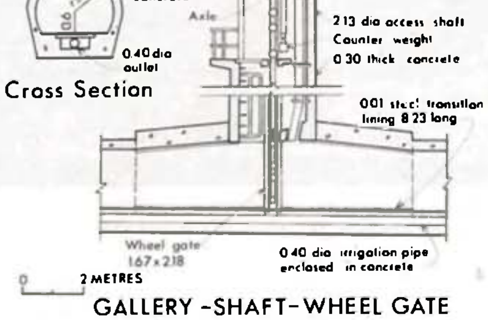
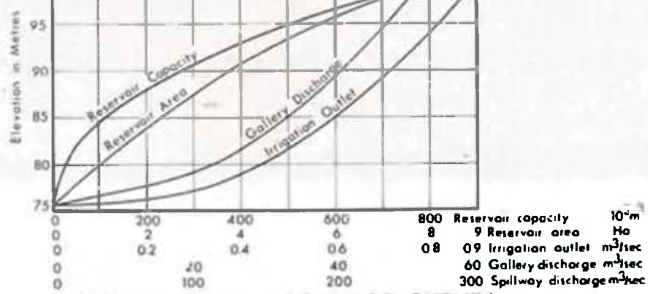


SPILLWAY SECTION



GALLERY -SHAFT-WHEEL GATE



CAPACITY-AREA-DISCHARGE CURVES

head. The total cost on the dam including the grouting works was £242,250.

The distribution system at a cost of £84,000 was constructed by the WDD between 1965 and 1966 and was made up of a long contour cast-in-situ reinforced concrete main channel 9,800 m long supplying water to many steel pipe laterals.

As the land was quite steep in certain areas, extensive land levelling was carried out covering about 2,000 donums at a cost of about £50,000 and which enables proper irrigation techniques to be applied.

6. MANAGEMENT

This project was constructed by the Government for the Irrigation Division of Pomos which signed a loan to pay £66,667 over 15 years at a rate of interest of 5.25%. The beneficiaries, however, insisted that Government should take over the Project and sell them the water per cubic meter. So an agreement was reached for the beneficiaries to buy the water at 10 mils/m^3 from the dam and 5 mils/m^3 from the overflow. For an accurate determination of the water sold it was decided to extend the distribution system by pipes to each private holdings and fix a water meter in every case. With this arrangement, the management of the dam is taken over and managed by the Government through a District Water Committee.

7. PROBLEMS OF SPECIAL INTEREST

An interesting problem during construction was the loss of water observed during rotary rig drilling for the foundation grouting from the top of the embankment.

The loss of water occurred first through an inclined hole drilled through the central good quality clay core and at a depth of 8 m from the crest of the embankment when water flushing the hole at a rate of 1.4 l/s. The hole was checked to be very nearly in the middle of the Polis clay core. The water pressure applied was only a little more than the hydrostatic head. Subsequently water loss was observed through all other boreholes drilled through the clay after a depth of about 9—10.5 m. Two test pits 12.5 m depth were then excavated in order to inspect the core visually as well as for taking undisturbed samples for laboratory tests.

The visual examination of the test pits revealed that the upper 6 m of the pits crossed through boulder clay and partly through the Polis good clay. The compaction was found to be satisfactory. The lower section of both pits passed through the Polis clay core completely. However, at some places, layers of the upstream boulder clay zone was found to intrude into the Polis clay. No open cracks or weak layers were found in either of the excavated pits.

The laboratory tests showed that the Polis clay is of high plasticity with a moisure content of 27.5% to 36% as compared with the optimum at Proctor of 25%. This was found to be a well compacted core with a permeability of 7.26x10—8 cm/s. In such brittle material some cracking of the core might be caused by uneven settlement on the rather steep abutments of the dam. The method of drilling by rotary rig and without casing in the clay was concluded to be reasonably good under

Pomos dam reservoir. — Upstream slide area.

the circumstances. The main disadvantage of not casing through the clay was that it softened the compacted material on a diameter of about 250 mm, leaving a pipe of loose and segregated material where the grout hole was. However, continuous casing would delay the work and be very costly. Due to the very fissured character of the diabase rock it was not possible to use packers for separating the grouting stages. It was therefore decided to grout through a pipe lowered to the bottom of the drilled hole which is plugged on top of the 50 mm casing. The grouted hole was then chopped and washed.

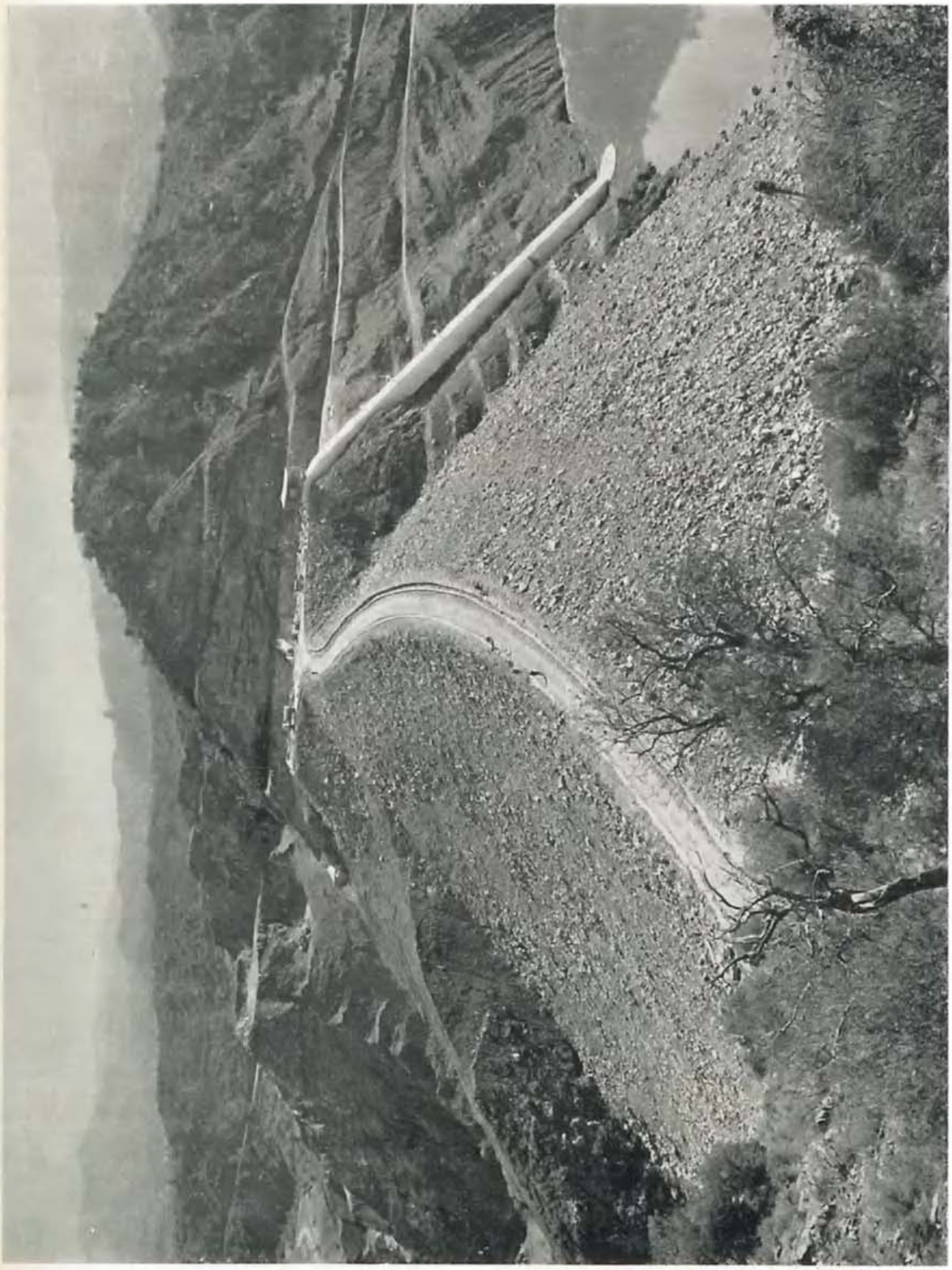
Because of the existence of the boulder clay layers traversing into the central core and to avoid diminishing the effectiveness of the core it was finally decided to stop drilling and grouting at 1.5 m centres, and instead increase the spacing at 3 m centres. Drilling was finally done using a mixture of water with clay and bentonite to replace flushing with water. Also the pressure in the rock was increased from a minimum of 3.16 kg/cm^2 to 5.27 kg/cm^2, whilst the filling up of the grout hole cavities in the clay core was carried out by low pressure grouting using a thick mix with sand in order to attain the previous rigidity of the disturbed zone around the boreholes and to prevent further erosion along the leakage path created by the water flushing.

Another post construction problem was the cracks in the reservoir which developed after the first impoundment in November 1966, on the right hand side at the tail end of the reservoir. Two slides appeared the lower one of which occurred because of the under-excavation for the borrow areas which rendered the slope unstable. This slide moved 3 m vertically with open cracks 0.9 m deep. This first slide triggered the second one immediately above it opening cracks up to 0.5 m depth.

Data

i.	*Catchment*	
	Area	36.3 km²
	Average rainfall	550 mm/a
	Average runoff	5.8 million m³/a
	1/1000 years flood	310 m³/s
	Maximum height	980 m
	Maximum length	12.05 km
ii.	*Reservoir*	
	Area	8.3 ha
	Capacity	0.84 million m³
	Live storage	0.80 million m³
	Length	900 m
iii.	*Embankment*	
	Structural height	38.39 m
	Height above ground level	32.30 m
	Hydraulic height	28.34 m
	Depth of foundation cut-off	6.09 m
	Freeboard	3.96 m
	Crest length	168 m
	Top thickness	6.000 m
	Base thickness	111 m
	Upstream slope	1:1.3
	Downstream slope	1:2
	Upstream core slope	1:0.37
	Downstream core slope	1:0.37
	Minimum core thickness	3 m
	Minimum core thickness	9 m
	Maximum core thickness	70,000 m³
	Rock fill	15,300 m³
	Random fill	14,600 m³
	Clay core	16,900 m³
	Gravel, sand filter	29,500 m³
	Rock rip rap	3,700 m³
	Total earth fill	150,000 m³
iv.	*Excavations*	
	Total	34,000 m³
v.	*Foundation Treatment*	
	Total drilling depth for grouting	380 m
	Total grout take	81,500 kg
	Average grout take	214 kg/m
vi.	*Spillway*	
	Size	4.5x12 m
	Capacity	280 m³/s
	Length	129 m
	Concrete	1,800 m³
	Steel reinforcement	15 t
vii.	*Gallery*	
	Size (Horse shoe shape)	0.6+1.2 m radius
	Capacity	52 m³/s
	Length	121 m
	Concrete	980 m³
	Permanent steel supports	9.4 t
	Steel reinforcement	23 t
	Wheel gate size	2.2x1.65 m
	Weight	11 t
	Aeration pipe Dia	0.30 m
	Length	23 m
	Transition lining	
	Thickness	16 mm
	Length	4 m
	Weight	10.8 t
viii.	*Outlet (Steel pipes)*	
	Size	0.40 m dia
	Capacity	0.58 m³/s
	Length	123 m
	Operating valve	0.40 m dia
ix.	*Distribution System*	
	Size	(0.35x0.23) to (0.60x0.35)
	Capacity	0.04 to 0.17 m³/s
	Length	9,800 m
(i)	*Main Conveyor*	*(Concrete channels)*
	Size	0.30 m
	Capacity	0.34 m³/s
	Length	850 m
(ii)	*Pipe Syphons*	
	Size	0.075, 0.10 & 0.15 m dia
	Capacity	0.014 m³/s
	Length	15,300 m

LEFKARA



The Lefkara dam.

PENDASKINOS—LEFKARA PROJECT

1. PURPOSE

The planning of the Famagusta Water Supply Project was carried out by the Department of Water Development.

Lefkara Dam is part of the Famagusta Water Supply Project and is intended to be the main source of domestic water for the Towns of Famagusta, Larnaca and regional villages. It is expected that the Project will satisfy the rapidly increasing water supply demands until the year 1980.

The dam is the second stage of water source development of the Famagusta Water Supply Project. The first stage was a groundwater development through boreholes and a subsurface dam on the Vasilikos river. After this dam another two dams on the two neighbouring rivers are planned to be introduced into the system. As an eventual source of supply and following the surface water development, sea water desalting will reinforce the system.

This project of the conjunctive use of ground and surface water resources of the region was actually compared with sea water desalting but has been found to be more economic in the ratio of 1 to 5.

Some brackish groundwater in the region is already blended with the fresh water whilst some more of it is under consideration to be desalted in the near future.

Within the plan of the development of the ground and surface water resources of the region an important part is the development of irrigation locally and this has started already from both sources. It is programmed to be phased together with the domestic water supply growth and at least an equal amount of water will be utilized for this purpose.

The damsite was chosen from several alternatives as being one of the best damsites in Cyprus, topographically and geologically. Its high elevation relative to other damsites has also been an important factor in its choice as it makes possible the conveyance of water to the Khirokitia Treatment Plant and from there on to Famagusta solely by gravity.

2. LOCATION

The dam was constructed on the Syrkatis tributary of the Pendaskinos river about 7 km from Lefkara village. It is situated at an elevation of 290 m asl.

3. PLAN

a. Water and Land

The catchment area of the Dam is about 36 km^2 with a long term average rainfall over the catchment of about 708 mm/a yielding an estimated annual average runoff of 8.2 million m^3. The probable maximum flood has been estimated at 930 m^3/s. The spillway design flood has been taken as 50% of the 930 m^3/s corresponding to a routed flood of 300 m^3/s. This design flood satisfies the 1/1000 years flood. Water was used by the Lefkara Irrigation Division prior to the construction of the Dam. The construction of the dam will make possible the provision of enough dependable water which was not possible without storage. The land commanded under the Irrigation Division is 615 donums out of which 430 donums are suitable to be planted in deciduous trees, citrus, and vegetables. The water to be used for irrigation will only be about 0.5 million m^3/a whilst that for domestic supplies will be 5.5 million m^3/a dependable supply.

b. Geology

The damsite lies on igneous diabase rock throughout. Alluvial deposits were virtually non-existent and bedrock was exposed in many places along the riverbed.

The diabase is sheeted with sheet thickness ranging from 0.30 to 1.50 m. The strike of the sheets is 40°—60° NE dipping at 60°SE except at the higher elevations of the left flank where the strike changes to 350°NW to 10° NE dipping at 70°W to 65°E. The diabase is intruded by a coarser grained gabbroic rock in the form of dykes usually parallel to the sheets. These dykes are generally associated with fracture zones at their contacts with the adjoining diabase sheets. Shear zones are also frequent at the sheet margins and these are associated with clay and polished surfaces. The diabase sheets are cut by joints at various altitudes and directions.

The surface rock, especially at the higher elevations is intensely weathered but the degree of weathering decreases with depth. Many of the bands of gabbro are weathered to a greater degree than the adjoining diabase sheets thus forming soft seams within the foundation.

Several boreholes were drilled and trenches excavated at the investigations stage. In addition, two adits were driven, one into each abutment.

The investigation boreholes were used for permeability tests of the rock. These revealed takes from 0 to 90 lugeons. The higher values were obtained at the higher elevations in both abutments while at the lower elevations values of over 10 lugeons were only found within 6 m of the bedrock surface due to the opening up of fractures and joints by stress relief.

With regard to constructional materials, rockfill was obtained from a quarry opened in the diabase near the damsite which was also used for filters and concrete aggregates after crushing. At the investigations stage it was envisaged that clay would be obtained from several borrow areas in white chalky deposits between the damsite and Lefkara village. These were investigated. An alternative clay material containing igneous fragments of rock within the reservoir area was finally used. A source of filter was investigated in the Pendaskinos river gravel downstream of the damsite near

Skarinou Station on the Nicosia—Limassol road. An alternative filter obtained from terraces in the river valley downstream was finally used.

4. MAIN FEATURES

a. Dam

The detailed design of the Dam was carried out by Messrs Howard Humphreys and Sons of the United Kingdom in 1970. It was designed as 74 m high rockfill dam with an upstream slope of 1:1.5 and a downstream slope of 1:1.6. The impervious element is a relatively thin central clay core flanked by transition materials on the upstream and downstream faces. No rip-rap was foreseen on the slopes other than the larger fraction of the rockfill.

A 250 m long 2.7 m dia concrete lined tunnel was provided in the right abutment to be used for flood diversion during construction and later as a scour outlet with 0.9 m dia pipe entry controlled by sluice valves. The tunnel was also to be used for the housing of the outlet pipework of the dam, which is designed to be supported near the crown of the tunnel.

At its upstream end the tunnel is connected to an inclined gallery via the circular valve chamber. The inclined gallery is constructed on the right flank and extends to dam crest elevation near the spillway where the access house is constructed. The inclined gallery replaces the conventional vertical tower and houses five intakes of the Drinking Water Supply pipe at different levels. The gallery is fitted with access steps which extend from the access house down to the valve chamber which houses the irrigation pipe intake and control valves as well as the scour pipe and control valves.

All outlets are fitted with hydraulically actuated sluice valves which can be remotely controlled from the console in the access house. Several manually operated valves are also fitted in series with the hydraulic ones for emergency manual operation.

The spillway was designed to be excavated in rock on the right abutment and is 20 m wide of free overflow type, with an unlined approach channel. The chute is concrete lined and finishes in a flip bucket at a height of about 68 m above river bed.

b. Distribution System

i. Irrigation

The Distribution System has been designed by the WDD.

As already mentioned the dam is provided with an 0.45 m irrigation outlet. This is to be extended by a 0.3 m dia steel pipeline, to the area to be irrigated about 2 km downstream of the dam, where the water will be distributed by means of a pipe network.

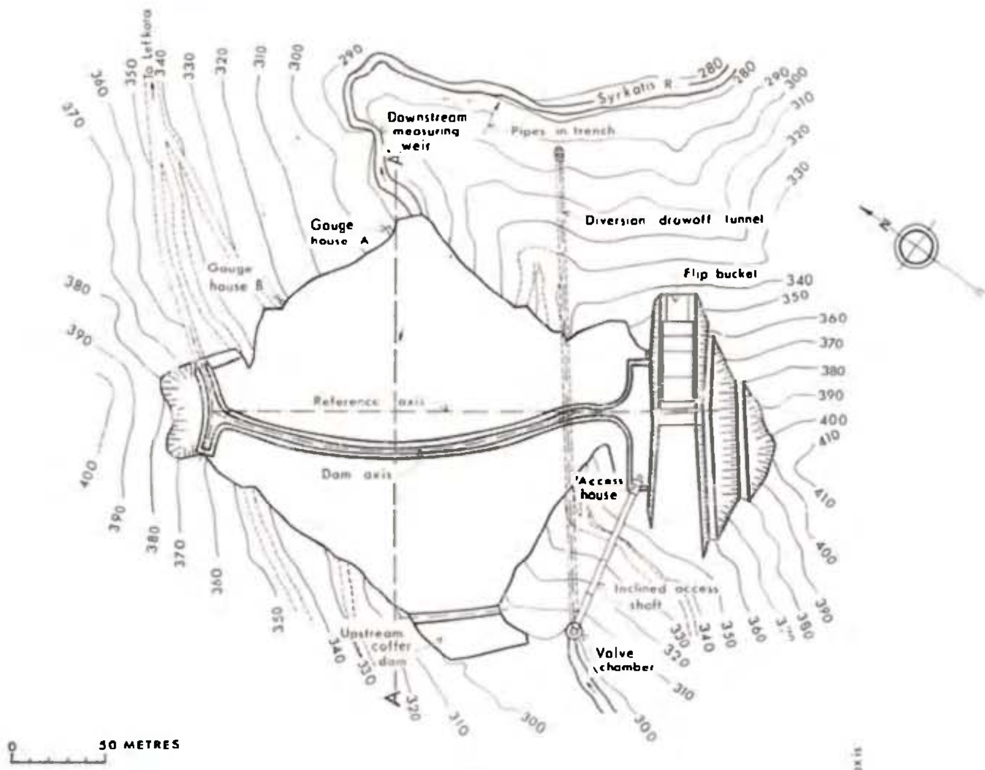
Khirokitia water treatment plant.

ii. Domestic Water

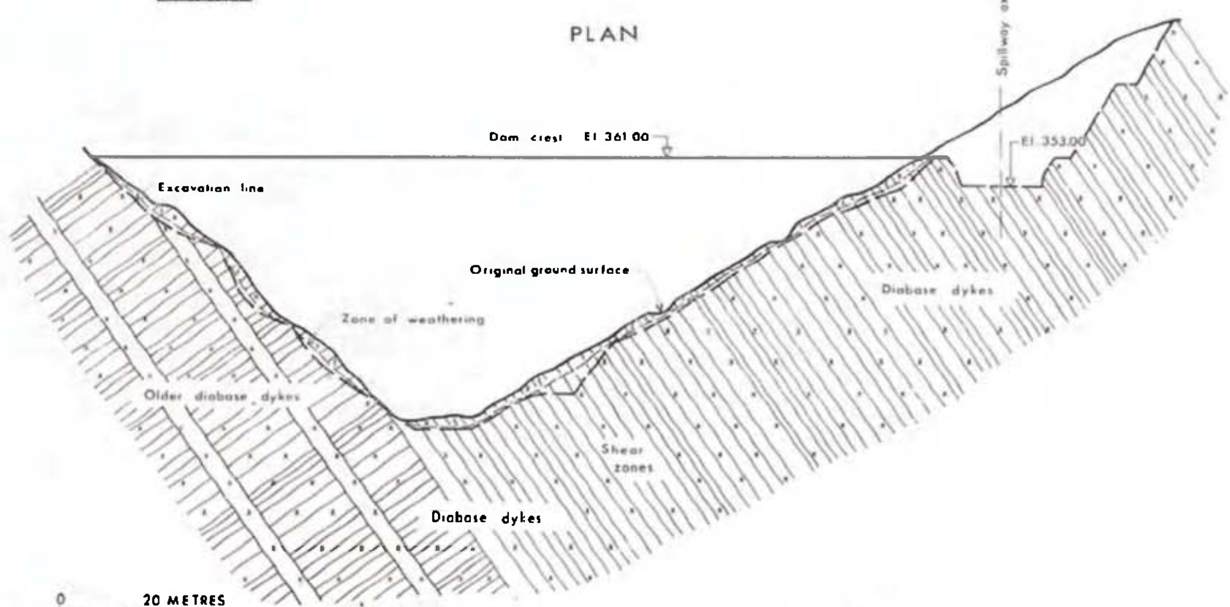
The 0.53 m dia pipe discharges into a break pressure tank immediately downstream of the dam from which the water flows under gravity through a 0.55 m nominal dia steel main to Khirokitia Water Treatment Plant about 13 km downstream and on the

PENDASKINOS - LEFKARA DAM

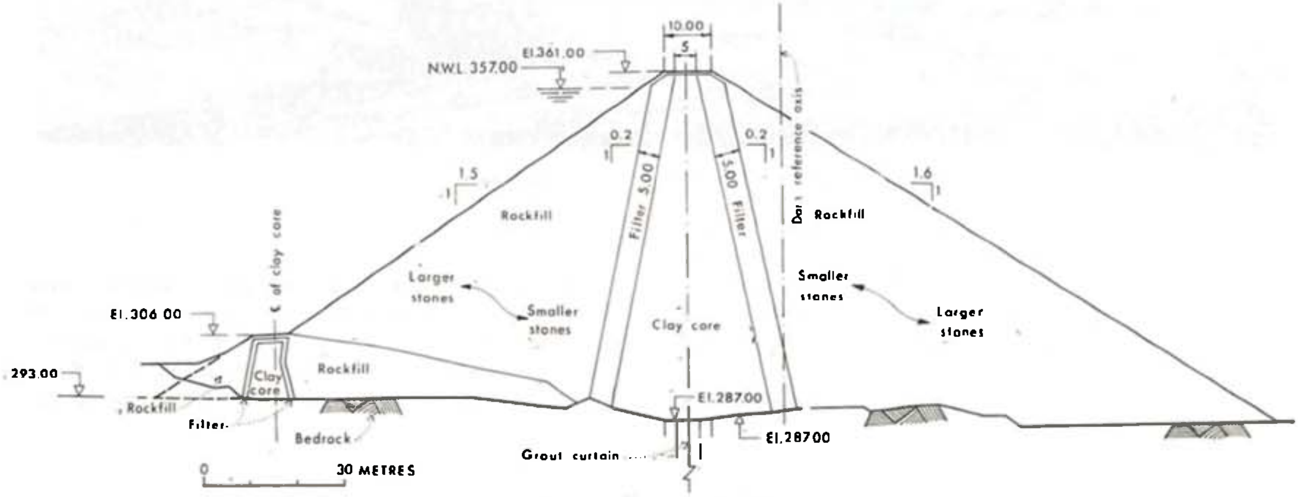
(SHEET 1)



PLAN



GEOLOGICAL SECTION ON DAM AXIS



MAXIMUM SECTION

PENDASKINOS-LEFKARA DAM

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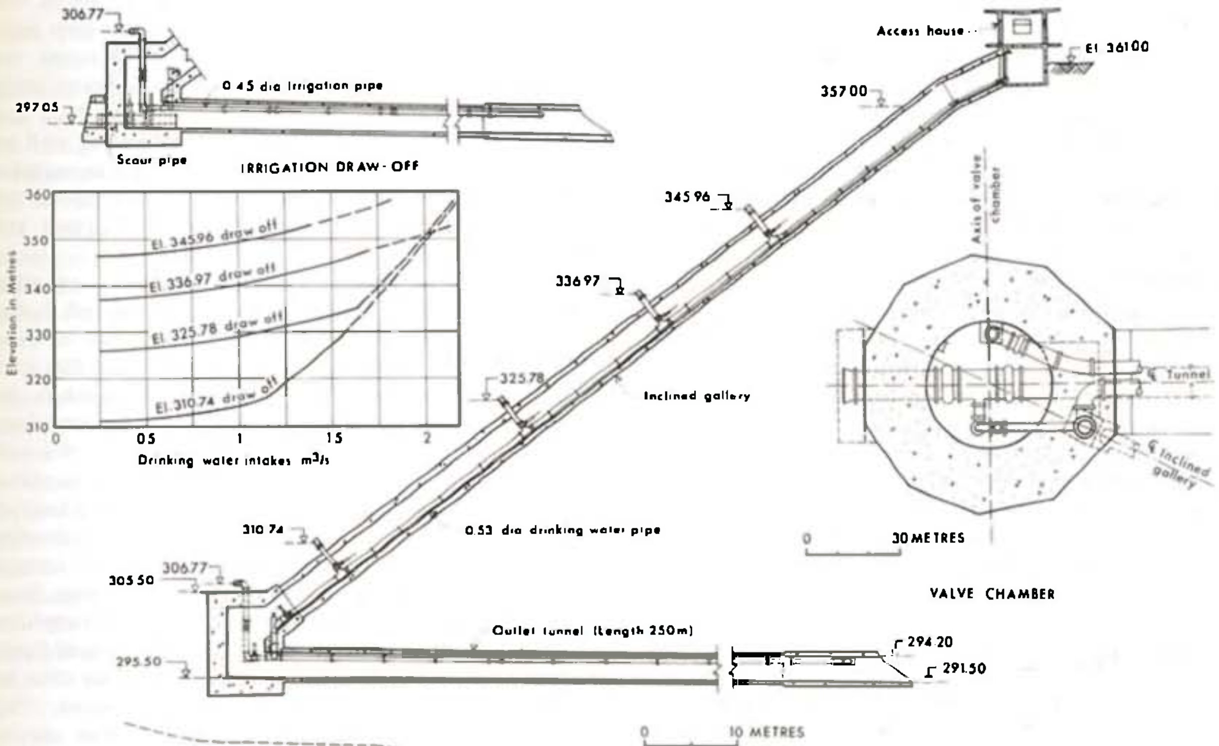
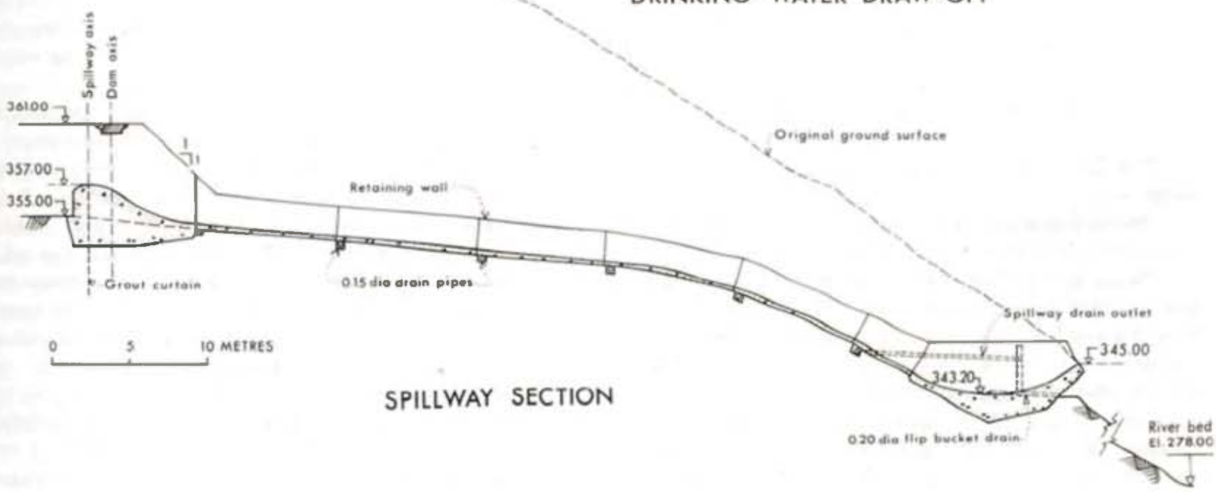
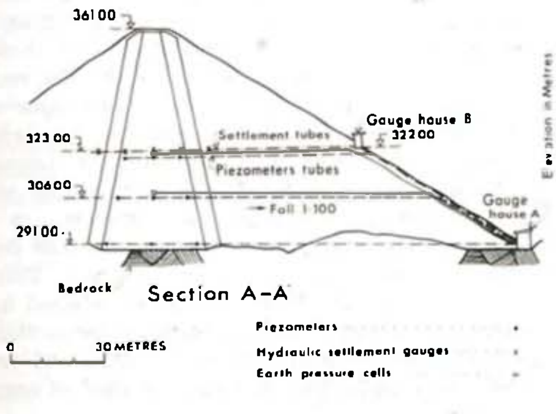


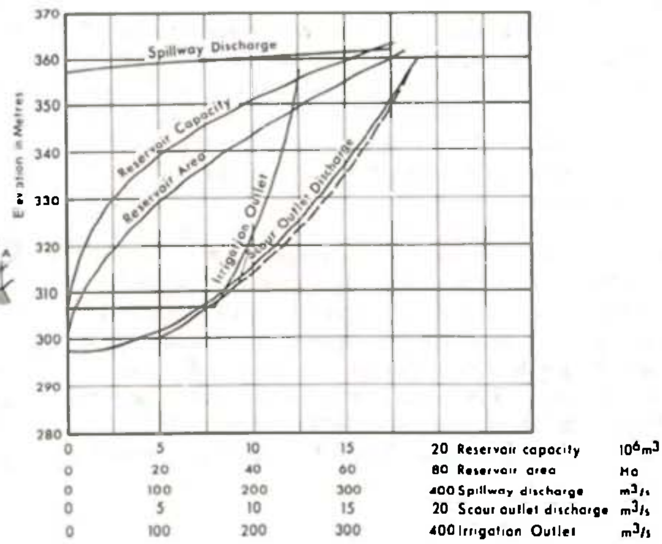
DRINKING WATER DRAW-OFF



SPILLWAY SECTION



INSTRUMENTATION



CAPACITY-AREA-DISCHARGE CURVES

main Nicosia—Limassol Road. The pipeline was designed and laid by WDD whilst the Water Treatment Works were designed by Howard Humphreys & Sons of the United Kingdom and constructed by WDD The Electrical and Mechanical equipment were supplied by Messrs United Filters Engineering Ltd., of the United Kingdom and have a maximum capacity of 20,300 m³/day with provision of extending the works to produce an ultimate output of 31,800 m³/day.

From the Treatment Works the treated water is discharged into a buried reservoir nearby from which it flows again under gravity to Phrenaros reservoir outside Famagusta Town through a 70 km long AC pipeline. Phrenaros reservoir is connected to the Famagusta Water Board reservoirs at Stavros and thence to the Town Distribution System.

A certain proportion of the treated water will also be made available to Larnaca Town and a number of villages along the line.

The design of the Khirokitia—Phrenaros Pipeline was carried out by Messrs Howard Humphreys & Sons and constructed by the WDD.

5. CONSTRUCTION

The construction of Lefkara Dam was undertaken by a joint venture: Messrs. L. Fairclough of the United Kingdom and MEDCON Construction Ltd., of Nicosia. The Contract valued at £1,125,000, was started on the 29th May 1971 and substantially completed by the 31st December 1973.

All vegetation within the dam reservoir was cleared by a separate contract during the construction of the dam.

The first year of construction period was taken up mainly by excavations and temporary works.

The drilling and grouting was undertaken by Sub-Contractors Technoexportstroy of Bulgaria.

Owing to the fact that the clearing of the riverbed had not yet been carried out when the drilling and grouting started, the Sub-Contractor had to begin at the higher elevations of the left abutment and work his way downwards following the clearance of the cut-off.

Construction of the upstream cofferdam started in January 1972 and was completed by the end of March.

The filling operations for the main dam started in June 1972 but it was not before August 1972 that the filling was in full swing.

The rockfill was excavated from quarries opened in diabase rock immediately downstream of the dam. Generally, the diabase yields good quality rockfill but the presence of weathered gabbro dykes and the mechanical breakdown of the rockfill resulted in the production of a rather high proportion of fines during handling and placing. As a result, very close control had to be excercised in order to keep the fines within acceptable limits and preserve the free draining characteristics of rockfill which are essential to this type of design. The rockfill was placed in sluiced layers of a maximum thickness of 1 m and compacted by a minimum of 6 passes of a vibrating roller of 13.4 tons static weight. The compaction was carried out with the rockfill wet in order to minimize post construction settlements.

Filter material was obtained from river deposits mainly from an area about 3 km downstream of the site along the Syrkatis river. Very thorough tests were carried out on this material in order to prove the soundness and durability of the individual grains prior to approval being granted for its use. Apart from grading and permeability testing, ageing tests as well as Los Angeles abrasion tests were carried out on samples of the material and compared to the results of the same tests on samples of Skarinou river gravel and crushed diabase rock.

The material proved suitable for use as an all-in transition material. The problems encountered during construction consisted mainly of the existence of oversized boulders (maximum size allowed was 120 mm). The Contractor chose to deal with the problem by assigning a group of labourers to the job of handpicking and removal of these boulders from the heaps of gravel unloaded on the construction surface. The problem was accentuated by the segretation of the larger fraction during unloading. The mixing action of the spreading blades was most essential in ensuring that no leakage paths through coarse segregated gravel without fines were left across the filter zones. The second problem was that of mixing the gravel with overburden of bands of silt during excavation at the borrow areas thus reducing its permeability to unacceptable values. This problem was dealt with by exercising a certain amount of selection during loading. Minimum compaction of the filter zones was carried out so as to allow the migration to the surface of cracks due to arching and to accommodate any differential settlement between the rockfill and clay core.

The clay used for the core of the dam was taken from borrow areas in red residual clay and slope wash of igneous origin overlying the diabase, both within the reservoir and downstream of the damsite. Being a residual soil above the level of permanent saturation, coarseness increased with depth and the use of each borrow area had to be stopped when the stone content rose above certain limits and the clay became sandy.

Moisture content adjustment was carried out at the borrow areas due to unavailability of water at the borrow areas, and later on it was carried out on the construction surface. Both methods proved satisfactory. The clay was compacted to dry density 100% of Proctor maximum and at optimum moisture content except in the 4 m wide zones adjacent to the relatively steep rock abutments where "wet" clay at about Optimum +4% was used in order to achieve a more plastic fill and minimize the danger of cracking. Compaction was carried out in layers of 22.5 cm compacted thickness using the same vibrating roller as for the rockfill. Compaction control consisted of daily sand replacement tests and moisture content determinations. In order to obtain a quick answer as to the degree of compaction of the material, immediately upon the taking of the sand replacement sample it was compacted in the proctor mould using Proctor effort and its bulk density at the placement moisture content obtained. This was compared to the actual bulk density as obtained by the sand replacement test and a percentage compaction was thus obtained by comparing dry densities and proved to be a very useful tool as a rapid method of compaction control.

The embankment reached crest level in November 1973. The total volume of the dam was about 830,000 m^3.

The following instrumentation has been installed in the dam in order to monitor and assess its performance:
15 "Bishop" type high air entry hydraulic piezometres
10 'Building Research Station" type total pressure cells
5 hydraulic overflow type settlement gauges and
31 surface monuments.

The settlement measured at the mid height of the maximum section was about 0.6 m at the end of construction. No marked differential settlement has been observed between the rockfill shoulders and the clay core.

The 250 m long 2.7 m dia tunnel was excavated through sound diabase with weathered gabbro seams in the right abutments in 3 months from August to October 1971. Both faces were worked simultaneously and a combined rate of progress of 3.5 m per day was achieved. No temporary supports were required other than at the two portals. Permanent steel supports were cast in the concrete lining in areas where the rock was weakened by the presence of weathered gabbro seams.

The tunnel was fully lined with concrete in two operations. First the quadrant invert was cast in 6 m sections and then the remainder of the lining followed in 3 m sections. This was completed at the end of March 1973.

Construction of the 95 m long, 2.8 m dia inclined gallery started at the end of January 1973 and was completed at the end of September of the same year.

The total volume of concrete used at the various sections of the works was about 4,350 m^3.

Grouting works were executed as follows:

The main treatment of the rock consisted of five rows of holes located in the centre of the cut-off with grout holes drilled 2 m apart to a depth based on the formula 1/3H+17m. The outer four rows served as contact grouting, the outer rows being 5 m deep and the inner rows 10 m, with hole spacings 2.5 m apart. The inclination of the boreholes on the left flank and in the riverbed was perpendicular to the excavation level. On the right flank and the spillway the holes were vertical.

The procedure followed regarding the five rows was that the drilling and grouting was carried out from the outer rows of holes to the inner leaving the curtain line last. The grout curtain holes were treated on the split-spacing method with primary holes at 8 m intervals and final distance between holes 2 m.

The drilling of the grout holes was carried out with rotary percussion rigs with hole size 50—64 mm using air flushing. All grout holes were drilled in one operation from the final excavation level to the full design depth.

The grouting was carried out mostly in stages of 5 m in ascending order using rubber packers.

The grout mixture was composed of water and ordinary portland cement with water cement ratios ranging from 6:1 to 1.5:1. The majority of the grout stages were grouted with 4:1 ratios.

The grout pressures applied ranged between 0.60 to 0.75 kg/cm^2 per meter depth measured at the top of the hole and the required pressures were regulated with the return line valve. The required pressure was maintained for at least 10 minutes before starting the next stage.

Grouting of any stage was considered finished when the consumption of the grout mixture fell to 1.5 l/min. for pressures up to 3 kg/cm- and 2 l/min for higher pressures. In cases of high consumption the intermittent method grouting was adopted that is to say grouting was stopped for some time to allow setting and continued afterwards.

The maximum grout take observed in a stage was 805 kg/m.

All grout holes were filled up with cement mortar after completion.

After completion of the grouting, control holes were drilled in various zones and water tests carried out in order to check the effectiveness of the grouting.

Additional holes were sunk in places where high permeabilities were recorded.

A total of 559 boreholes have been drilled of an aggregate depth of 6,820 m. Of these, 6,255 m were drilled with rotary precussion rigs and 565 m with drills.

The total quantity of cement injected amounts to 134,000 kg. Table 11 gives the details of the drilling and grouting work performed.

LEFKARA DAM — GROUTING RESULTS

TABLE 11

Line No. or Other Order	Number of Boreholes	Number of Stages	Total Drilling for Grouting Metres	Consumption of Cement on the Left Flank Grout Holes Kg	Consumption of Cement on the River Bed Area Grout Holes Kg	Consumption of Cement on the Right Flank & Spillway Grout Holes Kg	Total Consumption of Cement Kg	Average Grout Takes Kg m
I	88	88	440	8,100	55	5,205	13,360	30.2
II	99	171	1,040	9.900	330	2.270	12.500	12.0
III								
Primaries	46	228	990	10,456	95	4,629	15,180	15.3
Secondaries	42	210	703	2,543	—	5,750	8,293	11.7
Tertiaries	84	416	1,562	8,483	—	6.804	15.287	9.7
IV	99	173	1,060	6.600	1,420	6.380	14,400	13.6
V	88	90	460	4,140	160	2,450	6,750	14.7
Test & control holes	13	95	565	15,856	13,139	19,235	48,230	85
Total	559	1,471	6,820	66.078	15,199	52,723	134,000	19.6

Data

i. Catchment

Area	36.3 km²
Average rainfall	708 mm/s
Average runoff	8.2 million m³/a
1/1000 years flood	300 m³/s
Maximum height	1,060 m
Maximum length	10 km

ii. Reservoir

Area	65.0 ha
Capacity	13.85 million m³
Live storage	11.6 million m³
Length	3.3 km

iii. Embankment

Structural height	74.00 m
Height above ground level	68.00 m
Hydraulic height	64.00 m
Depth of foundation cut-off	6.00 m
Freeboard	4.00 m
Crest length	233 m
Top thickness	10 m
Base thickness	238 m
Upstream slope	1:1.5
Downstream slope	1:1.6
Upstream core slope	1:0.2
Downstream core slope	1:0.2
Minimum core thickness	5 m
Maximum core thickness	35 m
Rock fill	599,000 m³
Clay core	145,000 m³
Gravel, sand filter	86,000 m³
Total earth fill	830,000 m³

iv. Excavations

Total	165,000 m³

v. Foundations Treatment

Total drilling depth for grouting	6,820 m
Total grout take	134 tons
Average grout take	19.6 kg/m

vi. Spillway

Size	20 m
Capacity	300 m³/s
Length	70 m
Concrete	1,480 m³

vii. Gallery

Size	2.7 m dia
Capacity	19 m³/s
Length	250 m
Concrete	634 m³
Operating gate size	0.9 m dia

viii. Inclined Gallery

Size	2.8 m dia
Length	95 m
Concrete	1,100 m³

ix. Outlet (Steel pipes)

	Scour	Domestic	Irrigation
Size	0.3	0.53	0.45 m dia
Design capacity	19	0.37	0.07 m³/s
Length (within works)	10	250	250 m

x. Distribution System (steel pipes)

(i) Main conveyor

Size	0.3 m dia
Capacity	0.07 m³/s
Length	2,070 m

(ii) Distribution Network

Size	0.10, 0.15 and 0.25 m dia
Capacity	0.015—0.045 m³/s
Length	5,640 m

CHAPTER V

EARTHFILL DAMS

INTRODUCTION

Earthfill dams have been introduced in Cyprus since 1896 when the first storage dams were built at Kouklia, Akhyritou and Syngrasi. Earth was the first building material used for dam construction much before masonry and concrete. Earthfill dams are either essentially preferred on technical grounds where gravity dams cannot be built, or on economic grounds where they may prove cheaper than other types.

a. The main technical criteria which necessitate earthfill dam construction are:

i. Weak foundation conditions, such as alluvial and clayey soils. Also many times where soft rock formations such as sandstones and chalks exist.

ii. Where seepage may be allowed through or under dams as in the case of groundwater recharge dams, provided with an upstream blanket and downstream drains.

b. The main criteria which would most probably infuence earthfill dam construction to be more economic than other types of construction are:

i. Weak foundation rock.

ii. Availability of earthfill at the site.

iii. Fiat topography of the site.

iv. Suitable spillway location.

v. Easy river diversion.

vi. Difficulties in accessibility of site.

vii. Seismic activity.

From table 1 we can see, that 15 out of the 25 large dams are earthfill, whilst there are over 130 other smaller earth dams built today in Cyprus for groundwater recharge purposes and which are dealt with in the last chapter.

For the future dams the majority will also be far more of earthfill than of all other types together. This is a consequence of the conditions existing in Cyprus as given above, which favour earthfill dams.

Table 10 gives the design data and the main properties of the materials used for the construction of the existing Earthfill Dams.

Yermasoyia dam under construction.

PEDHIEOS—ATHALASSA PROJECT

1. PURPOSE

The main purpose for constructing this dam was due to the serious flooding of the Athalassa Government Farm which includes an Agricultural Farm, an Animal Husbandry Farm, the Agricultural Research Institute Lands and the Forest Nursery.

These floods which come from the Kaloyiros tributary of the Pedhieos river caused considerable damage to the farms at frequent intervals before the construction of the dam. It was, therefore, decided to build this dam primarily to intercept the floods of the river and to save the farms from damages. As there existed downstream of the Government farms irrigation intakes belonging to the villages of Kaimakli and Eylenja, it was originally proposed that these villages also participated in the Project. However, due to the present urbanized character of these villages which in fact, have now become sectors of Nicosia, the beneficiaries abandoned these intakes and showed no interest for the Project. Therefore, it was finally decided that the dam should be built solely by the Government for the protection of the farms and for supplying water for their needs. The water from the dam is mixed in a 220 m^3 capacity balancing reservoir just downstream of the dam with the water coming from three boreholes. The purpose of the water from the boreholes is to ensure a more reliable summer supply as well as to enable experimentation with different water qualities by appropriate mixing of the water of the various supply sources.

2. LOCATION

This Dam is located on the Kaloyiros tributary of the Pehdieos river at a site just at the outskirts of Nicosia in a SE direction and at an altitude of about 150 m asl.

3. PLAN

a. Water and Land

No automatic flow measurements have been recorded on this stream, before construction, but from meteorological data it has been evaluated that the average runoff of the Kaloyiros at the damsite is about 0.36 million m^3/a. The maximum flood estimated at the dam site which would occur at a frequency of 1/1000 years is estimated to be 39 m^3/s.

Water use rights on this stream were exercised by the beneficiaries of the Irrigation Divisions of Eylenja and Kaimakli villages. The Eylenja Division used to command 460 donums of wheat and barley whilst the Kaimakli Irrigation Division used to command 1800 donums cereals. At the same time 450 donums of the Athalassa Farm were commanded.

b. Geology

The dam and reservoir sites lie on the Athalassa—Nicosia Formation. This Formation is made of Pliocene marine deposits consisting of alternating beds of marls and calcareous sandstones. The marly beds are thicker and more frequent at the lower part of the succession whilst at the upper part the calcareous sandstones dominate with some occasional conglomerates and pebble beds.

Permeability problems through the upper part of the succession may exist and have to be taken care of.

The investigations carried out reached the impermeable marly beds which were found in the riverbed overlain by shallow alluvial gravel and sand deposits. The problem of silt deposition would be important because the Athalassa—Nicosia Formation which forms the main geology of the catchment consists of soft and highly erosive rocks under intensive rainfall conditions. This problem has been taken into account in deciding the storage requirements of the reservoir.

4. MAIN FEATURES

a. Dam

The detailed design of the dam was undertaken by the Department and included the distribution systems required by the farms.

The dam is made up of a clay core wall which is keyed into the marl foundation, on either side of which gravel and random fill zones have been provided. As the sandstone rock foundation of the spillway was good, and the capacity of the reservoir was made large to cope with the floods, the spillway has been left unlined in the first instance. An 0.30 m dia outlet steel pipe has been laid through the dam from its upstream to its downstream side and has been encased in concrete with suitable cut-off collars at intervals.

The operation of the pipeline is done through a downstream valve.

b. Distribution System

A piped distribution system was designed by the WDD for conveying the water of the dam and boreholes and distributing it to the Agricultural Research Institute and the Department of Agriculture Farms.

5. CONSTRUCTION

The WDD constructed this dam between April and December, 1962. No difficulties occurred during construction except the filling in with clay of the chain-of-wells which passed under the dam.

This was done satisfactorily and no significant leakage occurred after impoundment. All fill materials were obtained from excavations. The compaction of the clay core was done by sheepfoot rollers in 0.15 m layers to a dry density of 1.640 kg/m^3 and at an optimum moisture content of 20%.

The total expenditure incurred on this dam reached £22,600 and was all paid by the Government.

All conveyors from the boreholes to the mixing reservoir and the distribution system therefrom are steel pipes laid by the WDD. The cost of the borehole conveyance system reached about £3.000 whilst that of the pipeline distribution network downstream of the reservoir reached about £3.500.

All costs were paid by Goverment.

7. PROBLEMS OF SPECIAL INTEREST

There were no serious problems after impoundment of water in the reservoir. However, there was concern especially by the Agricultural Research Institute because of the high Ph value of the water which reached values up to 9.5 and made the water alkaline. Also boron content values are excessive reaching up to 1.35 ppm. On the other hand, the salinity and total hardness of the water was satisfactory, the former being about 120 ppm and the latter 165 ppm. The reason for this high Ph value of the water was found

Erosion of unlined spillway.

6. MANAGEMENT

This dam is now managed by the Farm of the Department of Agriculture for the irrigation of its lands, and for supplying water to their stock farm and by the Agricultural Research Institute for their irrigation experiments.

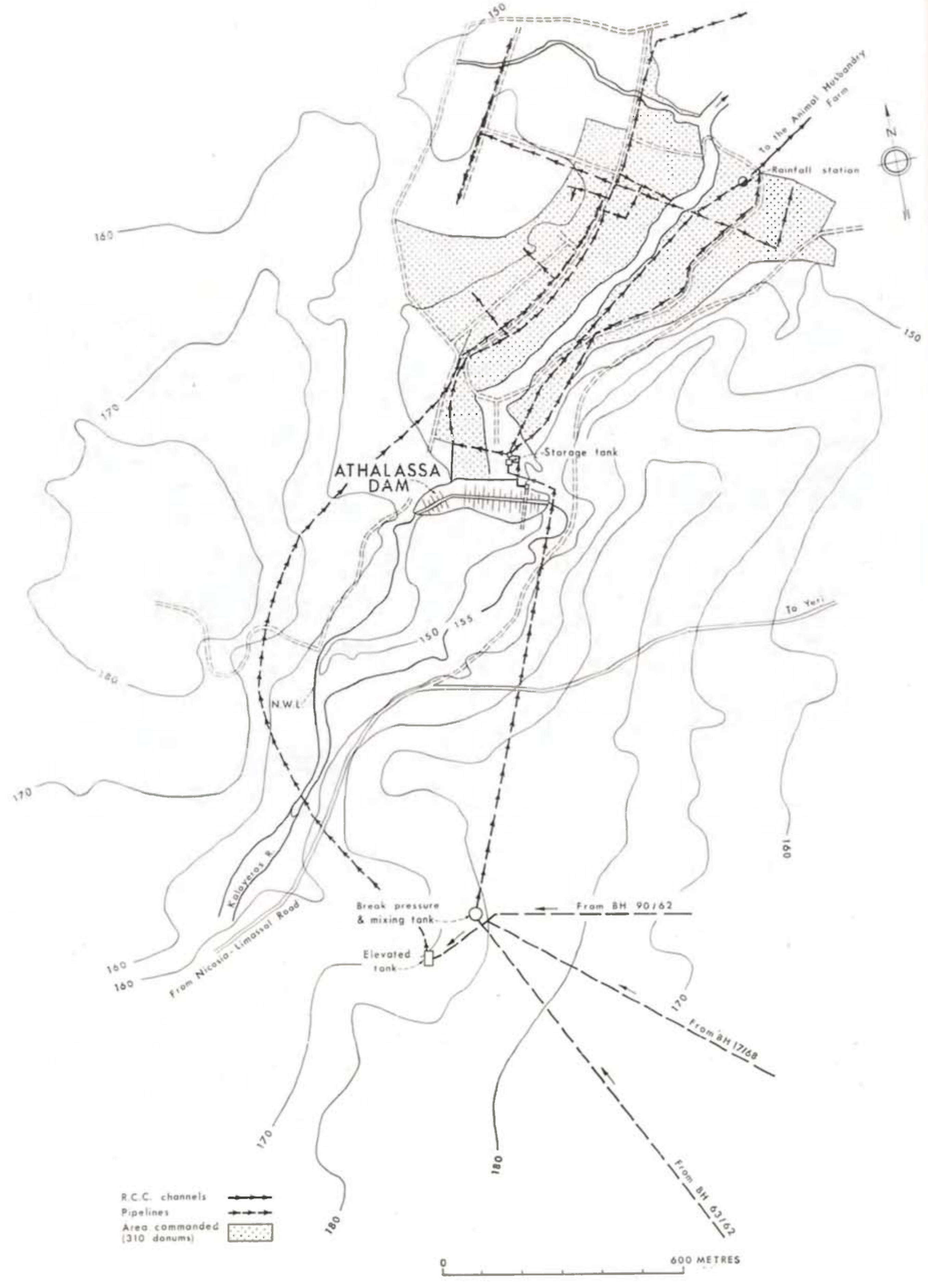
The land commanded by the distribution system is that of the Agricultural Research Institute for about 310 donums.

to be the sewage and industrial waste disposal from minor industries which are situated upstream of the reservoir and quite near to it.

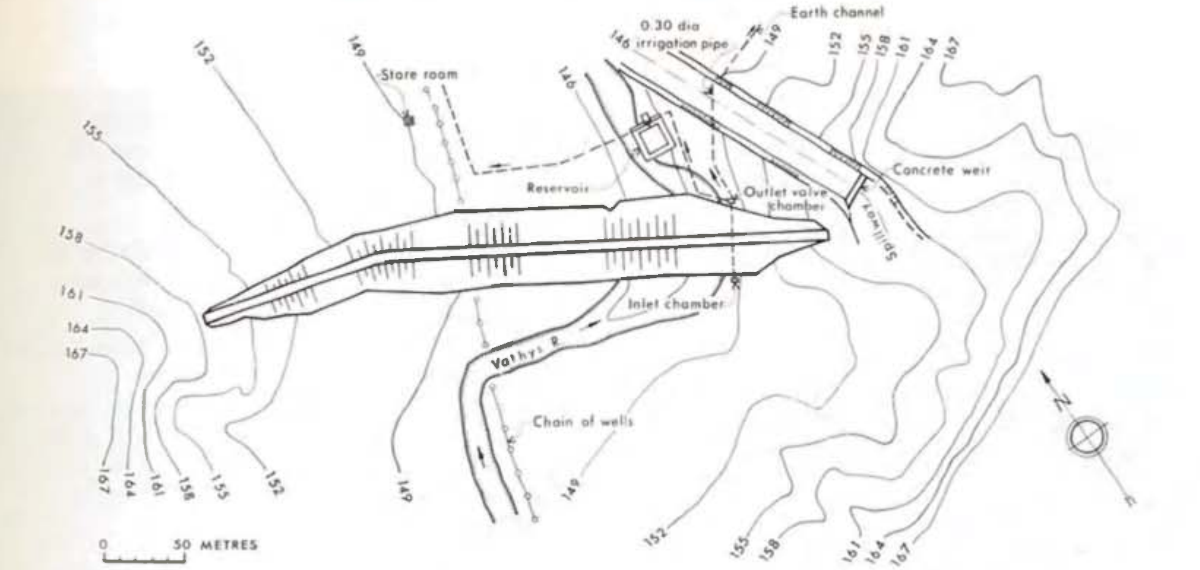
Measures are being taken to stop the contamination of the water.

Another interesting aspect was that during the big flood in 1969 damages occurred to the earth spillway and as a consequence the crest was built in concrete and the downstream chute was formed by excavation and its sides planted with acacia trees to stop erosion. The cost of this work reached about £900.

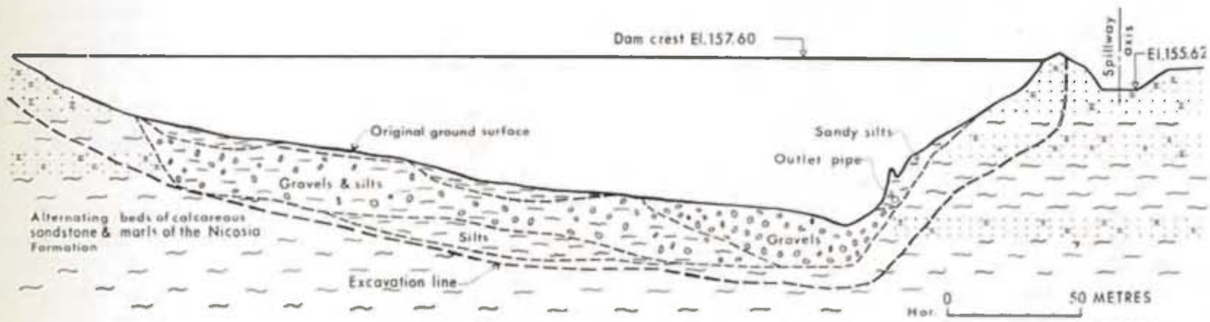
PEDHIEOS - ATHALASSA - DISTRIBUTION SYSTEM



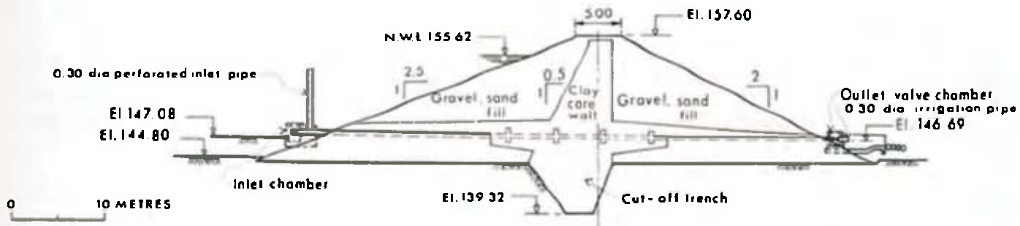
PEDHIEOS-ATHALASSA DAM



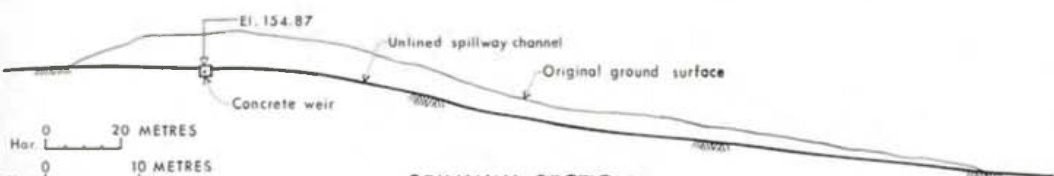
PLAN



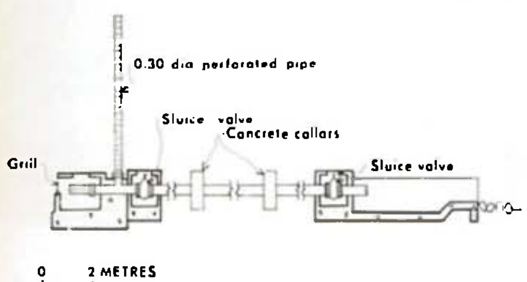
GEOLOGICAL SECTION ON DAM AXIS



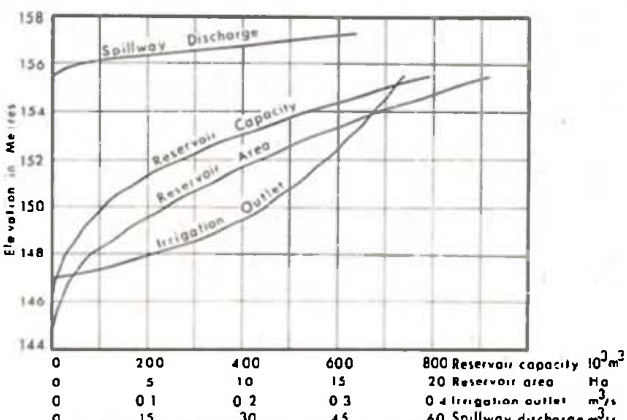
MAXIMUM SECTION



SPILLWAY SECTION



INLET & OUTLET DRAW-OFF PIPES



CAPACITY-AREA-DISCHARGE CURVES

Data

i. Catchment

Area	34 km^2
Average rainfall	345 mm/a
Average runoff	0.36 million m^3/a
1/1000 years flood	39 m^3/s
Maximum height	368 m
Maximum length	15 km

ii. Reservoir

Area	23 ha
Capacity	0.79 million m^3
Live storage	0.78 million m^3
Length	1.300 m

iii. Embankment

Structural height	18.28 m
Height above ground level	12.80 m
Hydraulic height	10.82 m
Depth of foundation cut-off	5.48 m
Freeboard	1.98 m
Crest length	415 m
Top thickness	5 m
Base thickness	63.50 m
Upstream slope	1:2 5
Downstream slope	1:2.0
Upstream core slope	1:0.5
Downstream core slope	Vertical
Minimum core thickness	2.40 m
Maximum core thickness	8.80 m
Random fill	71.000 m^3
Clay core	17,200 m^3
Total earth fill	88.200 m^3

iv. Excavations

Total	18.800 m^3

v. Spillway

Size	30x2.4 m
Capacity	46.5 m^3/s
Length	240 m

vi. Outlet (Steel pipes)

Size	0.30 m dia
Capacity	0.25 m^3/s
Length	54 m

vii. Supply Conveyor from three boreholes to balancing reservoir (AC Pipes)

Size	0.25, 0.20, 0.15 0.10 m dia
Capacity	0.014 m^3/s
Length	8,640 m

viii. Distribution system (steel pipes)

(i) Main Conveyor

Size	0.15 m
Capacity	0.014 m^3/s
Length	1.830

(ii) Distribution network

Size	0.15 m, 0.10 m dia
Capacity	0.014 m^3/s
Length	990 m

(iii) Conveyor from balancing reservoir to Animal Husbandry Farm

Size	0 20, 0.15 m dia
Capacity	0.014 m^3/s
Length	945 m

(iv) Conveyor from elevated tank to Livestock Farm

Size	0.10 m dia
Capacity	0.006 m^3/s
Length	3000 m

GEUNYELI

The Geunyeli dam.

PEDHIEOS—GEUNYELI PROJECT

1. PURPOSE

This dam has been built to supplement the spate water supply available from the intake on the Almyros tributary of the Pedhieos river, which was used by the Geunyeli Irrigation Division for flood irrigation of wheat and barley. As no groundwater of any importance can be obtained from the area and substantial floods come down the river during the winter season, the construction of the dam has enabled the storage of water to take place, which is used for supplementary irrigation of wheat and barley during spring and early summer months. It is necessary to utilize the water from the dam as early as possible preferably before the month of July for many reasons such as: the infrequent flow in the river, the deterioration of the quality of the water with time and storage, and the great evaporation losses. With the use of the water for early vegetable crops in the dry farming villages of Mesaoria, such as Geunyeli storage dams contribute largely to the village economy.

2. LOCATION

The dam is located on the Almyros tributary of the Pedhieos river and at about a distance of 2.5 km upstream of the Geunyeli village. It lies at an altitude of about 150 m asl.

3. PLAN

a. Water and Land

From an automatic flow recorder installed on the river since 1956 and from interpolation from nearby recorders and meteorological data, it has been estimated that the average runoff of this stream at the damsite is of the order of 0.625 million m³/a.

Water from this stream was utilised prior to the construction of the dam by the people of Geunyeli who had an intake on the river and an Irrigation Division managing their irrigation system. The dam has been built to supply water to this intake through a pipeline, leading into the main conveyor canal which has been lined as far as Geunyeli village.

b. Geology

The area falls well within the Kythrea Formation of the Southern foot hills of the Kyrenia Range. The upper part of the catchment is the Hilarion limestone, which is overlaid at its foot hills by the Lapithos chalks, which in turn are overlaid by the Kythrea Formation. The Kythrea Formation which is found at the damsite consists of alternating beds of marls, sandstones and shales, densely folded along the EW axis parallel to the mountain range. This Formation is gently sloping towards the main Mesaoria plain and forms ideal reservoir sites. Geologically the Formation is also ideal and it could be considered as water tight. Limited permeability can actually occur through confined joints in the sandstone beds of the Formation which, in most cases rapidly silts up.

Investigations have been carried out to reach the impermeable marly beds which were overlaid by shallow alluvial sand deposits. The problem of silt deposition in the reservoir is important because by nature, the Kythrea Formation is highly erosive and under heavy rainfall conditions, substantial silt is transported down the river. Another problem of this Formation is the connate salts and carbonates which are washed through and contaminate the water in the river. A high percentage of boron is also present in the water.

4. MAIN FEATURES

a. Dam

The detailed design for the dam was undertaken by the WDD including the distribution system.

The dam is made up of a single zone uniform clay fill embankment with a shallow cut-off keyed into the impermeable marl. On the upstream side of the dam and around water level a hand placed rip-rap has been provided. A narrow gravel and sand filter zone with a downstream blanket has also been provided to enable proper drainage of the embankment. Because of the sound sandstone foundation at the spillway and the large capacity of the reservoir which exceeds the normal runoff of the river, the spillway has been left unlined apart from a concrete wall to protect the embankment.

A 0.30 m dia steel pipeline has been laid from the upstream to the downstream side conveying water into the irrigation channel. The intake arrangement is made up of a vertical perforated pipeline. The pipeline across the dambody has been embedded in concrete with concrete collars at suitable intervals. The operation of the pipeline is done from a valve on the downstream side of the dam.

b. Distribution System

A distribution system designed by the WDD includes a trapezoidal concrete lined main conveyor from the dam to the lands commanded for irrigation at Geunyeli and an earth channel distribution network.

5. CONSTRUCTION

The WDD constructed this dam between April and December 1962. The main difficulty during construction was the non-availability of sufficient water at the site for the construction of this dam. Therefore a small coffer dam was built early in the winter of 1962 and filled with about 0.045 million m³ of water which was used during construction. All materials except the aggregate for the concrete were obtained from the site. The compaction of the fill was done by sheepfoot rollers. The total expenditure on this dam was £22,000.

The lining of the main trapezoidal channel as far as Geunyeli was constructed by the WDD and reached the cost of £4,000. It was constructed during the same period as the Dam.

The Geunyeli Irrigation Division has undertaken to contribute 25% of the total cost, having obtained a loan from the Government payable in 20 years at a rate of interest of 5.5%.

6. MANAGEMENT

The Project is managed by the Geunyeli Irrigation Division.

The land commanded by the distribution system is about 850 donums, the main crops being wheat, barley and some spring vegetables.

7. PROBLEMS OF SPECIAL INTEREST

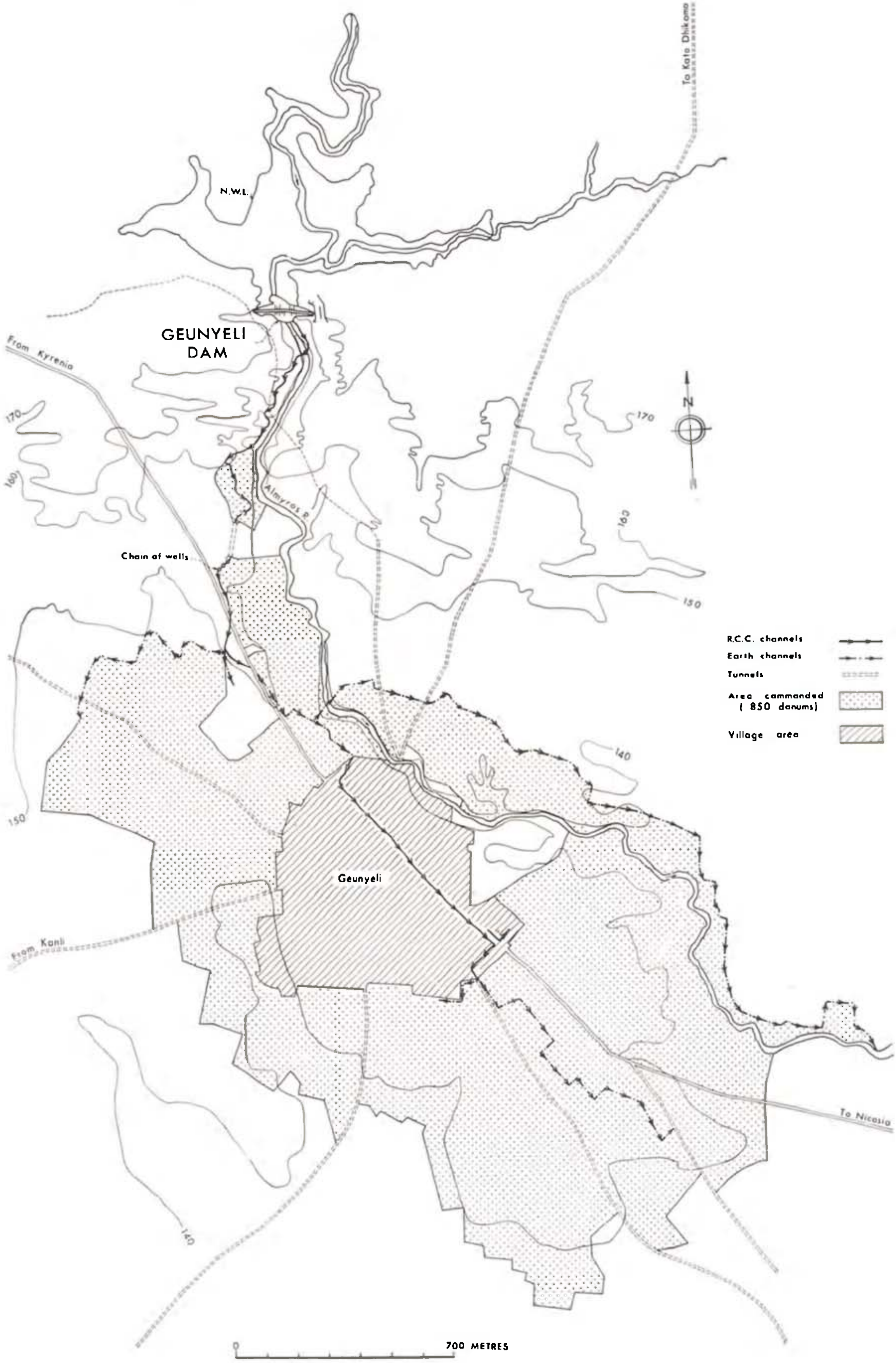
No serious problems developed after the impoundment. However, during continuous operation for the last few years, it has been noticed that some erosion occurred on the unlined spillway, and it may be necessary to carry out some further concrete lining works in the future.

The usual problem of the deterioration of the quality of the water due to the disolution of salts and carbonates of the Kythrea Formation is also present. It has been observed that the Sodium Chloride concentrations rise from about 80 ppm to about 400 ppm in one year's storage, whilst the total hardness rises from 200 ppm to 580 ppm during the same period. This problem favours the annual operation of the dam during short irrigation periods to avoid the deterioration of the quality of the water during long storage periods due to evaporation. The reservoir after emptying is closed to receive fresh water in the following winter season.

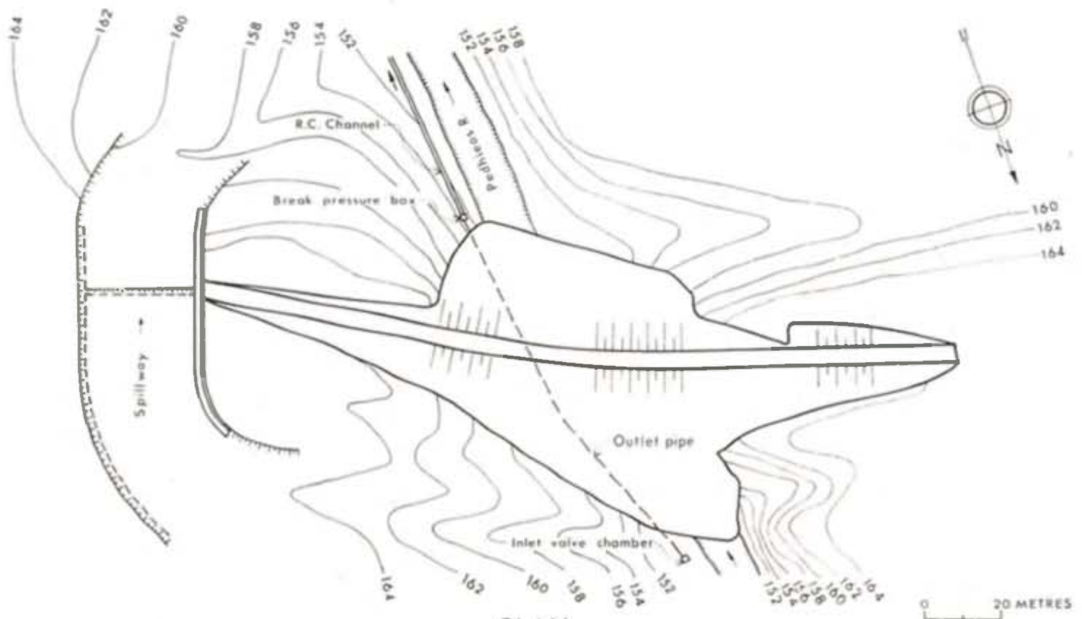
Data

i.	*Catchment*	
	Area	26 km^2
	Average rainfall	490 mm/a
	Average runoff	0.625 million m^3/a
	1/1000 years flood	170 m^3/s
	Maximum height	921 m
	Maximum length	7.7 km
ii.	*Reservoir*	
	Area	27.6 ha
	Capacity	1.045 million m^3
	Live storage	1.00 million m^3
	Length	1,200 m
iii.	*Embankment*	
	Structural height	18.46 m
	Height above ground level	16.15 m
	Hydraulic height	13.22 m
	Depth of foundation cut-off	2.31 m
	Freeboard	2.93 m
	Crest length	196 m
	Top thickness	5.00 m
	Base thickness	90 m
	Upstream slope	1:3
	Downstream slope	1:2
	Random fill	41,500 m^3
	Gravel Sand filter	3,000 m^3
	Rock rip-rap	1,500 m^3
	Total earth fill	46,000 m^3
iv.	*Excavations*	
	Total	15,000 m^3
v.	*Spillway*	
	Size	36x2.9 m
	Capacity	170 m^3/s
	Length	57 m
	Concrete	450 m^3
vi.	*Outlet (Steel pipes)*	
	Size	0.30 m
	Capacity	0.20 m^3/s
	Length	109 m
vii.	*Distribution System*	
(i)	*Main conveyor (Concrete channels)*	
	Size Trapezoidal	(0.6+1.2)x0.6 m
	Rectangular	0.75x0.75 m
	Capacity	0.56 m^3/s
	Length	2,382 m
(ii)	*Distribution Network (Earth channels)*	
	Size	0.9x0.45 m
	Capacity	0.30 m^3/s
	Length	4,695 m

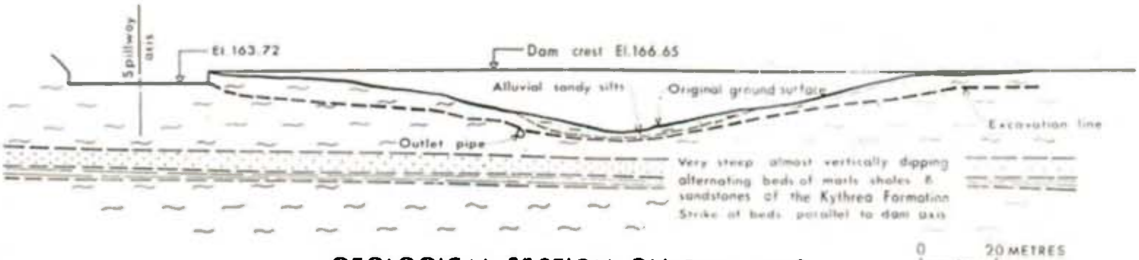
PEDHIEOS - GEUNYELI - DISTRIBUTION SYSTEM



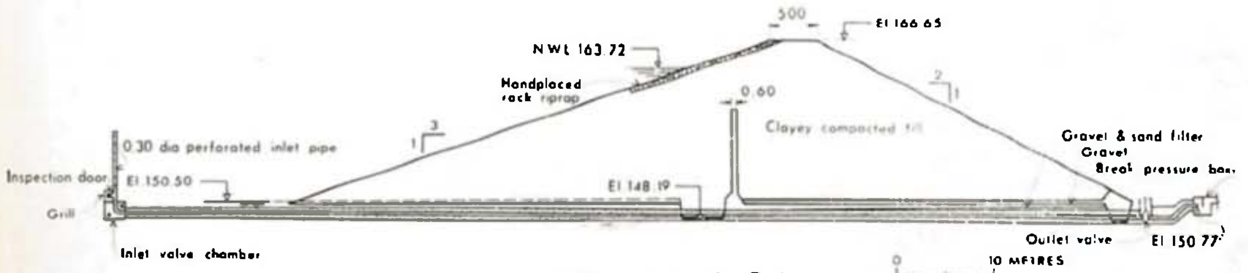
PEDHIEOS - GEUNYELI DAM



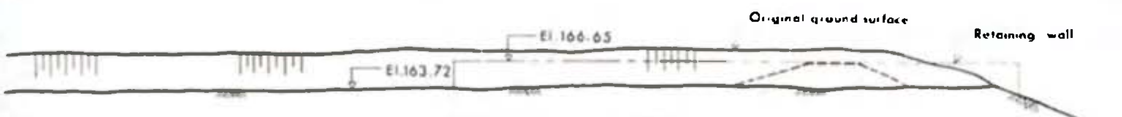
PLAN



GEOLOGICAL SECTION ON DAM AXIS

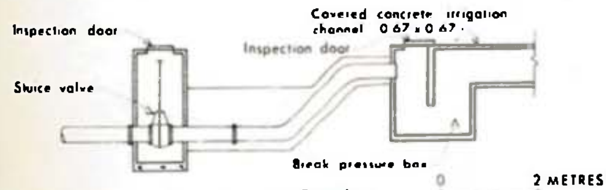


MAXIMUM SECTION

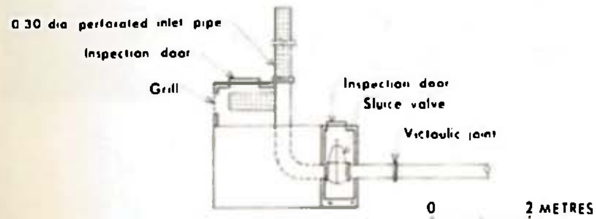


SPILLWAY SECTION

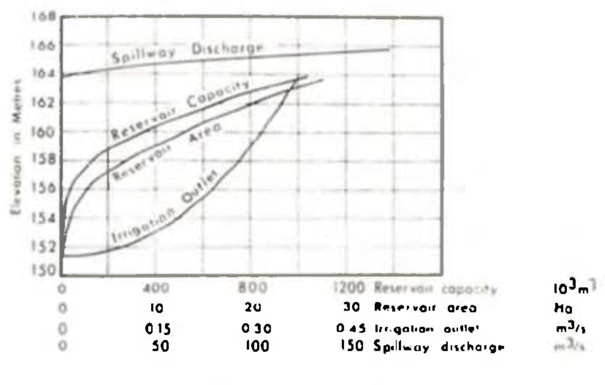
0 10 METRES



DRAW-OFF OUTLET

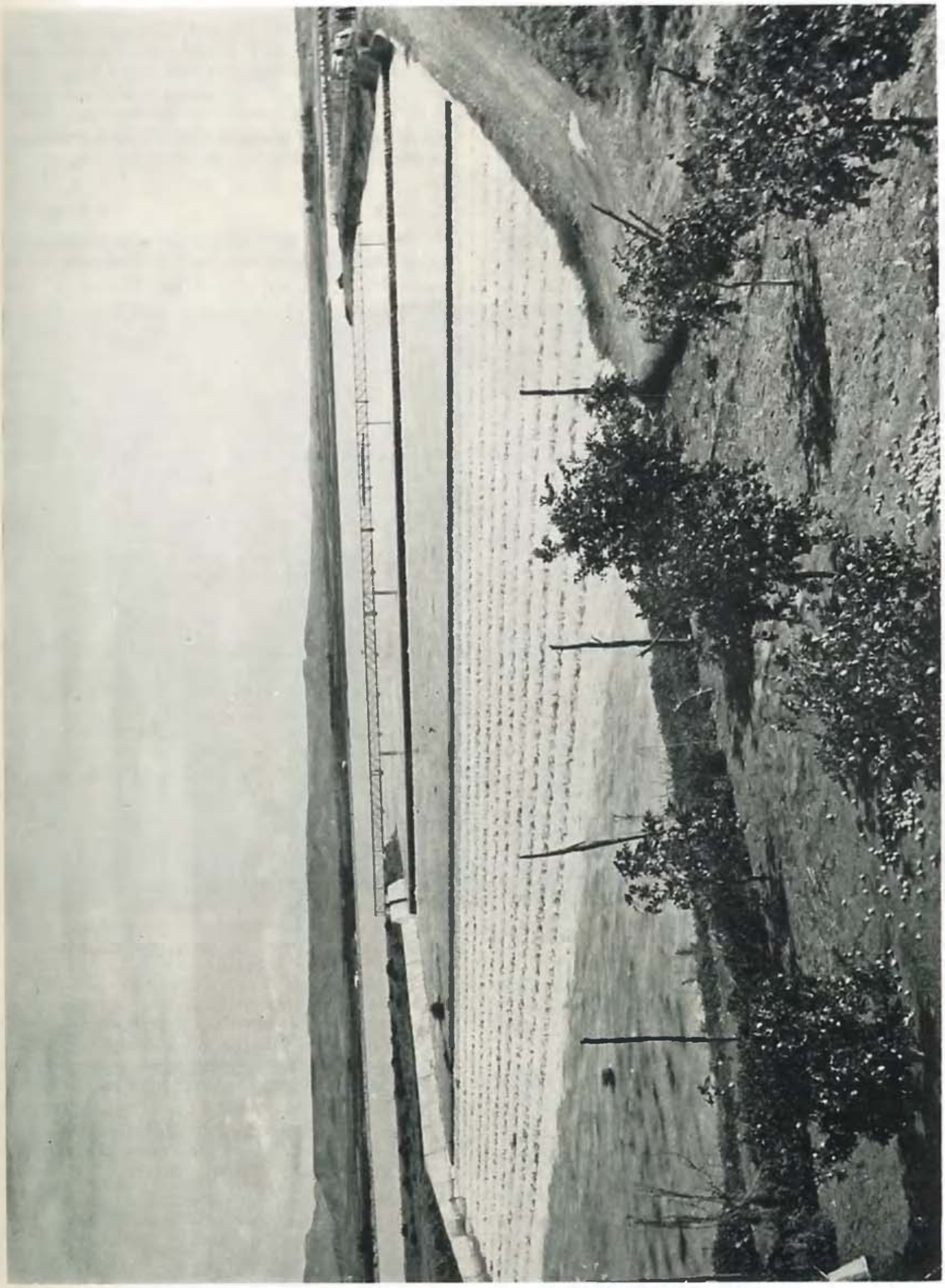


DRAW-OFF INLET



CAPACITY-AREA-DISCHARGE CURVES

MORPHOU



The Morphou dam.

SERAKHIS—MORPHOU PROJECT

1. PURPOSE

The decline of the water table at Morphou aquifer due to the overpumping which was a result of the uncontrolled extension of the citrus plantations and the increasing needs for domestic water supply especially that of Nicosia, called for the provision of additional water supplies and the proper conservation of the available water resources. During the 10 years which preceded the construction of the dam between 1953 and 1963, the aquifer was drilled by some 2,350 boreholes ranging in depth up to about 150 m. The amount of water that was being pumped annually when the dam was constructed was 65 million m^3. Out of this quantity about 2 million m^3 were used for Nicosia and regional villages water supply whereas the remaining was consumed for the irrigation of some 45,000 donums of land, most of which was citrus plantations.

As the extraction exceeded by about 15 million m^3 the natural recharge which is effected by the Serakhis river, some other regional rivers and direct rainfall, the result was that the average drop of the water table was at the rate of about 1 m annually.

At the coastal region at Syrianokhori where until 1953 high water table and marshy conditions prevailed, the water table dropped up to about 4 m below sea level creating serious danger for sea intrusion.

Consequently, the Nicosia water supply boreholes were abandoned at this region and other replacement boreholes were drilled inland.

A plan produced in 1960 with subsequent modifications provided for a series of development and conservation schemes to include:

a. Lining of canals, and piped conveyors to reduce the seepage and evaporation losses.

b. More efficient water application methods such as sprinkler and hose-basin irrigation to reduce the water requirement on the irrigated lands.

c. The enaction and enforcement of more strict water legislation for the control of drilling, extraction and use of the water resources and their proper distribution.

d. The carrying out of groundwater recharge schemes, through dams, spreading grounds and boreholes.

e. The conveyance of additional supplies of water to the region through river diversion works to be built on the Western rivers as far as Tylliria where considerable surplus of water exists even after allowing for significant local development. The Morphou dam has been built on a part of the aquifer formed of deep river sand and gravel alluvium which actually forms part of the Morphou aquifer and through which infiltration and replenishment takes place.

2. LOCATION

The dam has been built on the main Serakhis river at the outskirts of Morphou being at a distance of 8 km upstream of the sea and at an elevation of about 50 m asl.

3. PLAN

a. Water and Land

The catchment area of the dam is about 458 km^2 reaching a maximum altitude of 1,560 m on the Troodos moutains. The average rainfall on the catchment is 480 mm. There are 4 automatic flow recorders on the rivers of this catchment and through evaluation of the flow records taken as well as from the numerous rainfall data, the average annual runoff estimated at the damsite is of the order of 13 million m^3. The maximum flood estimated at the damsite at a frequency of 1/1000 years is about 1,400 m^3/s.

The Morphou Teratsia Irrigation Division had an intake on the river just downstream of the site where the dam was constructed which diverted water into a canalized distribution system which commanded a substantial part of the citrus plantations. The dam has been built for this Irrigation Division and was connnected by means of a conveyor pipeline to the intake of the canalized irrigation system. This connection enables the direct irrigation of the plantations when water is available during the irrigation season. Also when surplus water is available in the winter season, the channels can be utilized for the provision of additional water for ground water recharge by seepage through the unlined canals or through excessive application of water on the irrigated land.

b. Geology

The dam has been built on Recent alluvial deposits made up of coarse gravel, sand, silts and clays. These sediments reach up to a depth of 150 m at places and constitute the Morphou aquifer which is the highest water producing in Cyprus. The catchment lies in a variety of geological rocks ranging from Recent alluvial deposits, Miocene Marls of the Pakhna Formation and Igneous made up of pillow lavas, basal and diabase rocks.

The investigations carried out included deep boreholes to determine the depth of the alluvium, test pumping for the permeability and other investigations in the borrow areas for the embankment materials.

4. MAIN FEATURES

a. Dam

The design of the dam and the spreading grounds downstream was done by the WDD. The embankment is formed of a clay core wall and zones of gravel and sand fill on either side of it. The clay core is connected to an upstream clay blanket which is of 45.70 m total length from the centre line of the clay core.

A selected gravel dumped rip-rap which has been reinforced later by rock has been placed on the upstream side.

A 75 m long spillway has been provided on the left abutment capable of discharging 360 m^3/s. At a later stage it was considered that its capacity was not enough to cope with the maximum flood anticipated and the embankment was raised by another 1.2 m which enabled the accommodation of 680 m^3/s. This copes with an estimated flood of 1/100 years. Together with a relief spillway in the form of a breach embankment at the tail end of the dam embankment, this capacity could be increased to accommodate the 1/1000 years flood at 1400 m^3/s.

The spillway channel is made up of concrete stepped weirs and protection walls as far as the river bed making up a most economic construction.

An outlet concrete gallery has been cast through the embankment to house the 0.38 m dia steel outlet pipe for irrigation operated from a downstream valve.

b. Spreading Grounds

As this dam is built on a major river it is bound to receive a considerable load of sediment which in a few years would render the reservoir bed impermeable. A series of spreading grounds were provided downstream of the dam in which clear water from the dam would be diverted for infiltration and to increase the amount of water recharged. Two spreading grounds in series have been built just downstream of the dam of a total capacity 0.134 million m^3 at the left abutment utilizing part of the river bed, the river flow having been canalized towards the right abutment. The banks of these spreading grounds are of clay core with gravel zones on either side and their maximum height is about 3 m. They are interconnected by a concrete overflow spillway from the first to the second reservoirs and the second reservoir is provided with a concrete overflow spillway to discharge the surplus into the river.

Overflowing spreading grounds on the left with flooded river course on the right.

A third spreading ground of 0.09 million m^3 capacity has been built further downstream and formed of gabbion walls across the river bed to provide a flexible construction and made water tight by a clay blanket on the upstream side protected by a gravel blanket.

5. CONSTRUCTION

The dam was constructed by the WDD between April and December 1962. No special problems were experienced. All materials except the concrete aggregates were obtained from the site. The clay core was compacted by sheepfoot rollers whilst the sand and gravel shells by vibrating rollers.

The total expenditure on the dam itself was £95,000 and another £13,000 was spent in 1969 for the raising of the embankment to provide additional spillway capacity as well as for further rip-rap protection and extension of the concrete spillway protection walls.

The construction of the spreading grounds was undertaken by the WDD, work having started in June 1967 and completed in December 1967. Two spreading grounds were constructed with clay core and sand

6. MANAGEMENT

The Project is operated by the Teratsia Irrigation Division which undertook to pay 25% of the cost of the dam over 10 years at 5% rate of interest, 33% of the cost of the spreading grounds and 50% of the cost of the concrete canals. The land commanded by the Teratsia Irrigation Division is about 7,000 donums of citrus plantations which are mainly irrigated from about 100 private boreholes.

Any water available from the dam is used as supplementary to that of the boreholes in order to

Precast concrete polycentric irrigation channels at Morphou.

gravel zones fitted with an overflow spillway. These spreading grounds suffered from a breach of one section during the excessive runoff of 1968. They were repaired in 1969.

A third spreading ground in the form of a gabbion dam across the river provided with an impermeable upstream clay blanket which was covered by a gravel blanket for protection against wave action was built between July and December 1969.

The total cost of the spreading grounds was £30,000.

The distribution system from the dam starts from the Teratsia Intake. The canals have been lined in concrete or prefabricated polycentric canals. Until 1969 42,600 m of concrete canals have been built at a total cost of about £130,000.

relieve the groundwater over extraction. The Irrigation Division also operates four boreholes which belong to the Division itself and which were drilled by the WDD at suitable sites near the spreading grounds so that they could be well recharged. These boreholes are being used to supply water to a depleted region of the Division, the groundwater table of which is below sea level. This supply enables the extraction of water from the private boreholes in that region to be reduced to its safe yield extraction.

7. PROBLEMS OF SPECIAL INTEREST

The groundwater recharge effect from the dam and spreading grounds has been quite significant.

The dam was filled on the 24th December 1962 to its full capacity of 2 million m^3. Measurements taken of the inflow, overflow and water level in the reservoir until the 23rd January showed that 4 million m^3 seeped through the reservoir bed with a maximum seepage of 330,000 m^3/day at full reservoir level. As the surface area of the reservoir is about 460,000 m^2, the maximum infiltration rate was 0.75 m^3/m^2/day. Observation wells showed that the water table rose from about 9 m near the dam to 1 m at 5 km downstream. A rising of the water table was also observed upto 1.5 m at 1.5 km upstream, 2 m at 1.5 km on the left side, and 1.5 m at 1.5 km to the right of the dam.

With time it was observed that the groundwater table near the dam was dropping whereas further downstream it was rising. There was a maximum rise near the coast of 0.30 m but the water table was still below sea level which indicated that there was no loss to the sea. Furthermore, the numerous boreholes in between the spreading grounds and the sea ensures that no losses occur.

At subsequent fillings of the reservoir, it was observed that the sediment transported decreased the rate of infiltration inspite of the efforts made to take away much of the silt for use in citrus gardens.

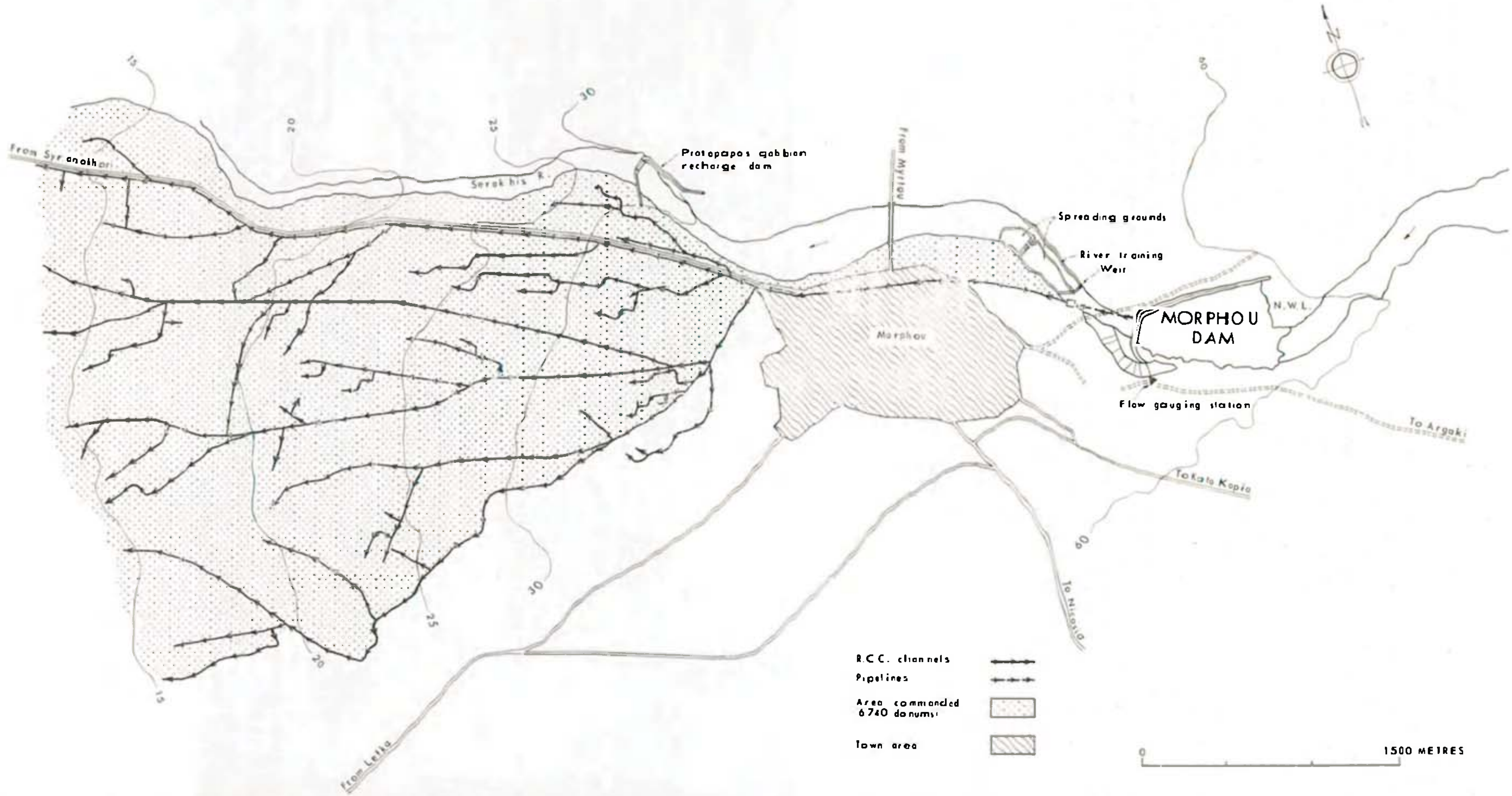
Therefore, to facilitate the rate of recharge it was decided to go ahead with the construction of a series of spreading grounds downstream of the dam.

The first inflow in the spreading grounds at the end of January 1968 was regulated at the controlling gate from the diversion weir to the reservoirs so that with an inflow of 1.5 m^3/s, the maximum water level in the reservoirs was maintained thus reaching equilibrium between the rate of inflow and the rate of infiltration. As the surface area of the gravelly base of the reservoirs is about 40,000 m^2 the rate of infiltration is 2.8 m^3/m^2/day. This rate of infiltration shows that the infiltration through the spreading grounds was about 3.5 times that through the dam reservoir. The reasons for the highest rate of infiltration can be attributed to the fact that water applied on the spreading grounds contained much less silt than the water in the dam reservoir, and that the gravel bed of the spreading grounds was made up of coarser gravels containing less silt than the dam reservoir.

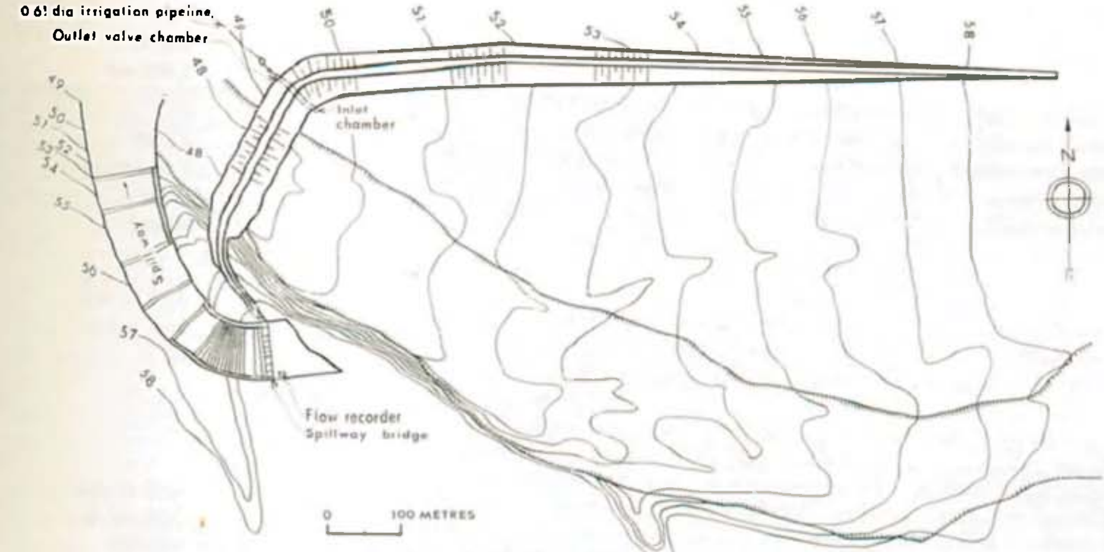
Another reason for the higher infiltration in the spreading grounds is the head of water which was maintained at maximum level throughout the spreading grounds whilst in the case of the dam it varies from maximum at the dam to zero at the tail end of the reservoir.

Sprinkler irrigation of citrus at Morphou.

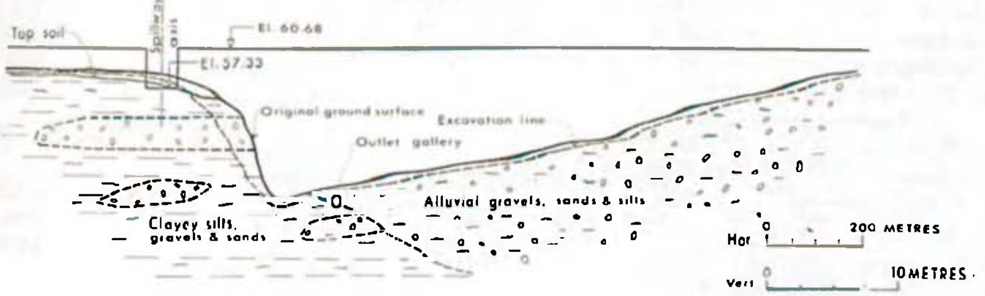
SERAKHIS-MORPHOU-DISTRIBUTION SYSTEM



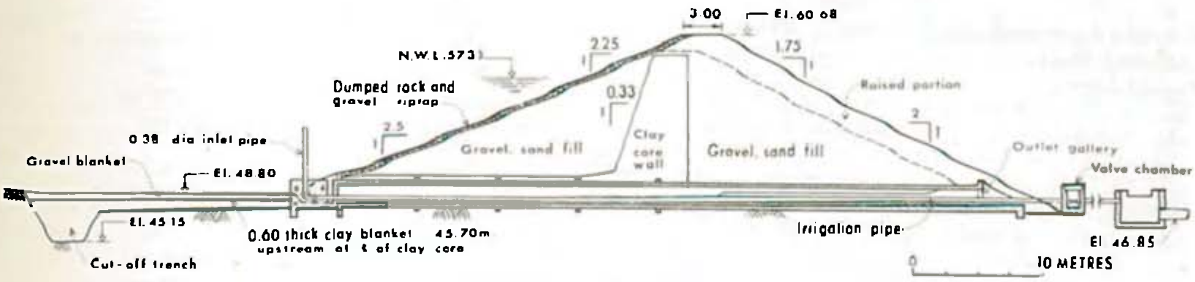
SERAKHIS-MORPHOU DAM



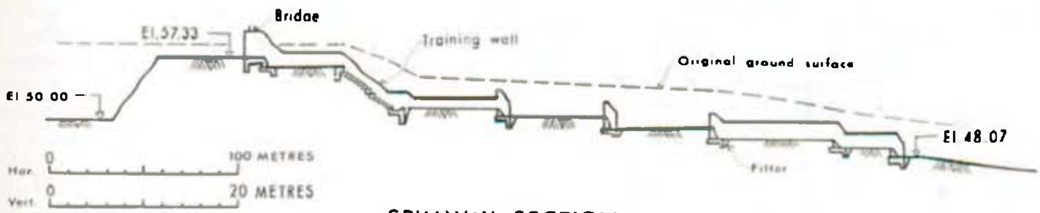
PLAN



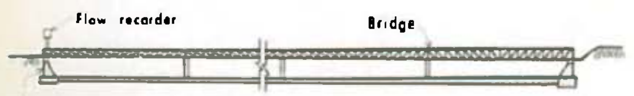
GEOLOGICAL SECTION ON DAM AXIS



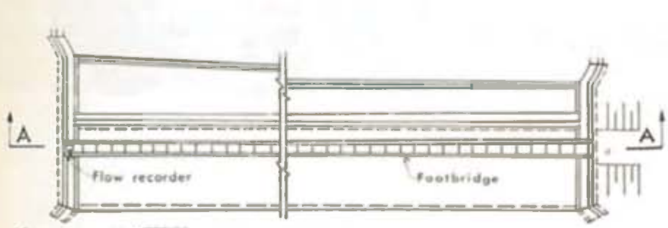
MAXIMUM SECTION



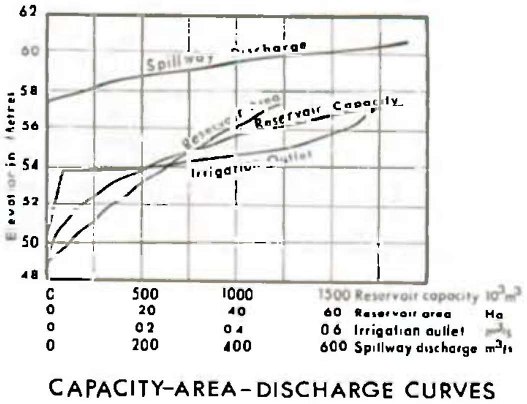
SPILLWAY SECTION



SECTION A-A



SPILLWAY PLAN

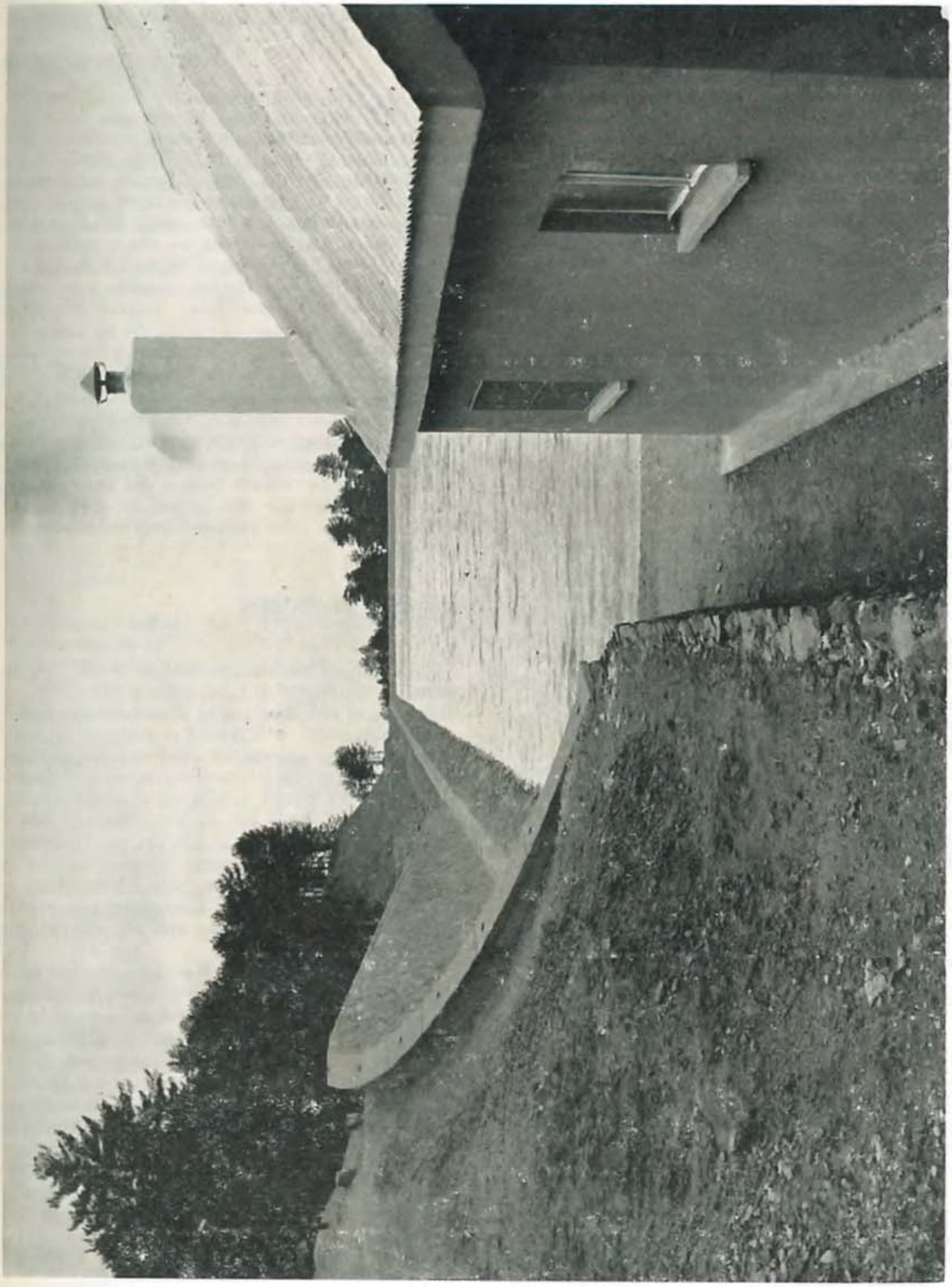


CAPACITY–AREA–DISCHARGE CURVES

Data

i.	***Catchment***	
	Area	458 km2
	Average rainfall	480 mm/a
	Average runoff	13 million m^3/a
	1/1000 years flood	1,400 m^3/s
	Maximum height	1,560 m
	Maximum length	44 km
ii.	***Reservoir***	
	Area	48 ha
	Capacity	2 million m^3
	Live storage	1.8 million m^3
	Length	975 m
iii.	***Embankment***	
	Structural height	15.53 m
	Height above ground level	11.88 m
	Hydraulic height	8.53 m
	Depth of foundation cut-off	3.65 m
	Freeboard	3.35 m
	Crest length	1400 m
	Top thickness	3 m
	Base thickness	56 m
	Upstream slope	1:2.5, 1:2.25
	Downstream slope	1:2, 1:1.75
	Upstream core slope	1:0.33
	Downstream core slope	Vertical
	Minimum core thickness	3 m
	Maximum core thickness	6.3 m
	Random fill	300,000 m^3
	Clay core	41,500 m^3
	Clay blanket and cut-off	24,000 m^3
	Gravel blanket	6,500 m^3
	Rock rip-rap	15,000 m^3
	Total earth fill	387,000 m^3
iv.	***Excavations***	
	Total	7.500 m^3
v.	***Spillway***	
	Size	75x2.1 m
	Capacity	680 m^3/s
	Length	450 m
	Concrete	3,400 m^3
vi.	***Gallery***	
	Size	1.3 m
	Capacity	For access
	Length	51 m
	Concrete	36 m^3
vii.	***Outlet (steel pipes)***	
	Size	0.30 m dia
	Capacity	0.11 m^3/s
	Length	60 m
viii.	***Distribution System***	
(i)	*Main conveyor*	
	Concrete pipes	
	Size	0.60 m dia
	Capacity	0.48 m^3/s
	Length	600 m
(ii)	*Concrete channels*	
	Size	0.9x0.6 to 1 2x0.6 m
	Capacity	0.48 m^3/s
	Length	1,890 m
(iii)	*Distribution Network*	
	(Concrete channels)	
	Size	0.23x0.45 to 0.45x0.9 m
	Capacity	0.08 to 0 28 m^3/s
	Length	40,200 m
ix.	***Spreading Grounds***	
(i)	*Twin reservoirs downstream of the dam*	
	Capacity	0.13 million m^3
	Clay fill	750 m^3
	Gravel and sand fill	15,000 m^3
	Concrete	380 m^3
(ii)	*Protopapas Gabbion dam*	
	Capacity	0.09 million m^3
	Gabbion wall	710 m^3
	Clay blanket	3,800 m^3
	Gravel and sand blanket	30,000 m^3

PRODHROMOS



The Prodhromos reservoir.

DHIARIZOS—PRODHROMOS PROJECT

1. PURPOSE

Prodhromos is the highest village in Cyprus after the Government settlement at Troodos and therefore the water resources are very limited and rather costly to develop. Although there is a lot of tourism and many people spend their summer vacations there, yet the main occupation of the village is irrigated agriculture producing mainly apples, pears, peaches, plums and cherries. The main sources utilized for this irrigation before the construction of the reservoir were the privately owned relatively big Hardji spring and a number of privately owned small springs from which about 180 donums of deciduous crops were irrigated. However, as the use of the Hardji spring was extended to accommodate domestic water supply requirements and more water was used to satisfy water use rights for irrigation of downstream villages, the need for additional supply of water for irrigation at Prodhromos was justified. As no other springs or stream summer flow were available for diversion, storage of winter water was the only possibility. However, due to the high altitude and very steep river beds, the construction of a dam similar to that at Agros would present even bigger problems of availability of storage and high rate of silting up. It was therefore decided to build an off channel reservoir which would be supplied through a diversion intake from the nearby Platania stream tributary of the Dhiarizos river conveying mainly the melting snow water of its catchment during the months of February, March and April.

As the availability of snow might not always meet the requirements of the reservoir, a pumping scheme was also incorporated to elevate surplus winter water from a lower point of the Hardji spring canal into the reservoir.

2. LOCATION

The reservoir is situated in the Damaskinari Forest about halfway to Troodos near the main road. It is at an altitude of 1,570 m asl.

3. PLAN

a. Water and Land

The average rainfall on this catchment is 1,070 mm/a which yields 0.42 million m^3/a as an average annual runoff. The maximum flood estimated at the weir at a frequency of 1/1000 years is about 15 m^3/s.

The diversion weir provides water from snowmelt of the Platania stream tributary of the Dhiarizos river which had not previously been utilized. The additional water which may be pumped from the Hardji spring has been previously used by Prodhromos Irrigation Division and an extensive lined canalization system exists.

b. Geology

The whole catchment lies on the Troodos igneous massif. More specifically the reservoir itself lies on ultrabasic igneous rock. Most of the site is formed of highly fissured and brecciated ultrabasic igneous rock partially decomposed with coarse sands and clays within the fissures. A big part of the site is made up almost entirely of fine clay which has been formed of the decomposition of the ultrabasic igneous rock under glaciation conditions. For design purposes a number of testpits were excavated on the site for the determination of the soil graduation, permeability and availability of material for the earthworks. The extent of the clay found made the design of a clay blanket to cover the whole reservoir as a more economic alternative than the use of other lining materials such as asphaltic sheets which had been seriously contemplated. Rock for the rip-rap had to be brought from a site about 1.5 km towards Troodos whilst the gravel and sand for the rip-rap filter had to be imported from the seashore 80 km away.

4. MAIN FEATURES

a. Dam

This project was designed by the WDD. The reservoir is the largest in Cyprus, being 270 m long by 96 m wide by 6 m deep and of a total storage capacity of 0.11 million m^3. It is formed of a random fill earth embankment, the material for which was obtained from excavating the site itself to form the reservoir. The mountain side of the Eastern part of the reservoir made up one of the sides of the reservoir. A clay blanket from material obtained also from the site was placed adjacent to the random fill embankment and reservoir bed to eliminate seepage losses through the embankment or the reservoir bed. This clay blanket was protected by a hand placed rip-rap provided with a gravel and sand filter zone.

A system of drains was provided under the reservoir bed to keep the base clear of any stagnant water and releave it from any hydrostatic pressure which might develop. An overflow pipe 0.20 m dia was provided. Any overflow can also be regulated before entering the reservoir at the settling tank or the diversion weir. The three outlet pipes from the reservoir, the washout and the drainage pipe were housed into a concrete gallery which was built through the embankment. The settling tank built to receive the water from the weir before entering the reservoir acts as a silt trap in order to keep the water entering the reservoir as clear as possible.

b. Distribution System

The Irrigation System is made up wholly of steel pipes designed by the WDD. It provided enough pressure at every outlet for the application of any system of irrigation. It is interconnected with the Hardji spring canalization system and commands an additional 170 donums as an extension to the original 180 donums irrigated from the spring.

5. CONSTRUCTION

The reservoir was constructed by the WDD between March and December 1962. No serious difficulties were encountered apart from some minor slides of the mountainous Eastern side. Also care had to be taken during winter freezing conditions to protect the concrete works under construction. The compaction of the clay blanket was done by sheepfoot rollers and that of the random fill by vibrating roller.

The total expenditure on the reservoir, deversion intake, conveyor pipeline and pumping scheme was £80,000.

The irrigation system was also constructed by the WDD between December 1963 to June 1964. The total cost of the distribution system including the pumping installation from Hardji spring and the rising main to the reservoir was £27,000. As the land to be irrigated was very steep terrain, land levelling and bench terracing on which the deciduous trees were planted, was carried out.

6. MANAGEMENT

The project comes under the operation of the Prodhromos Irrigation Division which undertook to contribute 25% of the total cost having obtained a loan from the Government payable in 20 years at 6% rate of interest. The land commanded by the distribution system of the reservoir is 170 donums being an extension of the 180 donums irrigated from the Hardji spring.

7. PROBLEMS OF SPECIAL INTEREST

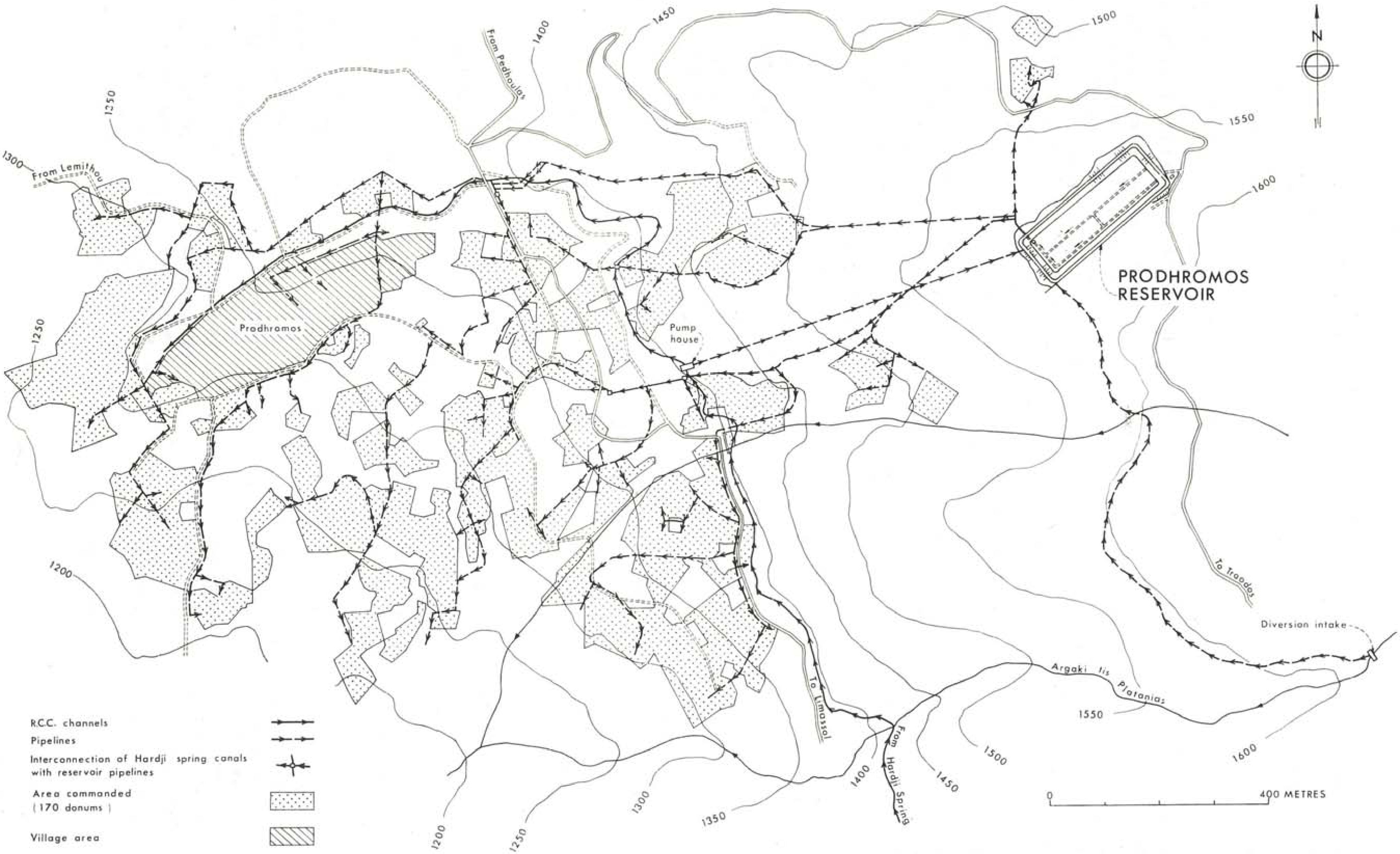
No special problems arose after construction. The silt trap operated well and the reservoir was proved to be completely water tight. The Eastern mountain side has to be well drained by keeping the concrete channel surface drain operating to avoid running water from the hillside soaking the reservoir side excessively and thereby causing local landsliding.

The main problem was the unreliable flow of the Platania stream which could not fill the reservoir every year. Thus a pumping scheme was carried out which enabled the pumping of water from Hardji spring during winter months when surplus was available and stroring it in the reservoir for use during summer. By Februa ry every year a decision is taken whether pumping from the spring is required.

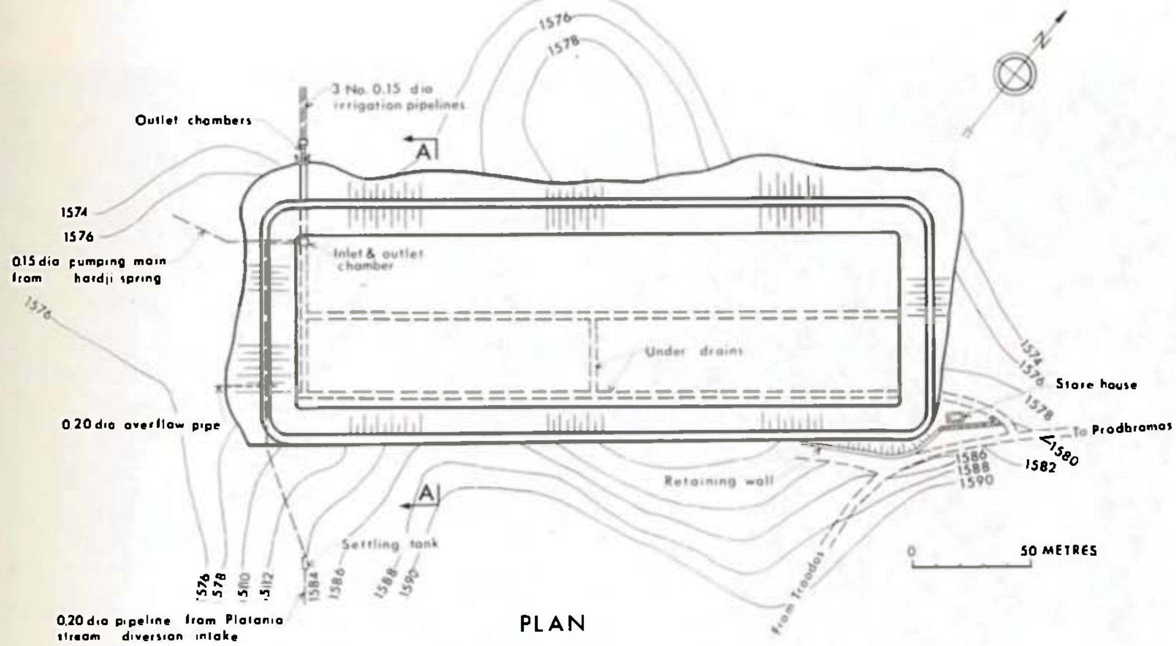
Data

i.	*Catchment*	
	Area	1 km²
	Average rainfall	1,070 mm/a
	Average runoff	0.42 million m³/a
	1/1000 years flood	15 m³/s
	Maximum height	1,800 m
	Maximum length	1,300 m
ii.	*Reservoir*	
	Area	2.6 ha
	Capacity	0.11 million m³
	Live storage	0.11 million m³
	Size	270x96 m
iii.	*Embankment*	
	Structural height	10 m
	Hydraulic height	6.34 m
	Freeboard	0.91 m
	Crest length	732 m
	Top thickness	3 m
	Base thickness	42 m
	Upstream slope	1:2
	Downstream slope	1:2
	Downstream core slope	1:2
	Upstream core slope	1:2
	Minimum core thickness	3.3 m
	Maximum core thickness	3.3 m
	Random fill	25,200 m³
	Clay core	43,000 m³
	Gravel, sand filter	3,700 m³
	Rock rip-rap	2,900 m³
	Total earth fill	74,000 m³
iv.	*Excavations*	
	Total	176,000 m³
v.	*Spillway (Overflow pipe)*	
	Size	0.20 m dia
vi.	*Gallery*	
	Size	1.3x1.2 m
	Capacity	For access
	Length	33 m
	Concrete	38 m³
vii.	*Outlets (steel pipes)*	
	Irrigation	
	Size	0.10 m dia
	Capacity	0.07 m³/s
	Length	50 m
	Washout	
	Size	0.10 m dia
	Drains	
	Size	0.15 m dia
viii.	*Distribution System*	
(i)	*Main Conveyor (from Platania Diversion Weir)*	
	Size	0.20 m
	Capacity	0.03 m³/s
	Length	1,400 m
(ii)	*Distribution Network (steel pipes)*	
	Size	0.05, 0.075, 0.10 m dia
	Capacity	0.0075 to 0.34 m³/s
	Length	13,000
ix.	*Pumping Scheme*	
	Pump engine	140 h.p.
	Pump capacity	0.034 m³/s
	Pumping head	200 m
	Length of rising main to reservoir	630 m
	Diameter of rising main	0.15 m

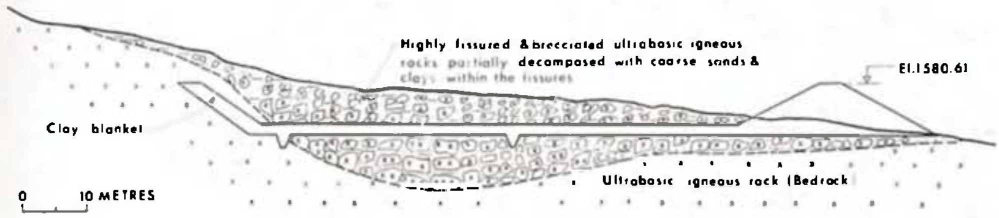
DHIARIZOS - PRODHROMOS-DISTRIBUTION SYSTEM



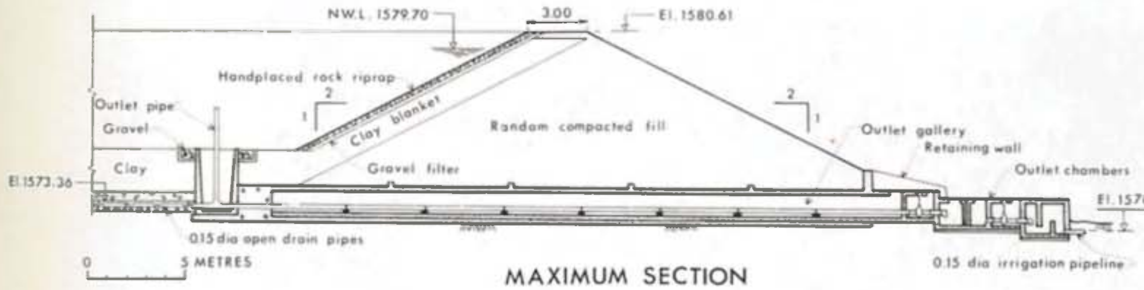
DHIARIZOS-PRODHROMOS RESERVOIR



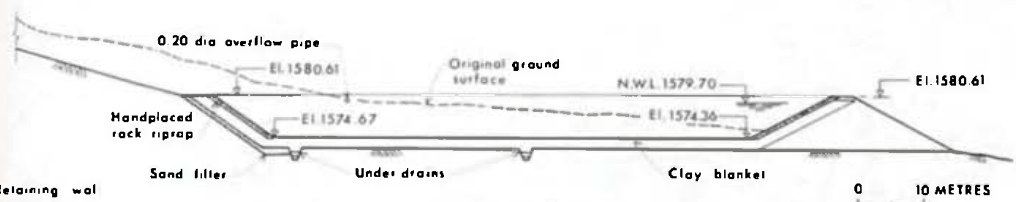
PLAN



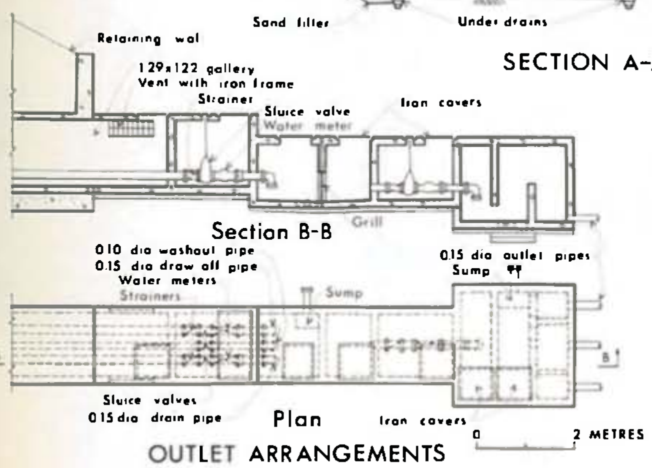
GEOLOGICAL SECTION ON DAM AXIS



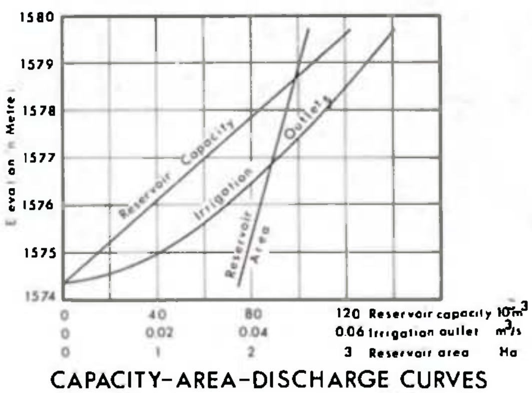
MAXIMUM SECTION



SECTION A–A

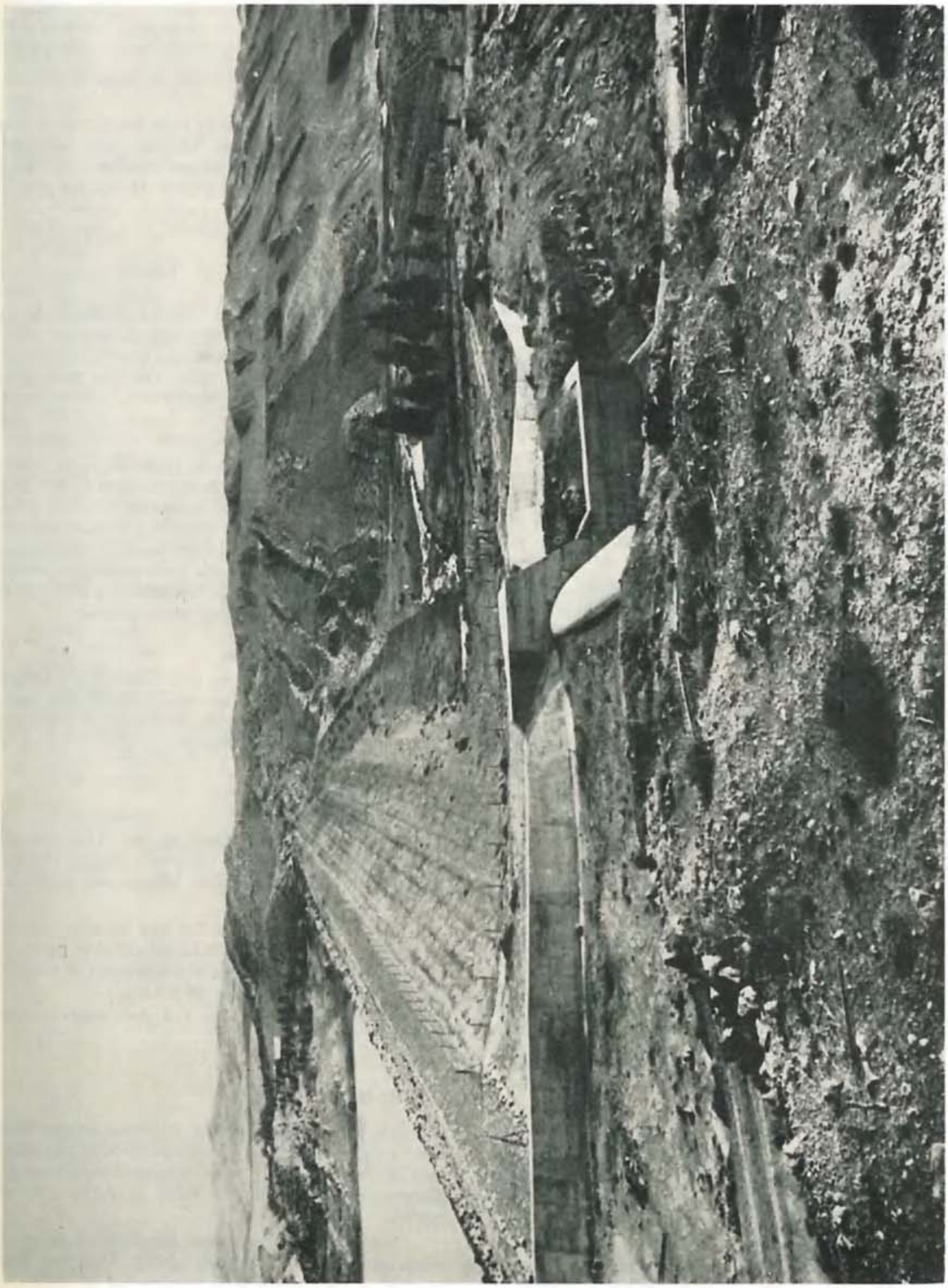


OUTLET ARRANGEMENTS



CAPACITY–AREA–DISCHARGE CURVES

KANLI KEUY



The Kanli Keuy dam.

PEDHIEOS—KANLI KEUY PROJECT

1. PURPOSE

This dam provides additional summer water which becomes available by storage of the spate flow of the Jinar tributary of the Pedhieos river. Kanli Keuy village could only grow wheat and barley prior to the construction of the dam. A small earth dam 4 m high of a storage capacity 0.2 million m^3 and at a cost of £500 was constructed in 1949 mainly as a soil conservation scheme. The stored water was used for supplementary irrigation of barley and wheat in March and April, and for some early summer crops until June. After June the empty reservoir flooded lands were used for summer and autumn crops which would grow without irrigation due to the amount of moisture which could be retained in the slow infiltrating soils of the reservoir.

This small dam lost about 9% of its capacity through silting up in the first winter of its construction 1949—1950 which was a year of excessive rainfall well above average. By 1963 when the new dam was constructed on the same site to replace the small one the total loss of capacity through silting up was well over 30%. The loss of capacity as well as the increased water demand were the deciding factors for the construction of the new dam.

2. LOCATION

The dam is located on the Jinar tributary of the Pedhieos river just upstream of the Kanli Keuy village and at an altitude of about 180 m asl.

3. PLAN

a. Water and Land

The average rainfall on this catchment is 76 mm/a resulting in an average annual runoff of about 0.7 million m^3. The maximum flood estimated at the damsite at a frequency of 1/1000 years is of the order of 35 m^3/s. Water use rights were exercised on this stream by an Irrigation Division of farmers from Yerolakkos and Kanli Keuy. In 1958 the Yerolakkos farmers abandoned their rights. The new dam was built to satisfy the Kanli Keuy Irrigation Division's water requirements only.

b. Geology

Most of the catchment and more specifically the damsite and reservoir fall within the Kythrea Formation which is quite water tight. No deep excavation problems were necessary and the cut-off was based on a sound water tight marl of the Formation. However, the serious problems of this formation are leaching salts and boron as well as heavy sediment transportation. Chemical analyses taken from the old small dam showed 413 ppm NaCl and 410 ppm total hardness. On the other hand the sediment problem was quite obvious from the records available on the old dam.

4. MAIN FEATURES

a. Dam

This dam was designed by the WDD. It is an earth dam formed of a homogeneous clayey soil provided with a downstream gravel and sand filter blanket and toe. A cut-off trench filled with clay goes down to the sound marl rock foundation. Limited hand-placed rip-rap above and below the normal water level has been used on the upstream side of the embankment.

Because the spillway rock appeared to be sound, and not many overflow floods were expected, the spillway was left unlined at the beginning but later on in 1967 due to heavy overflow floods it was lined throughout and provided with a stilling basin at downstream river bed level. A 0.30 m dia steel outlet pipe with a perforated inlet pipe and a downstream control valve has been provided supplying irrigation water.

b. Distribution System

This system is made up of a main steel conveyor supplying water into a masonry channel and earth channel distribution network commanding the 4,000 donums of the Irrigation Division.

5. CONSTRUCTION

This dam was constructed by the WDD between April and December 1963. It was a simple homogeneous earth bank construction without any particular difficulties.

The compaction of the fill was done in 0.15 m layers to a dry density of 1.15 kg/m^3 and at an optimum moisture of 18%. The lining of the spillway in concrete was done in 1967 at a cost of £4,000.

The total expenditure on this dam reached about £34,000.

6. MANAGEMENT

This project is under the operation of the Kanli Keuy Irrigation Division, which undertook to contribute 25% of the total cost, having obtained a loan from the Government payable in 15 years at 5.25% rate of interest.

The land belonging to the beneficiaries which is commanded by the irrigation system from the dam is about 4,000 donums mostly cultivated in cereals.

7. PROBLEMS OF SPECIAL INTEREST

The unlined spillway erosion which occurred by the flood of 1967 necessitated the concrete lining of this spillway throughout its length provided with a stilling basin at river bed level.

The increase of salinity and boron concentration of the water in the reservoir has to be closely watched. Every few years the reservoir may have to be emptied to avoid excessive concentrations.

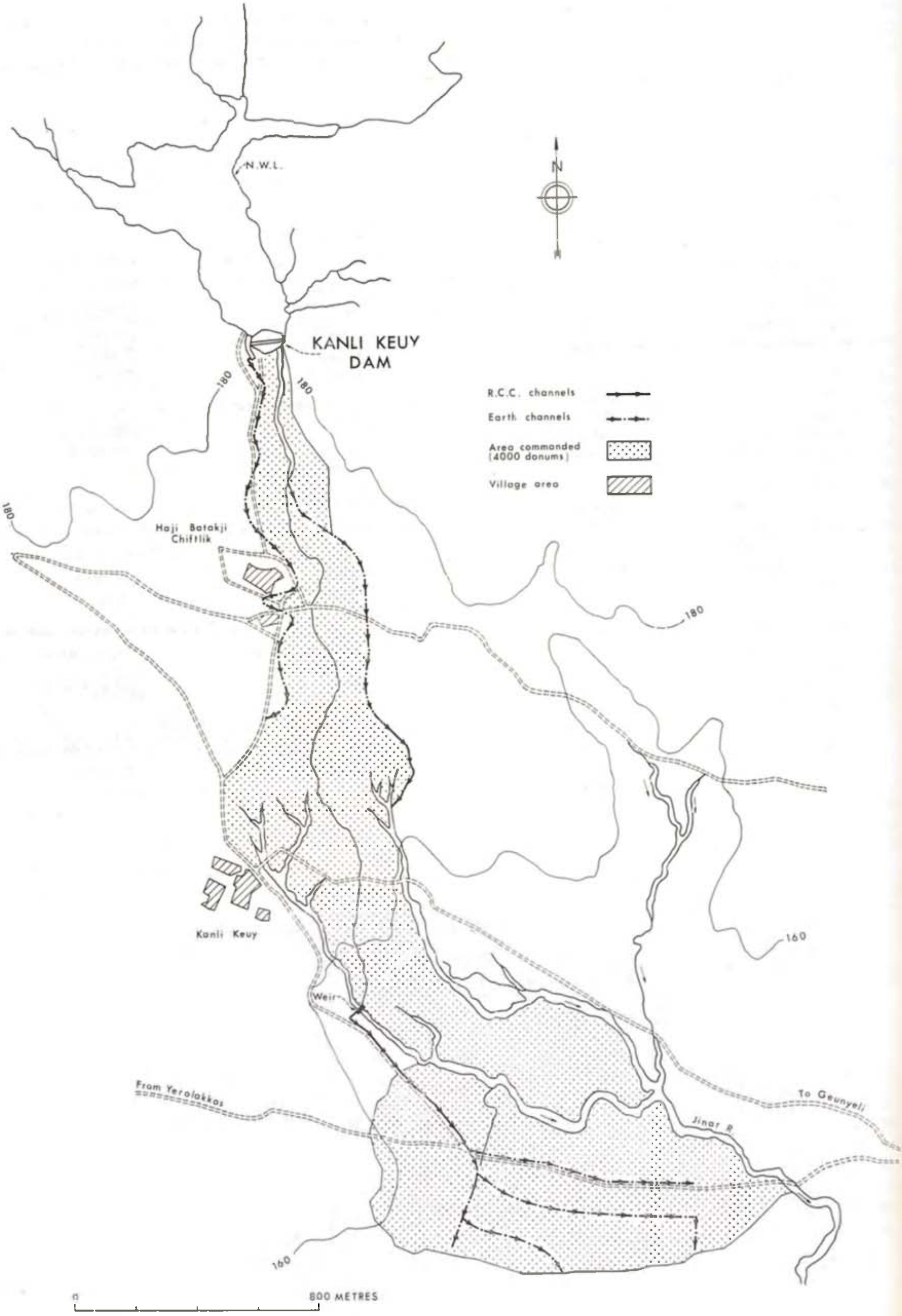
In April 1968 after 5 years of continuous storage, the quality of the water in the reservoir was:

NaCl 400 ppm, $CaSO_4$ 180 ppm and Boron 0.55 ppm.

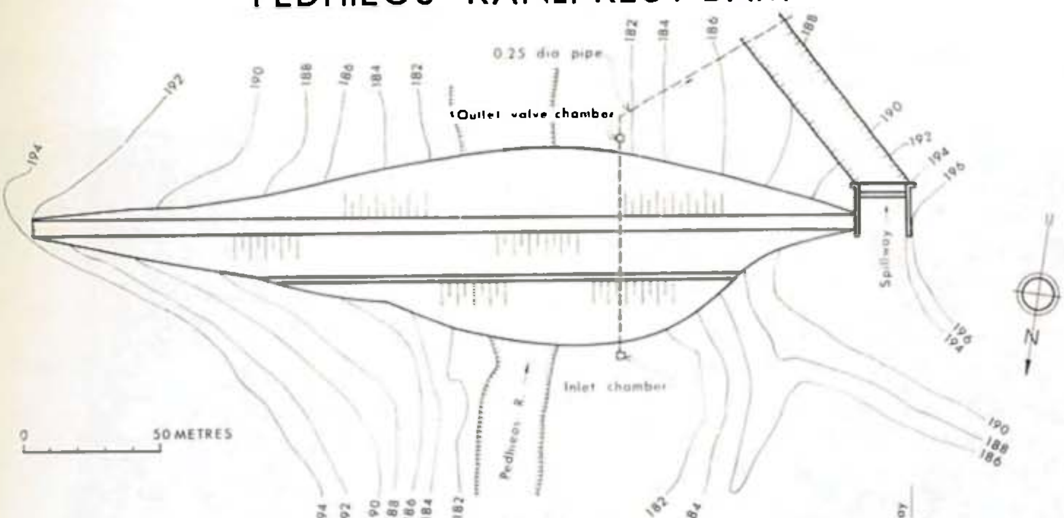
Data

i.	*Catchment*	
	Area	33 km^2
	Average rainfall	476 mm/a
	Average runoff	0.7 million m^3/a
	1/1000 years flood	35 m^3
	Maximum height	900 m
	Maximum length	9.5 km
ii.	*Reservoir*	
	Area	39 ha
	Capacity	1.13 million m^3
	Live storage	1.00 million m^3
	Length	1,356 m
iii.	*Embankment*	
	Structural height	19.5 m
	Height above ground level	12.36 m
	Hydraulic height	10.53 m
	Depth of foundation cut-off	6.70 m
	Freeboard	1.83 m
	Crest length	297 m
	Top thickness	6 m
	Base thickness	70.10 m
	Upstream slope	1:3, 1:2.75
	Downstream slope	1:2
	Random fill	47,412 m^3
	Gravel sand filter	2,294 m^3
	Rock rip-rap	2,294 m^3
	Total earth fill	52,000 m^3
iv.	*Excavations*	
	Total	45.840 m^3
v.	*Spilluay*	
	Size	18.20x1.83 m
	Capacity	110 m^3/s
	Length	27.43 m
	Concrete	474 m^3
vi.	*Outlet (Steel pipes)*	
	Size	0.30 m dia
	Capacity	0.38 m^3/s
	Length	80 m
vii.	*Distribution System*	
(i)	*Main Conveyor (Steel pipes)*	
	Size	0.25 m dia
	Capacity	0.56 m^3/s
	Length	110 m
(ii)	*Distribution Network (Earth channels)*	
	Size (Trapezoidal)	(1.52+0.60)x0.45 m
	Capacity	0.56 m^3/s
	Length	4575 m
(iii)	*Masonry Channels*	
	Size (Trapezoidal)	(1.52+0.60)x0.45 m
	Capacity	0.56 m^3/s
	Length	690 m

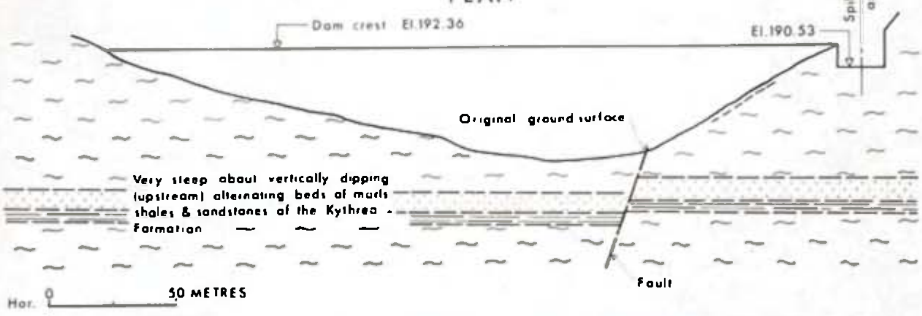
PEDHIEOS-KANLI KEUY-DISTRIBUTION SYSTEM



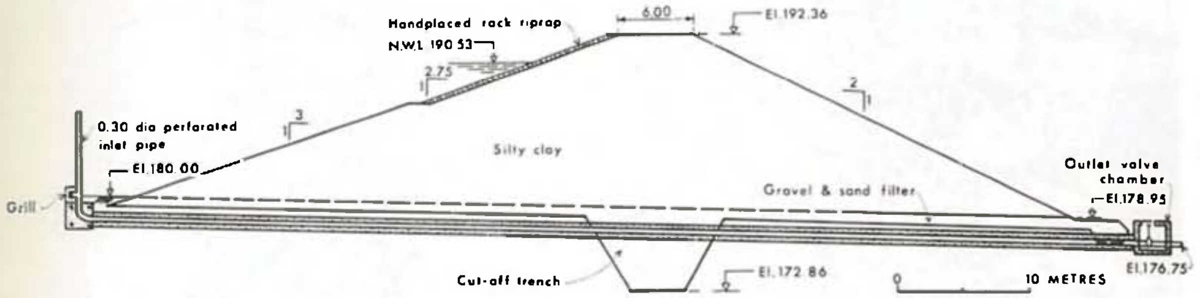
PEDHIEOS - KANLI KEUY DAM



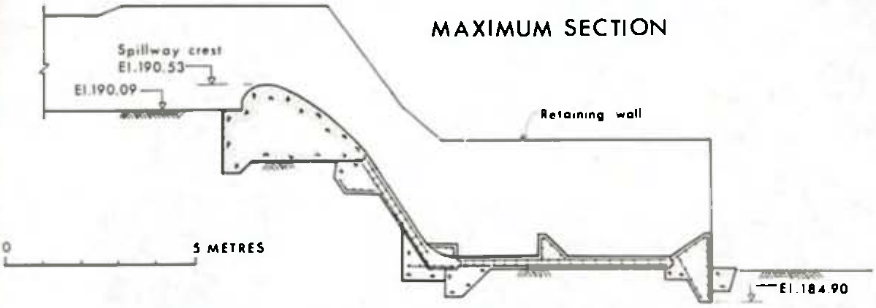
PLAN



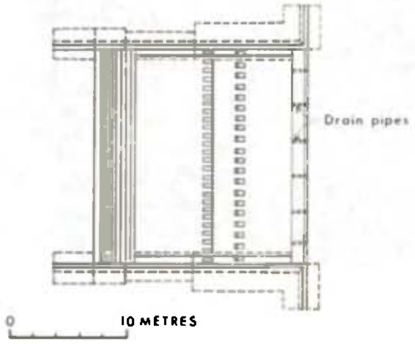
GEOLOGICAL SECTION ON DAM AXIS



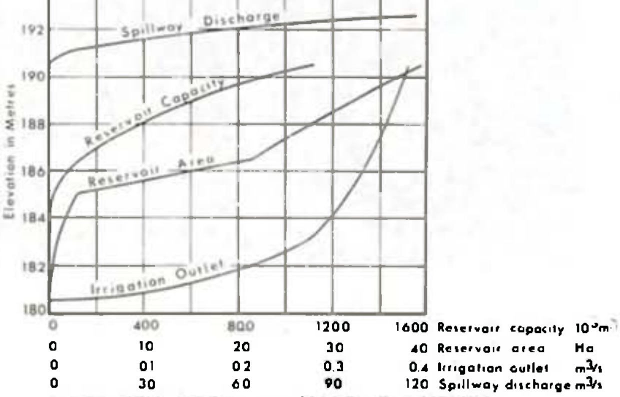
MAXIMUM SECTION



SPILLWAY SECTION

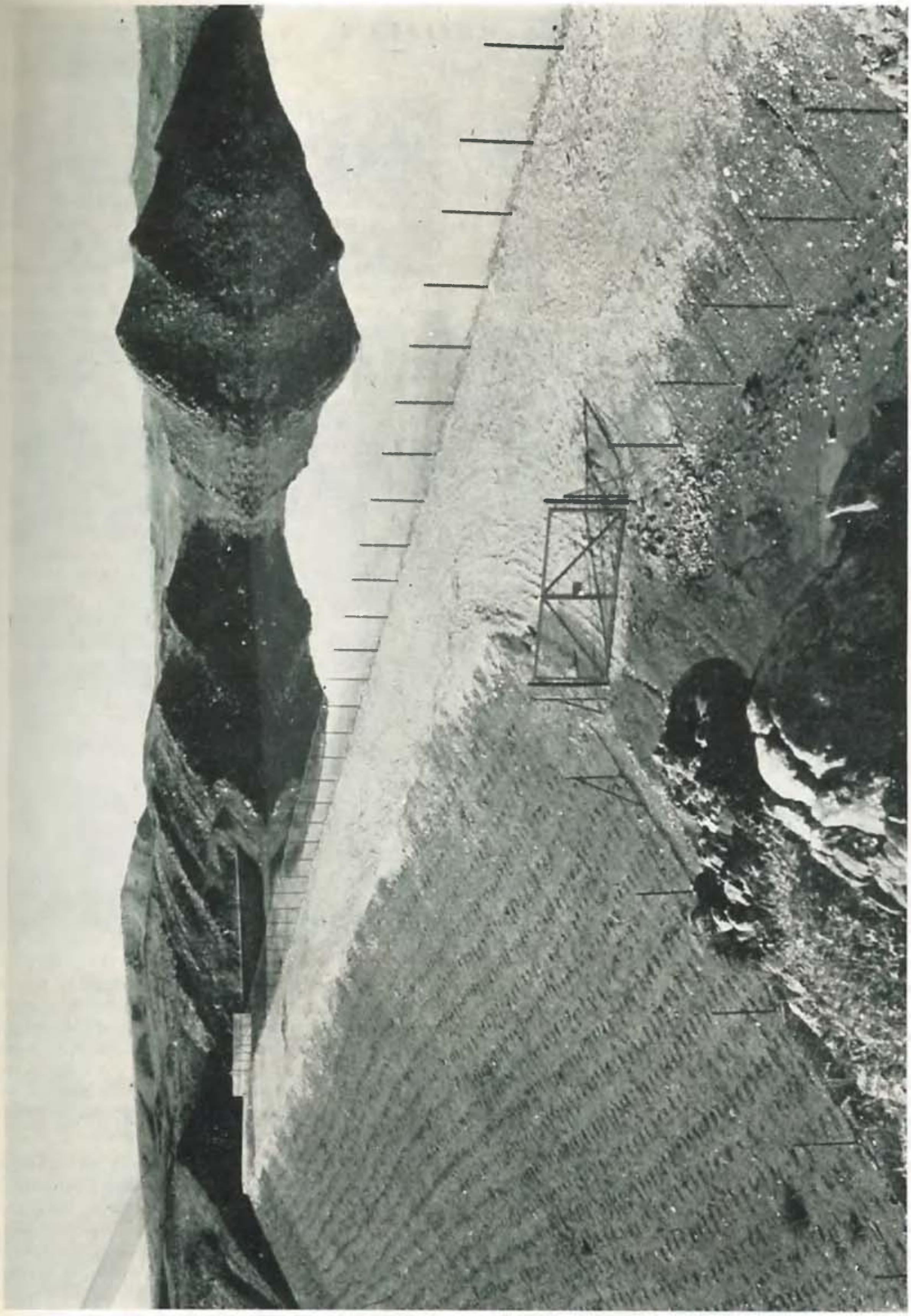


SPILLWAY PLAN



CAPACITY-AREA-DISCHARGE CURVES

MIA MILEA



The Mia Milea dam.

PEDHIEOS — MIA MILEA PROJECT

1. PURPOSE

Mia Milea village lies on the foothills of the Kyrenia range on the Kythrea Formation where no groundwater of any significance can be found.

The surface water resources of the region are rather scarce but could be considered significant for the irrigation of cereals and early summer crops. In this village the main income is from dry farming, and the conservation and utilization of the surface water is of importance for the development of this region.

The main purpose of this dam is to store the water of the Symeas tributary of the Pedhieos river during winter floods, and thereafter use it for irrigation during spring and early summer months. The great evaporation losses during summer and the spate flow of the stream requires that most of the water is used as early as possible and preferably before July. The use of this water for irrigation should be for crops such as potatoes and other vegetables as well as for supplementary irrigation required for cereals during the months of March and April.

2. LOCATION

The dam is located on the Symeas tributary of the Pedhieos river at a distance of about 3 km upstream of the Mia Milea village. It is situated at an altitude of about 150 m asl.

3. PLAN

a. Water and Land

Automatic flow recorders are not available on the stream but from other nearby rivers and based on meteorological data it has been estimated that the average runoff of this stream at the damsite is of the order of 0.16 million m^3/a. The maximum flow estimated at the dam site at a frequency of 1/1000 years is estimated to be of the order of 23.7 m^3/s.

Water from this stream has been used by the people of Mia Milea who have an Irrigation Division for managing the intake and earth channels emanating from the stream. The dam was built to supply water to this intake and has been taken over by the Irrigation Division for management purposes. The irrigation practised before the construction of the dam from this intake was spate irrigation of cereals and olive trees which are the main crops of the land belonging to the beneficiaries. The channels supplying water to the area were excavated and are still in an unlined state. This dam is supplementary to a similar smaller dam on an adjacent smaller stream.

b. Geology

The general geology of the catchment is characterised by the Kythrea Formation in the lower region, and the Hilarion Limestone surrounded by the Lapithos chalks in the upper part of the catchment. The Kythrea Formation consists of alternating beds of marls, sandstones and shales which are densely folded along an EW axis. This Formation which is sloping towards the plain is ideal for storage both from the topographical and geological aspects. Topographically we can get a significant storage, whilst from the geological point of view the foundations and abutments can be considered as almost water tight. Apart from the limited permeability and jointing in the confined sandstone beds no other leakage problems exist. Investigations have been carried out to reach the impermeable marly beds, overlaid by alluvial gravels, sands and silts. The cut-off trench and the excavations did not exceed 6 m from the original river bed at it's deepest. The fault plane on the right hand side of the dam site did not develop into a leakage problem. The problem of silt deposition in the reservoir can be considered serious because the Kythrea Formation is highly erosive and in heavy rainfall a lot of silt is transported in the river. A problem of these rocks is the connate salts of the Kythrea Formation, including high chloride and carbonate concentrations and which under erosion conditions result in the rising of the salts and hardness of the water in the reservoir.

4. MAIN FEATURES

a. Dam

The design work for the dam was done by the WDD.

The dam was made up of a clay core wall in the centre reaching into the impermeable foundation and made of selected clay from the site. A gravel and sand filter blanket on the downstream side of the clay core wall was applied. A random fill zone was used on either side of the clay core wall, the material for which was obtained from the excavations at the site. A selected rock hand placed rip-rap has been placed on the upstream side of the dam, around the maximum water level, to protect the face of the dam from erosion which would be caused by the waves. A concrete weir with a chute coming down on a stilling basin at river bed elevation to divert the over flow, was built as a spillway on the right abutment.

A 0.30 m dia steel pipe has been laid across the dambody from the upstream to downstream side conveying water into the irrigation channels. The intake arrangement is made of a vertical perforated pipeline. The pipeline crossing the dambody has been embedded in concrete with suitable concrete collars at intervals. The operation of this irrigation pipe is done through a sluice valve on the downstream side.

b. Distribution System

This is a network of earth channels which are under consideration to be lined in concrete when proper utilization of the water is achieved.

5. CONSTRUCTION

The construction of the dam was carried out by the WDD between April 1963 and April 1964. All material except aggregates for concrete were obtained from the site. The clay core was compacted using sheepfoot rollers. The random fill was compacted with a sheepfoot roller.

The control of the compaction was done by the use of field laboratory equipment at the site. The total expenditure on the dam was about £28,000.

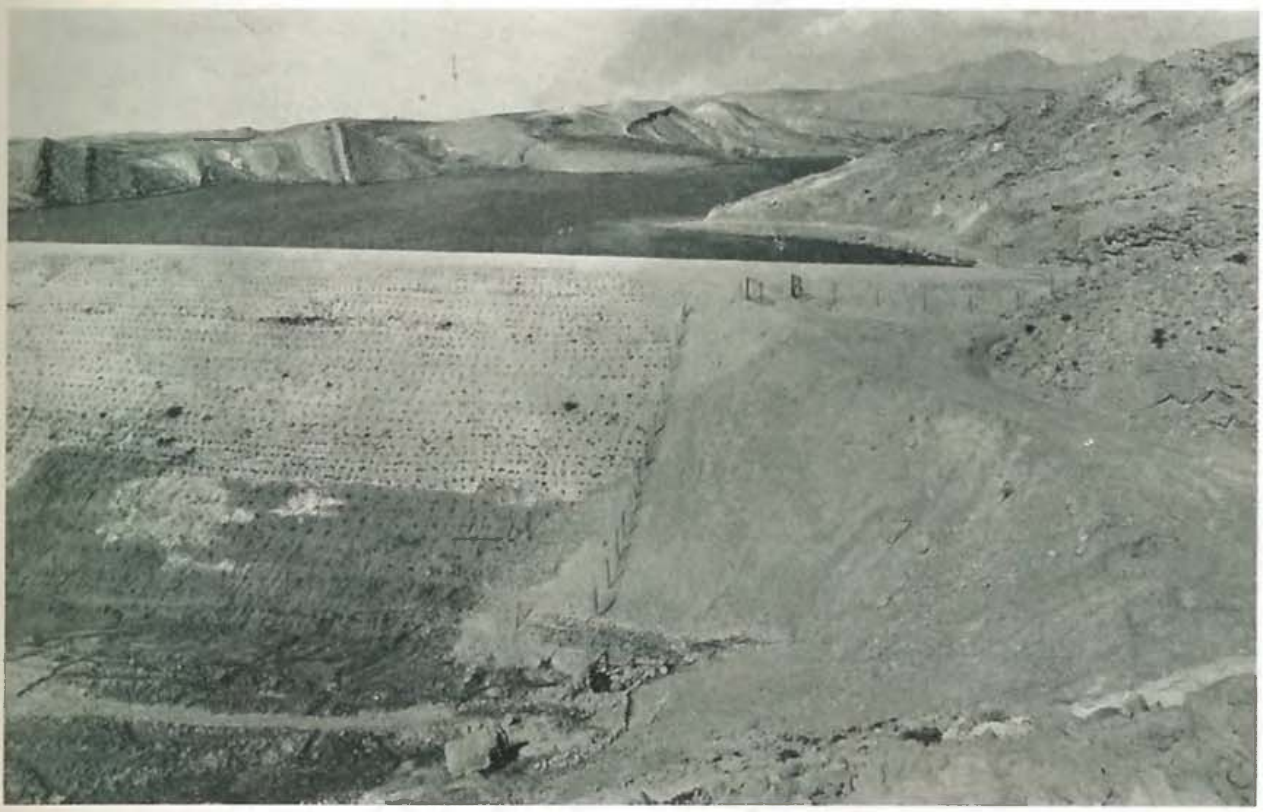
6. MANAGEMENT

The Project is operated by the Irrigation Division, which has contributed 25% towards the total cost, having obtained a loan from the Government, payable in 15 years at a rate of interest of 5.25%.

The land commanded by the distribution network is about 1,300 donums and the main crops are vegetables, potatoes, wheat and olive trees.

7. PROBLEMS OF SPECIAL INTEREST

Immediately after impounding a serious leakage developed on the left abutment of the dam. After careful examination it was found that the leakage occurred through a joint of the sandstone beds of the Kythrea Formation which found it's way into the reservoir. Gravel fill was placed at the region of the leakage, in order to filter the water passing through the abutment. The leakage, in the meantime, was reduced from about 5.2 l/s in 1964, to about 1.5 l/s in 1969, at full reservoir head, due to silting on the upstream face of the reservoir. The total leakage in an irrigation season from January to June is 13,000 m^3, i.e. 3.75% of the capacity of the reservoir. The total hardness of the water has increased from 80 ppm in 1964, to 680 ppm in 1969 and the sodium chloride content from 30 ppm in 1964 to 460 ppm in 1969.



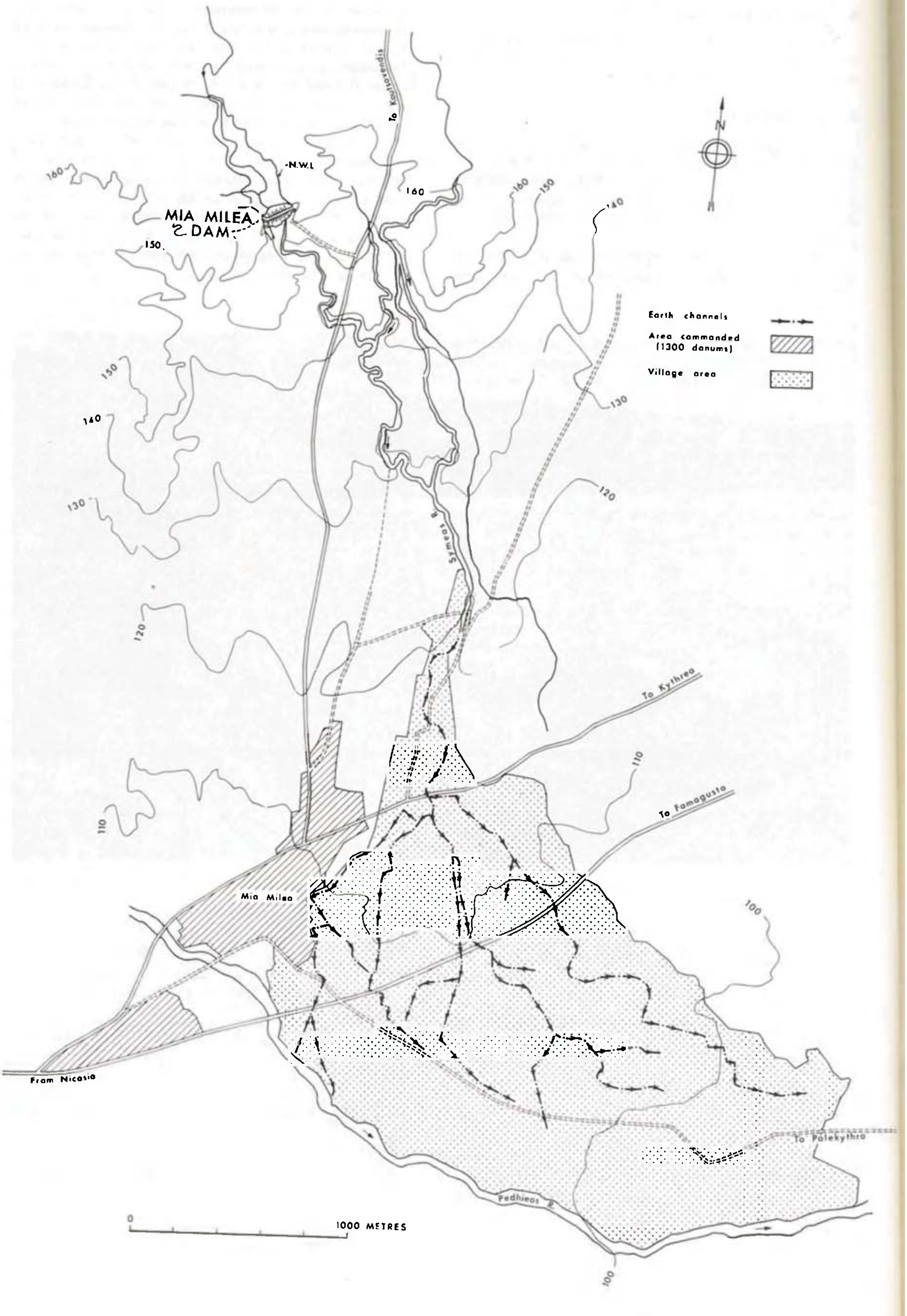
Seepage on the left abutment with drainage and weighing of toe.

This increase is due to the chloride and calcium contents of the Kythrea Formation which has been transported through erosion of the rocks into the reservoir.

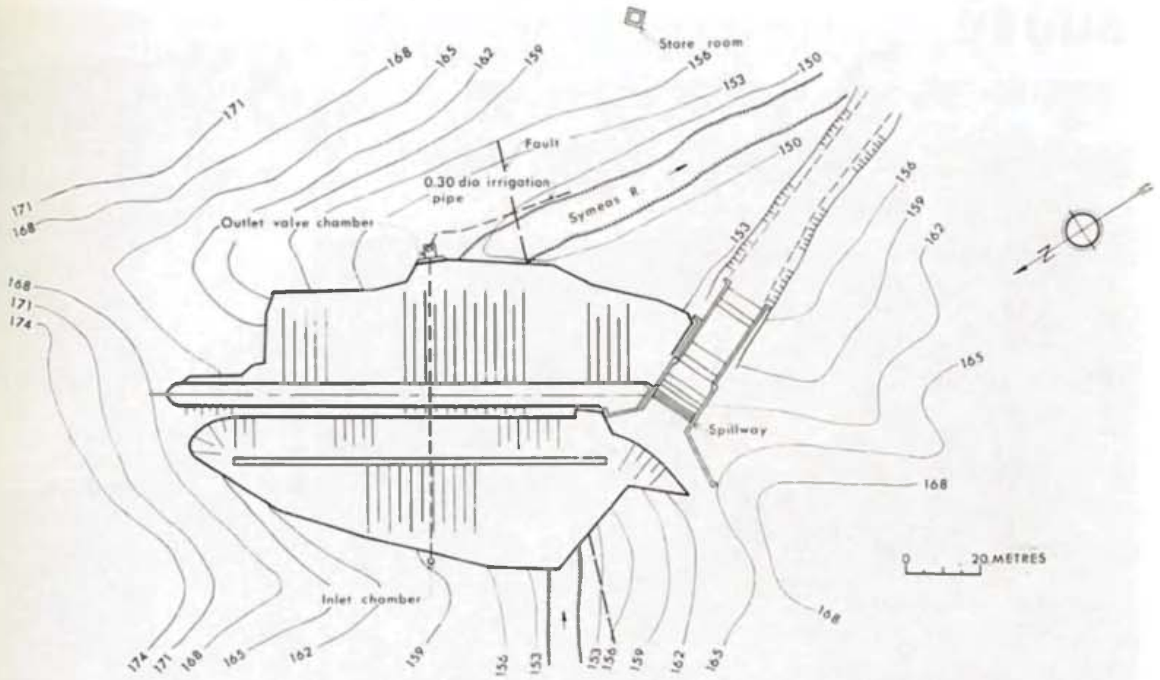
The deterioration of the quality of the water with time and storage is one more reason in favour of an annual operation of the dam with a short irrigation period preferably until June, thereafter emptying the reservoir to receive fresh water in the coming winter season.

Another problem was the erosion of the soft sandstone bedrock of the spillway channel which was subsequently lined in concrete.

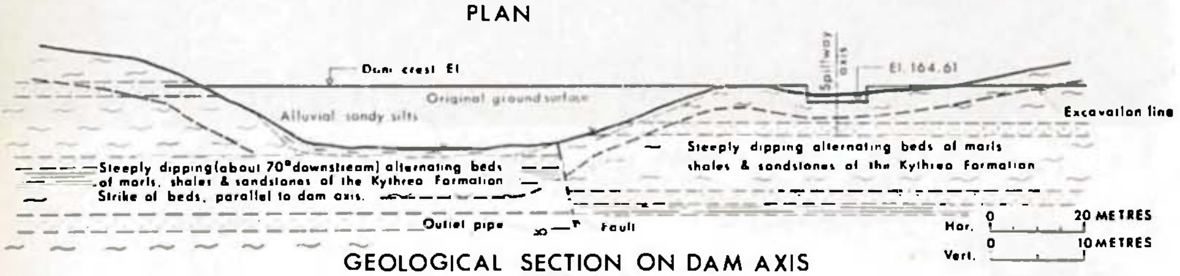
PEDHIEOS-MIA MILEA-DISTRIBUTION SYSTEM



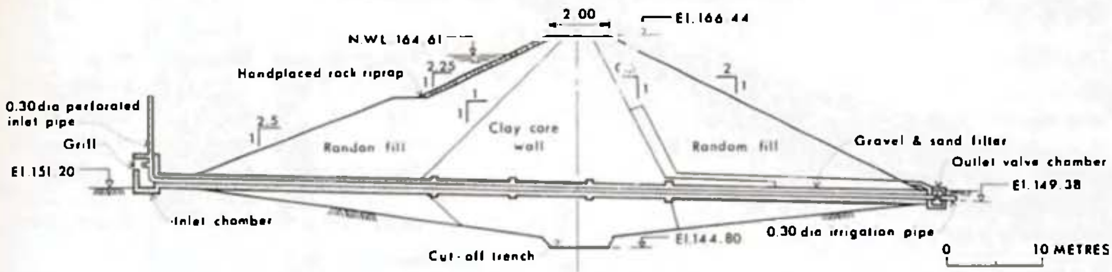
PEDHIEOS-MIA MILEA DAM



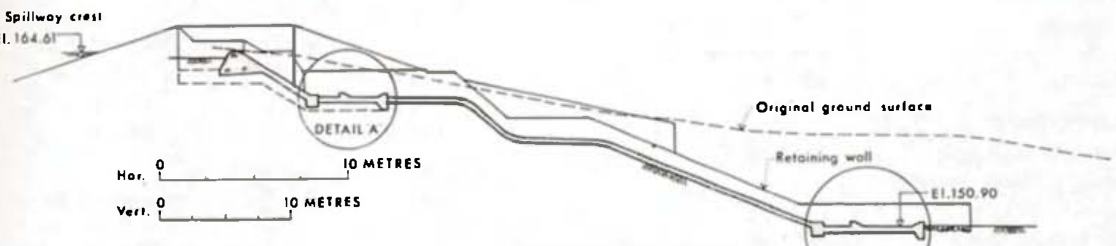
PLAN



GEOLOGICAL SECTION ON DAM AXIS

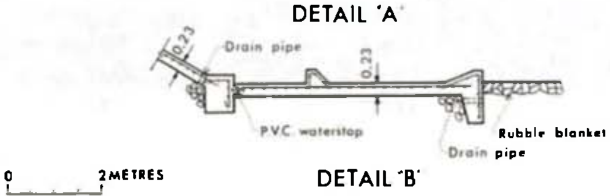


MAXIMUM SECTION



SPILLWAY SECTION

DETAIL 'A'



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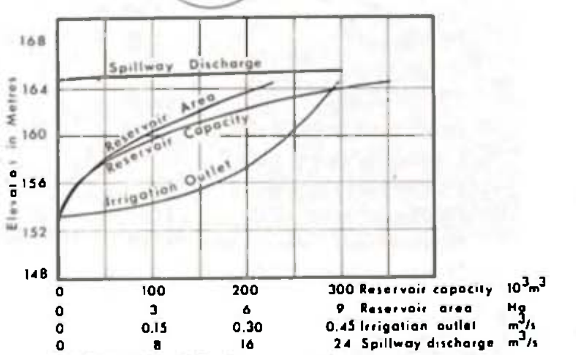
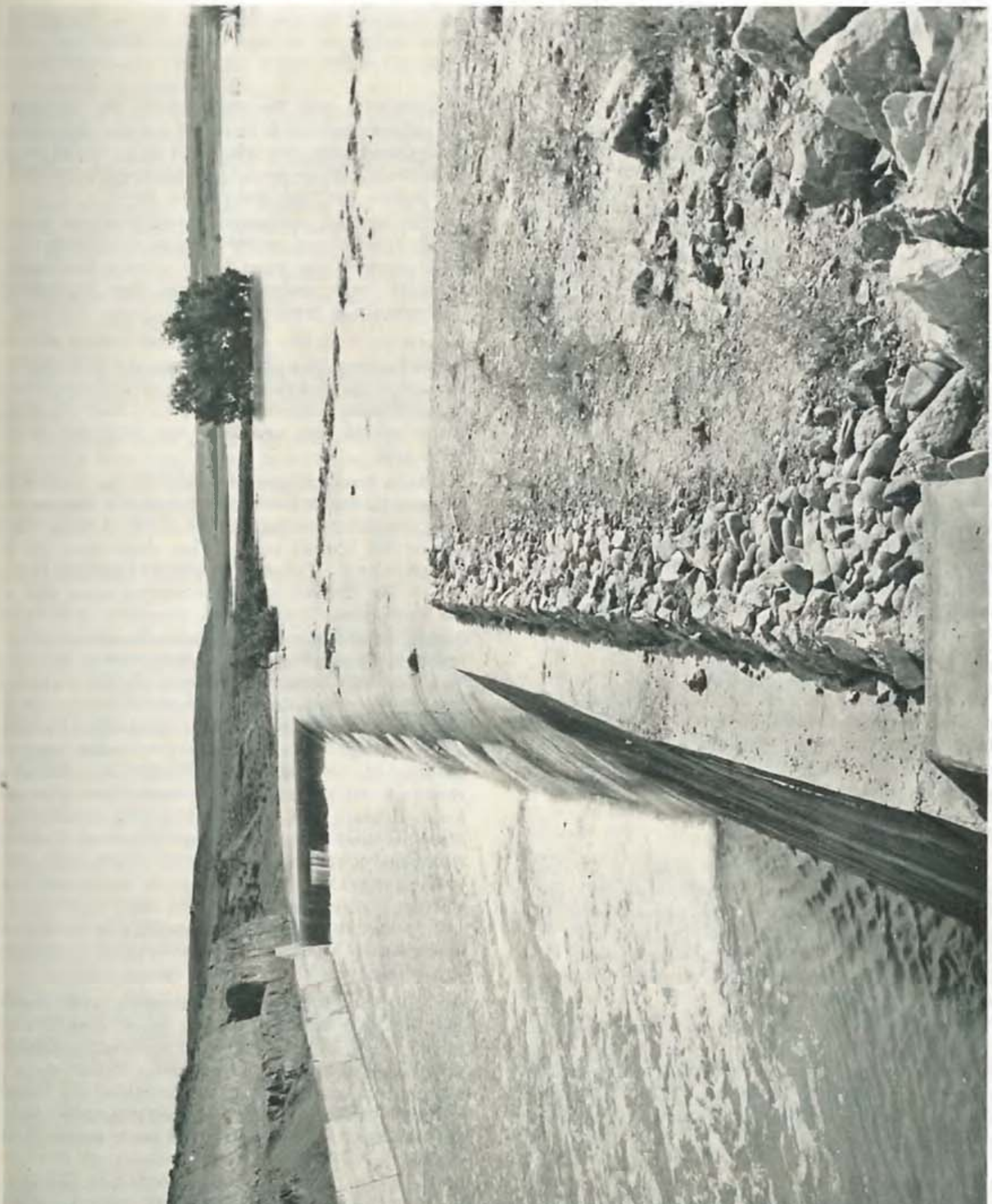


CAPACITY-AREA-DISCHARGE CURVES

Erosion of spillway unlined channel.

Data

i. Catchment	
Area	6.75 km²
Average rainfall	370 mm/a
Average runoff	0.16 million m³/a
1/1000 years flood	23.7 m³/s
Maximum height	870 m
Maximum length	7 km
ii. Reservoir	
Area	6.8 ha
Capacity	0.35 million m³
Live storage	0.34 million m³
Length	630 m
iii. Embankment	
Structural height	21.64 m
Height above ground level	15.24 m
Hydraulic height	13.41 m
Depth of foundation cut-off	6.40 m
Freeboard	1.83 m
Crest length	125 m
Top thickness	2 m
Base thickness	75 m
Upstream slope	1:2.25, 1:2.5
Downstream slope	1:2
Upstream core slope	1:1
Downstream core slope	1:0.5
Minimum core thickness	3 m
Maximum core thickness	24 m
Random fill	25,400 m³
Clay core	24,900 m³
Gravel, sand filter	1,680 m³
Rock rip-rap	490 m³
Total earth fill	52,470 m³
iv. Excavations	
Total	15,700 m³
v. Spillway	
Size	12.6x1.8 m
Capacity	23.7 m³/s
Length	42 m
Concrete	240 m³
vi. Outlet (Steel pipes)	
Size	0.30 m dia
Capacity	0.45 m³/s
Length	250 m
vii. Distribution System (Earth channels)	
(i) Main Conveyor	
Size	1.2x0.6 m
Capacity	0.45 m³/s
Length	735 m
(ii) Distribution Network	
Size	0.9x0.6 m
Capacity	0.11 m³/s
Length	7,830 m



The Ovgos dam.

SERAKHIS — OVGOS PROJECT

1. PURPOSE

The main justification for building the dam was the deterioration of the quality of the groundwater in the Ovgos aquifer which was due to continuous contamination by the Ovgos river water which at certain intervals contained water of high salt, hardness and boron concentrations. Through measurments of the river flow and quality observations, it was concluded that in most cases flood waters would not contain high percentage of salts, carbonates and boron whereas the small flows especially below 0.1 m^3/s would almost certainly contain unacceptable quantities of such chemicals. There are samples taken from the area before the construction of the dam which show NaCl of up to 2000 ppm, total hardness up to 1300 ppm, and boron up to 1 ppm. Therefore, it became very important to provide to the area, most of which is planted in citrus which have a very low tolerance of salts and boron, additional quantities of water which would be of a better quality. Furthermore, overpumping in the area which was the result of the expansion of citrus plantations necessitated the provision of additional water supplies.

2. LOCATION

This dam was built on the Ovgos tributary of the Serakhis river at about a distance of 9 km upstream from the sea and at an elevation of about 60 m asl.

3. PLAN

a. Water and Land

On this catchment the average rainfall is 380 mm/a which could result to an average runoff of about 2 million m^3/a. Automatic flow measurements have been continuously recorded since the year 1954 at a station about 2.5 km downstream of the damsite. The maximum flood estimated at the damsite at a frequency of 1/1000 years is 480 m^3/s.

There were no water use rights from this river in the vicinity of the damsite, except from a spring belonging to an Irrigation Association which had its sources at the damsite which ceased to flow for many years due to the lowering of the water table through overpumping.

b. Geology

The whole of the reservoir area lies on Recent to Pleistocene river deposits that have filled an old river valley. To the North of the reservoir area there appears an outcrop of the Athalassa Formation bioclastic limestone which was the Northern bank of the old valley.

The thickness of the river deposits was tested by two boreholes near the river course, the one where the embankment was to be placed and the other within the reservoir area, and was found to be 19 and 20 m respectively. These deposits thicken towards the South and become less thick towards the North.

The deposits are loosely consolidated and consist mainly of sands and gravels with silt especially to their upper horizons and clean to their lower Marl bands. These are very poorly sorted and they are covered with a sandy or brownish soil.

Almost the whole catchment area consists of sedimentary rocks. The older rocks are met to the Northern part of the area on the Kyrenia range. These belong to the Hilarion Limestone Formation. They are resistant to erosion and contribute only about 4% to the whole area.

Quite resistant to erosion also are the outcrops of Lapithos formation consisting of chalk and chert series. They are met as a strip to the North of Ovgos river and on the Kyrenia range. They cover about 5% of the whole area. The most significant formation in this area is the Kythrea formation covering about 45% of the whole area. The Kythrea formation is a flysch deposit consisting mainly of a marl-sandstone series. Owing to great pressure from the North it has been highly folded. Generally the beds of this formation have an EW strike and they are highly inclined, sometimes vertical. So the streams coming down from the Kyrenia range to the South meet successive layers of soft marl and hard sandstone. The marl has a tendency to erosion while sandstone appears more resistant and so many small hills are formed giving a hummocky type landscape. These beds, especially near Skylloura village have a great percentage of NaCl which with $CaSO_4$ appear as white strips as it covers sandstone outcrops where it was left after the evaporation of saline water rising because of capillary forces through the permeable sandstone beds. Other Miocene outcrops have a small extent and they are insignificant.

The Pliocene is here represented with Nicosia and Athalassa formations. They cover about 20% of the area consisting essentially of marls with occasional pebble beds and bioclastic limestone. Except where a hard surface cover of secondary limestone was formed there is a tendency to erosion. The fanglomerate series of Pleistocene age are met to the South covering 10% of the area.

Pleistocene to recent deposits are to be found in the river valley. These are of fluvial origin consisting of poorly sorted marls, sands and gravels and without any serious bedding. In some places the existence of two river terraces is apparent, the new one having also more calcareous pebbles than the old one.

The saline beds of Kythrea formation bring their soluble minerals among which the NaCl they contain.

These beds consist of a series of alternating beds of sandstones and marls of low and almost zero permeability. The rain water enters and soaks these beds diluting some of the NaCl they contain. Some of the water may reach isolated aquifers of saline water within these beds. The other is held within the pores of these beds as capillary water.

After the rain, evaporation begins during which the capillary water moves to the surface enforced by the capillary forces, where it evaporates leaving the soluble minerals it contains on the surface.

Rain washes away these minerals into the river valley. A high percentage of NaCl in the river water when the runoff is low and a low percentage when the runoff is high, is usually experienced. Also a high percentage of NaCl is expected in the water flowing early in the rainy season whilst lower concentrations are expected in the later seasons.

This was verified from samples taken from Ovgos river at a point about 2.5 km dawnstream from the damsite.

The lowest figure for salinity was observed for a flow of 1.6 million m^3/day on 20th January 1962 with an NaCl content of 41 ppm. The highest percentage was observed for a flow of 0.025 million m^3/day on the 15th December 1961 with 1568 ppm NaCl content.

4. MAIN FEATURES

a. Dam

The design of the dam and distribution system was done by the WDD. The dam is composed of a clay core wall with gravel and sand fill on either side. As the sand and gravel alluvium at the river bed is deep reaching up to 21 m below river bed, it was decided to have a clayey upstream blanket instead of a cut-off wall. This blanket which is 0.9 m thick extends for 42 m upstream of the clay core to which it is connected. It is protected by a gravel blanket 0.6 m thick covering the whole of the clay blanket. A handplaced rock rip-rap has been placed on the upstream slope. A concrete lined side spillway has been provided with three stilling basins up to river bed and lined in concrete throughout its length. A 1.3 m dia concrete lined gallery passes through the embankment from its upstream to its donwstream side. Within this culvert a 0.6 m dia steel irrigation outlet pipe has been placed. This irrigation pipeline which is connected to the irrigation canal just downstream of the dam, is operated through a hand operated valve on the downstream side.

An interesting feature of the project is the saline water diversion works A diversion weir just upstream of the reservoir was built which diverts water up to 0.03 m^3/s through the dam gallery to the downstream side. As this proved insufficient for the diversion of the saline water, an additional pipeline capable of diverting 0.2 m^3/s was added in 1968.

b. Distribution System

This is made up of a main trapezoidal concrete lined canal conveying the water of the dam to the main project area and a network of distribution system which has been concrete lined and conveys the water of the many boreholes in the region in addition to that of the dam.

5. CONSTRUCTION

The dam and distribution system were constructed by the WDD between June 1963 and October 1964. The work had no particular difficulties, and all materials except concrete aggregates and the rock rip-rap were obtained from local borrow areas. The clay core was compacted by sheepfoot rollers in 0.15 m layers to a dry density of 1,680 kg/m^3 and an optimum moisture content of 17.2%. The compaction of the sand and gravel fill zones was done by means of heavy wheel type pneumatic rollers.

The total expenditure on the dam was £114,000.

For the distribution, £62,000 worth of channel lining has been completed.

6. MANAGEMENT

This project is operated by the Ovgos Irrigation Division, which undertook to pay 25% of the cost of the dam and between 33% and 50% of the cost of the distribution system having obtained a loan from the Government payable over 8 years at 5% rate of interest.

The land commanded by the whole distribution network which is fed with water from the dam as well as from the regional boreholes, is about 6,370 donums most of which is cultivated in citrus plantations.

7. PROBLEMS OF SPECIAL INTEREST

Since the completion of the dam, the most important problem was that of the contamination of the reservoir water by high concentraction of salts and boron.

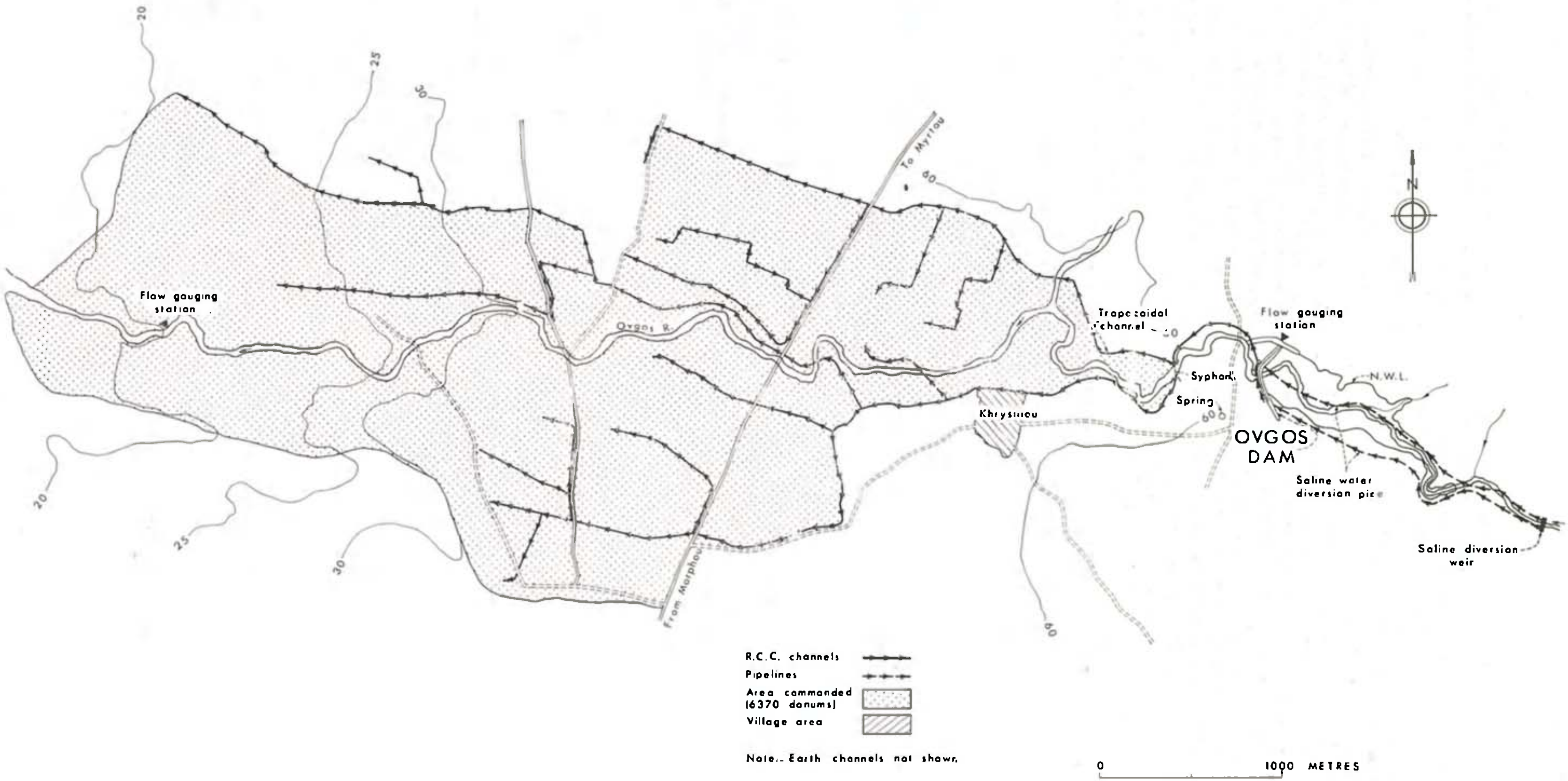
Fortunately the soils of the region are mostly sandy and therefore it is possible to utilize water with relatively high salt contents provided that the water applied for irrigation is in excess of the normal requirements. Water with 500 ppm NaCl, is undesirable whilst in excess of 800 ppm should not be used. The boron content exceeding 0.3 ppm is harmful for citrus.

During the first year of storage until 1965 the salinity of the water in the reservoir varied from 112 to 480 ppm NaCl. The boron varied from 0.16 to 1.2 ppm However, between 1967 and 1968 excessive rainfall on the Skylloura part of the catchment which is made up of the Kythrea formation and contributes largely to the contamination of the water resulted to the deterioration of the water in the reservoir. During these years it became necessary to discharge some of the water to the sea as the salinity reached up to 3,500 ppm NaCl.

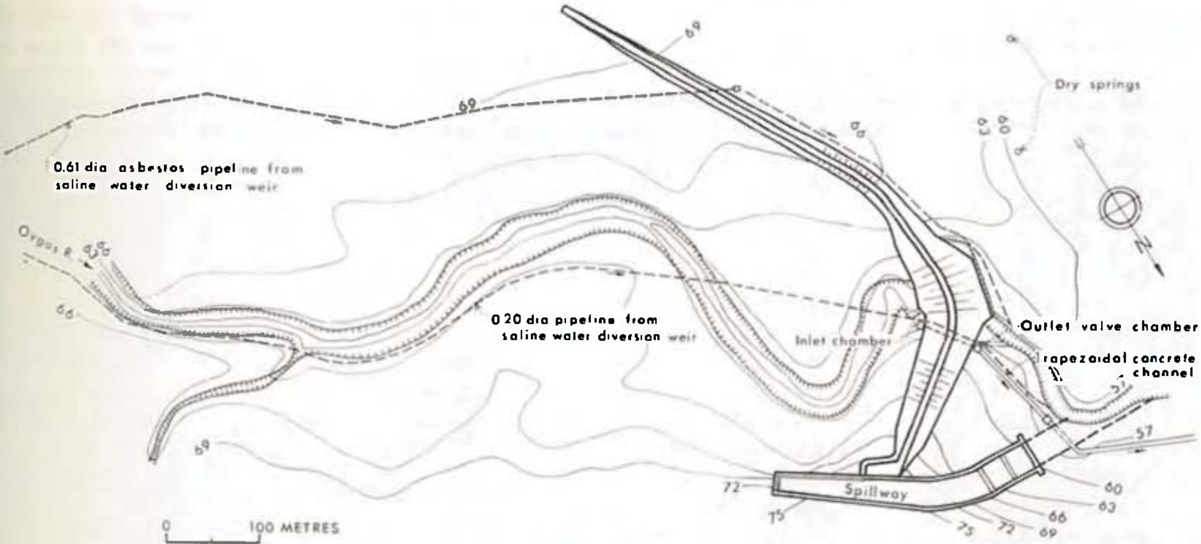
From 1964 to 1969 out of a total of 4 million m^3 stored, 2.4 million m^3 were of good quality having less than 500 ppm NaCl, whilst out of the remaining 1.6 million m^3 with NaCl above 500 ppm, only 0.4 million m^3 were discharged to the sea, whilst 1.2 million m^3 were used in excessive water applications to allow for leaching.

The deterioration of 1967—1968 justified the con-

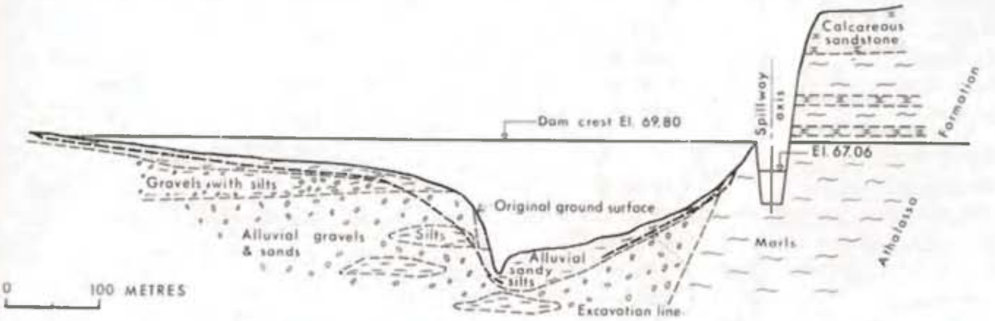
SERAKHIS-OVGOS-DISTRIBUTION SYSTEM



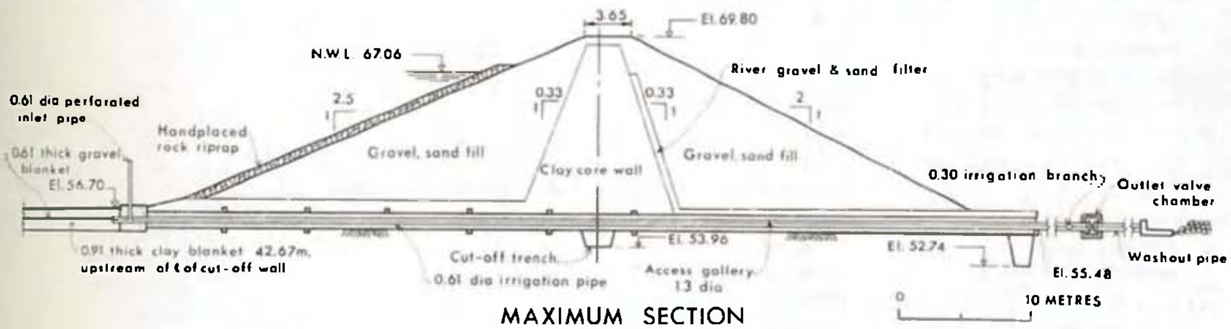
SERAKHIS-OVGOS DAM



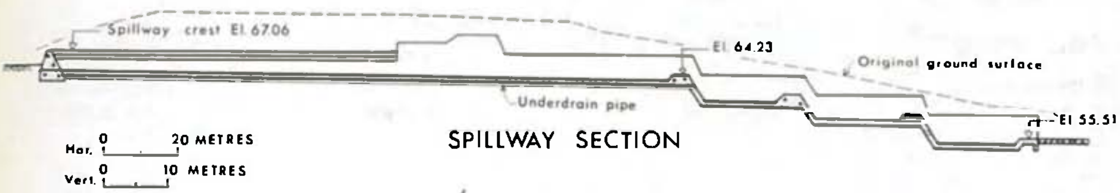
PLAN



GEOLOGICAL SECTION ON DAM AXIS

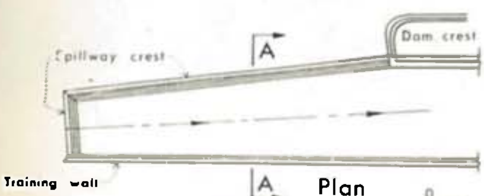


MAXIMUM SECTION



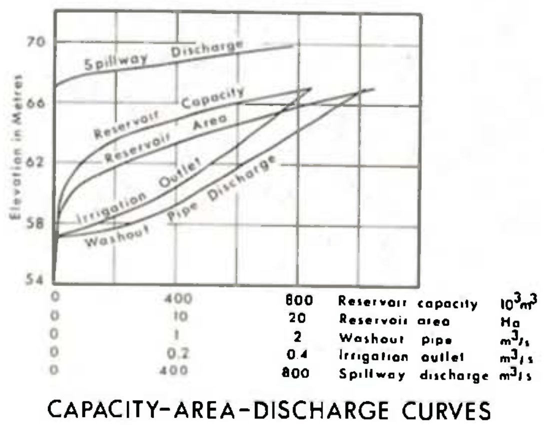
SPILLWAY SECTION

Section A-A



Plan

SPILLWAY



CAPACITY-AREA-DISCHARGE CURVES

struction of the second saline diversion scheme with a 0.6 m dia pipeline capable of diverting up to 0.2 m^3/s.

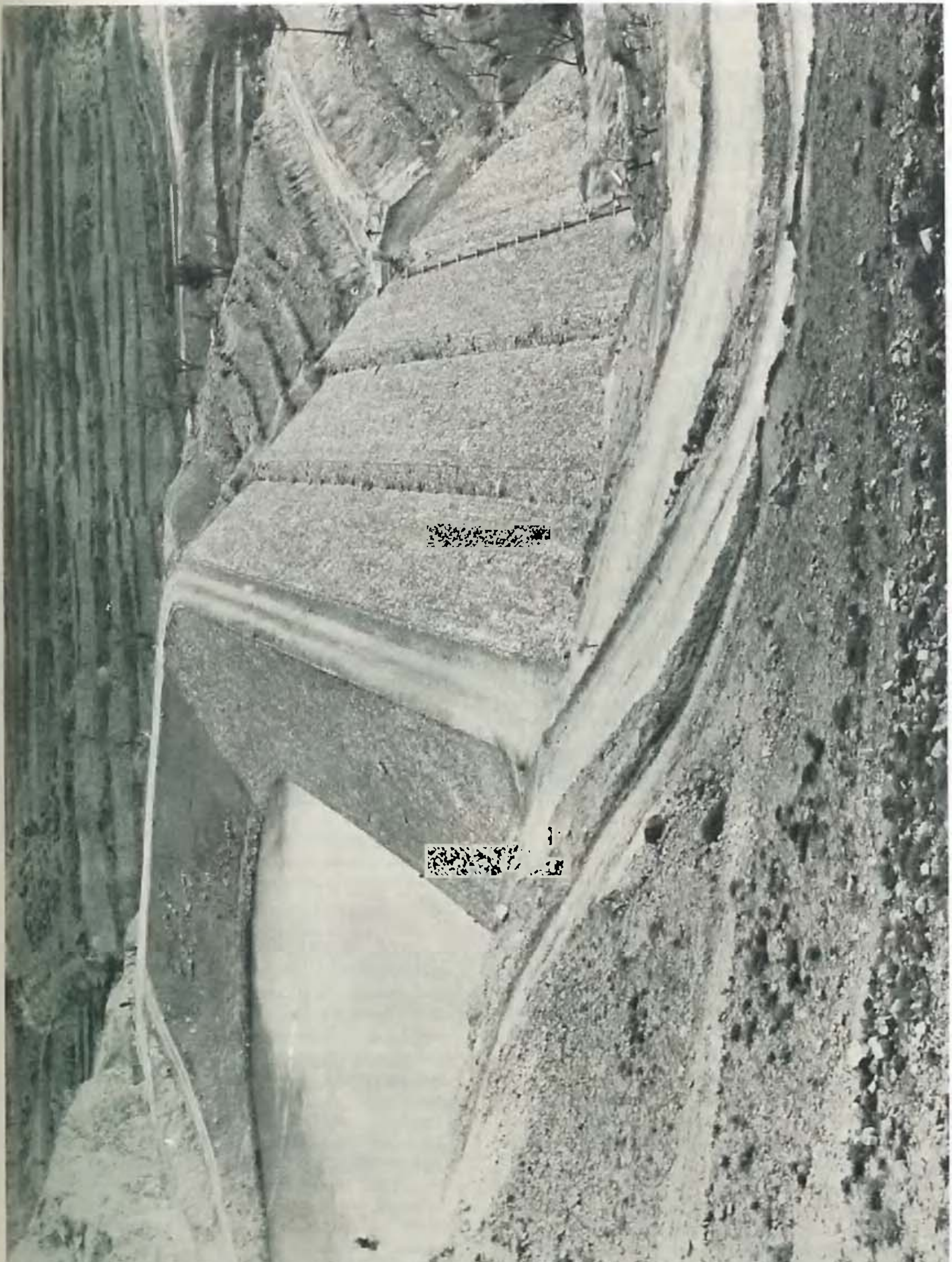
To improve the situation further it is being considered to convey additional good quality water from the Serakhis river valley to the Ovgos reservoir for mixing purposes.

Another interesting development was that during the first impoundment in 1964, the dry Ovgos Spring just downstream of the damsite started flowing again, the flow increase being relative to the reservoir level. The owners of this spring raised claims on this water.

Data

i.	*Catchment*	
	Area	186 m^2
	Average rainfall	380 mm/a
	Average runoff	2 million m^3/a
	1/1000 years flood	480 m^3/s
	Maximum height	660 m
	Maximum length	30 km
ii.	*Reservoir*	
	Area	26 ha
	Capacity	0.83 m^3
	Live storage	0.58 m^3
	Length	150 m
iii.	*Embankment*	
	Structural height	17.06 m
	Height above ground level	13.10 m
	Hydraulic height	10.36 m
	Depth of foundation cut-off	3.96 m
	Freeboard	2.74 m
	Crest length	720 m
	Top thickness	3.65 m
	Base thickness	65 m
	Upstream slope	1:2.5
	Downstream slope	1:2
	Upstream core slope	1:0.33
	Downstream core slope	1:0.33
	Minimum core thickness	2.4 m
	Maximum core thickness	31.5 m
	Random fill	75.000 m^3
	Clay core	49,000 m^3
	Gravel blanket	3,800 m^3
	Sand filter	7,500 m^3
	Clay blanket	7,500 m^3
	Rock rip-rap	3,800 m^3
	Total earth-fill	146,600 m^3
iv.	*Excavations*	
	Total	16,500 m^3
v.	*Spillway*	
	Size	72x2.7 m
	Capacity	780 m^3/s
	Length	264 m
	Concrete	5750 m^3
vi.	*Gallery*	
	Size	1.3 m dia
	Capacity	For access
	Length	54 m
	Concrete	38 m^3
vii.	*Outlet (Steel pipes)*	
	Size	0.6 m dia
	Capacity	0.44 m^3/s
	Length	66 m
viii.	*Distribution System*	
(i)	*Main conveyor (Concrete channels)*	
	Size Trapezoidal	(1.3+0.3)x0.4 m
	Rectangular	07.x0.4 m
	Capacity	0.17 m^3/s
	Length	3000 m
(ii)	*Distribution network (Concrete channels)*	
	Size	0.45x0.22, 0.6x0.3 m
	Capacity	0.06—0,17 m^3/s
	Length	25,500 m
ix.	*Saline Diversion*	
(i)	*1st Phase*	
	Size	0.2 m dia
	Capacity	0.03 m^3/s
	Length	1,860 m
(ii)	*2nd Phase*	
	Size	0.6 m dia
	Capacity	0.2 m^3/s
	Length	1,650 m

AGROS



The Agros dam.

KOURIS—AGROS PROJECT

1. PURPOSE

Agros village is a high altitude village on the Troodos mountains in the Pitsilia region. The terrain is very steep and the river Kouris has part of its catchment here. To impound and utilize significant quantities of water in this region is practically impossible and even the use of limited quantities is very costly. The main income of this region is from rainfed vines which are used for wine production and from other rainfed crops such as almond trees. Because of the limited land development possibilities and the high rainfall during winter and spring, even the supply of relatively small quantities of summer water would contribute largely to the successful cultivation of more remunerative crops such as apple, pear, peach and plum plantations. The building of the dam which stores the little flow of the small catchment of the stream in which it is built and in addition the domestic water supply surplus, the surplus of some nearby springs and of what has been possible to divert from nearby small catchments, has enabled the cultivation of about 300 donums of mountainous land converted into cultivable after extensive land levelling and the cultivation of apple, pear, peach and plum tree plantations.

2. LOCATION

This dam is located on a small tributary of the Kouris river West of the village of Agros at an altitude of 1,120 m asl.

3. PLAN

a. Water and Land

The average rainfall on the catchment is 780 mm/a giving an average annual runoff of about 120,000 m^3. The maximum flood estimated at the damsite at a frequency of 1/1000 years is about 6 m^3/s. No water use rights were exercised on this stream previously. Another source of water filling the dam is the surplus of the domestic water supply estimated at 40,000 m^3 annually and the surplus of other nearby springs estimated at another 20,000 m^3/a.

b. Geology

The bedrock is made up of gabbro with extensive fracturing. In many places the rock lies under a shallow terrace mantle composed mainly of weathered bedrock with remnant structure. Because of the deeply weathered and fissured rock, leakage was anticipated from the reservoir and foundations. The silting up of the reservoir was another serious problem due to the high erosion of the catchment being covered with little vegetation mainly vineyards and which could receive heavy rainfall. The dam alignment makes a sharp break between the main embankment and the left abutment. This allows full utilization of a natural low ridge and hilltop as a portion of the dam and contributes in increasing the very limited reservoir capacity. In addition the side embankment can provide a built-in safety fetaure to act as a relief dyke in case of excessive floods above the flood provided on the design.

4. MAIN FEATURES

a. Dam

This dam was designed by the WDD. It is an earth embankment with a clay core in the middle, upstream zones of gravel, sand and silty material and downstream zones of rock fill. Dumped rock rip-rap has been placed on the upstream side and handplaced on the downstream. A 0.25 m dia outlet pipe has been fixed through the embankment for the supply of water to irrigation. From the downstream side up to about 25 m upstream a small infiltration gallery has been built to collect the water from a spring covered by the embankment fill. A tunnel 1.30 m dia was also provided from downstream up to about half way upstream for housing the outlet pipe. The upstream part of the pipe has been encased in concrete fitted with concrete collars at intervals.

A check dam of gabbion walls to act as a silt trap was provided just upstream of the reservoir to reduce the sediment entry in the reservoir. This silt trap is easily accessible for desilting.

b. Distribution System

The distribution system designed by the WDD is made up of a contour channel peripheral to the land commanded with a network of steel pipe distribution to each plot of land delivering water by gravity.

5. CONSTRUCTION

The construction was done by the WDD between April 1964 and the end of December 1964.

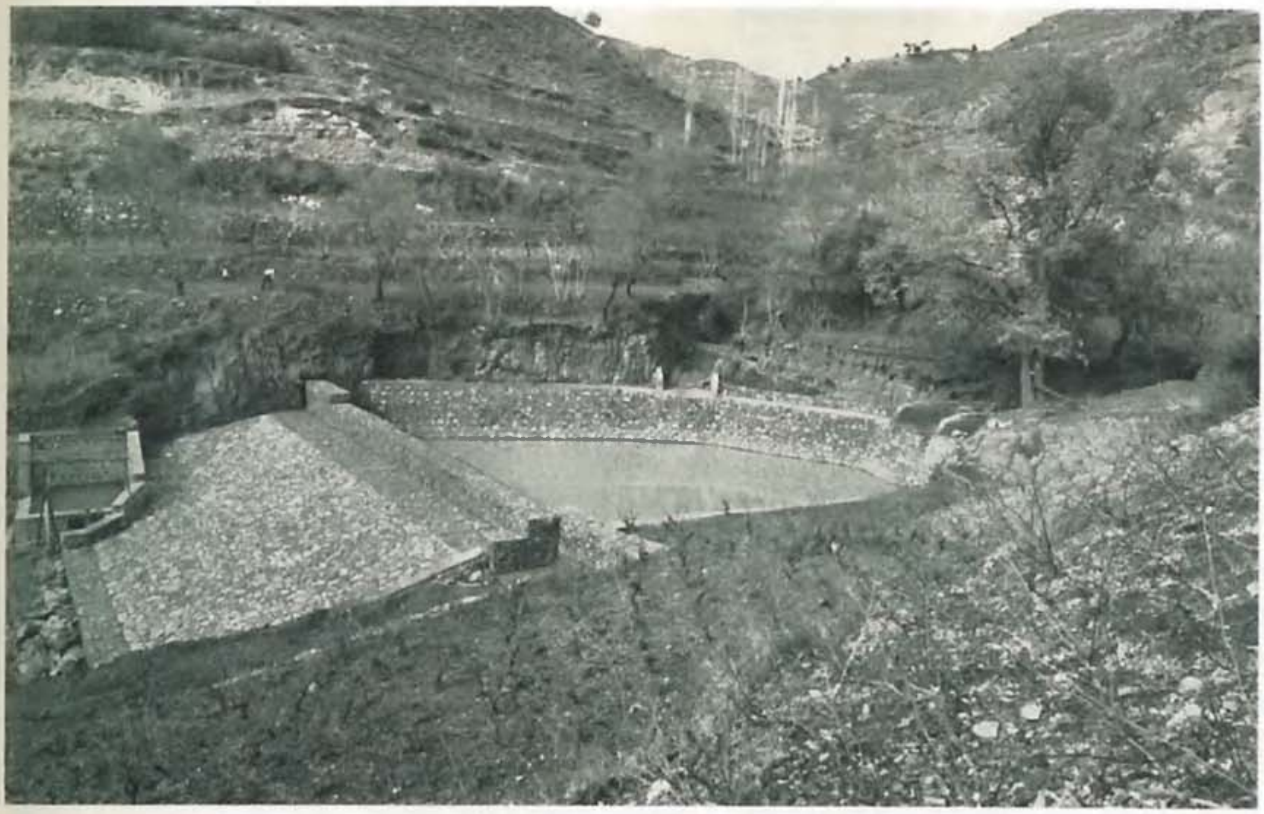
During the excavation of the cut-off it was found that there existed extensive cracks and joints on the rock and a decision was taken to grout the cut-off trench. Grout holes were drilled along the cut-off at 2.25 m spacing up to half the height of the dam on the abutments. A grout concrete cap was cast and the grout mix used was 1:3 cement and water reduced to 1:2 when the pressure took time to build up. Each hole was pressure grouted in 3 stages, the first 4.8 m, the second up to 9.6 m and the third up to 12.6 m The pressure was applied up to 4.30 kg/cm^2 for the first stage and up to 7 kg/cm^2 for the second and third stages. Compressed air up to 7 kg/cm^2 for 10 minutes was applied as a test to the grouting. The grouting operations lasted 8 weeks. In all, 23 holes were drilled of a total depth of 300 m and the total grout consumption was 15 tons of cement. The total cost of this grouting work was £1,750. The consumption in cement varied between 17 and 115 kg/m.

The compaction of the clay core was done in 0.15 m layers to dry density of 2,450 kg/m² and an optimum moisture content of 21.5%. The number of passes with the sheepfoot rollers were 24. Close control was kept both by site and laboratory tests. The total cost of the dam was £45,000.

The land which has come under irrigation is 300 donums of a mountainous area just above the village which has been suitably land levelled. The main crops that have been planted are apples, pears, peaches and plums.

7. PROBLEMS OF SPECIAL INTEREST

With the first impoundment which occurred in January 1965 it became evident that a lot of water was being lost through the jointed and fractured gabbro



Silt trap upstream of Agros dam.

rock of the reservoir sides. On the other hand seepages downstream of the cut-off and an old spring did not increase their yield which proved that the grouting carried out was effective. As a temporary measure for reducing the seepage through the reservoir sides, a solution of asbestos mill waste obtained from the Asbestos Mines at Amiandos, was poured into the reservoir using a boat. The results were that the reduction in losses was about 15%.

The following years showed that the seepage through the reservoir side continued at the same rate. It was therefore decided to apply a clay blanket in the reservoir which was found to be the cheapest solution for reducing the losses.

This work has been done in three phases. The first one was executed from November 1967 to January 1968 the second one from August to December 1968 and the third one in the Autumn of 1973. The total quantity of clay used was 16,000 m³ at a total cost of £12,500. The width of the blanket which covered the whole reservoir up to spillway reservoir level is

After filling of the reservoir, leakage was observed and a clay blanketing of the whole reservoir was constructed at a cost of £12,500.

Also subsequent to the construction of the dam and in order to have enough water to fill the reservoir in dry years, a pumping scheme has been constructed to pump water to the dam from the same stream downstream over a distance of 1,750 m and a head of 150 m. The cost of this project was £13,500.

The construction of the distribution system, the cost or which was £5,000 was also undertaken by the WDD, and work was started and completed within the dam construction period. The total cost of all works was £76,000 out of which the Irrigation Division undertook to pay £19,500 through a loan payable in 15 years and at 5.25% rate of interest.

6. MANAGEMENT

This project is administered by the Agros Irrigation Division which was formed for this purpose.

about 1 m at right angles to the slopes. The clay was placed at 0.15 m layers compacted by sheepfoot rollers. The reduction of the reservoir capacity by the application of the blanket was 15%

One other important problem which appeared with the operation of the dam is that although the clay blanked proved successful, yet for certain years of low rainfall the reservoir would not fill completely. Two alternative studies have been produced. The first one provided for 20 l/s to be pumped from a lower point in the same stream at a distance of 1,750 m and a difference of elevation of 150 m from the dam. The cost of pumping, however, would be 19 mils/m³.

The second alternative provided for the diversion by gravity of a nearby higher elevated stream at a distance of 1,750 m of a quantity of about 20 l/s during the winter season only.

The second alternative is cheaper and it avoids pumping but has met the objections of a nearby village which utilizes the water of the stream in Summer, although it was made clear to them that only Winter surplus water would be diverted to the dam.

Because of the objections, the pumping scheme has been executed in the Autumn of 1973.

Clay blanket on original rip-rap.

Data

i. Catchment

Area	0.43 km^2
Average rainfall	780 mm/a
Average runoff	120,000 m^3/a
1/1000 years flood	6 m^3/s
Maximum height	1,200 m
Maximum length	1 km

ii. Reservoir

Area	1.5 ha
Capacity	0.10 million m^3
Live storage	0.10 million m^3
Length	141 m
Clay blanket	5,250 m^3

iii. Embankment

Structural height	26.20 m
Height above ground level	16.15 m
Hydraulic height	14.62 m
Depth of foundation cut-off	10.05 m
Freeboard	1.53 m
Crest length	171 m
Top thickness	5 m
Base thickness	83 m
Upstream slope	1:2
Downstream slope	1:1.6
Upstream core slope	1:0.3
Downstream core slope	1:0.3
Minimum core thickness	2.4 m
Maximum core thickness	16 m
Random fill	13,500 m^3
Rock fill	10,500 m^3
Clay core	13,000 m^3
Gravel, sand filter	9,500 m^3
Sand filter	5,400 m^3
Rock rip rap	600 m^3
Total earth fill	52,500 m^3

iv. Clay Blankets

Thickness	1 m
Volume of fill	10,500 m^3

v. Grouting

Total drilling 23 holes	300 m
Total grouting 23 holes (cement)	15 tons

vi. Excavations

Total	20,500 m^3

vii. Spillway

Size	6x1.2 m
Capacity	6 m^3/s
Length	50.7 m
Congrete	150 m^3

viii. Gallery

Size	1.30 m dia
Length	28.5 m
Congrete	22 m^3

ix. Outlet (Steel pipes)

Size	0.25 m dia
Capacity	0.10 m^3/s
Length	65 m

x. Distribution System

(i) Main conveyor (concrete channels)

Size	0.30x0.20 m
Capacity	0.084 m^3/s
Length	1,830 m

(ii) Distribution network (steel pipes)

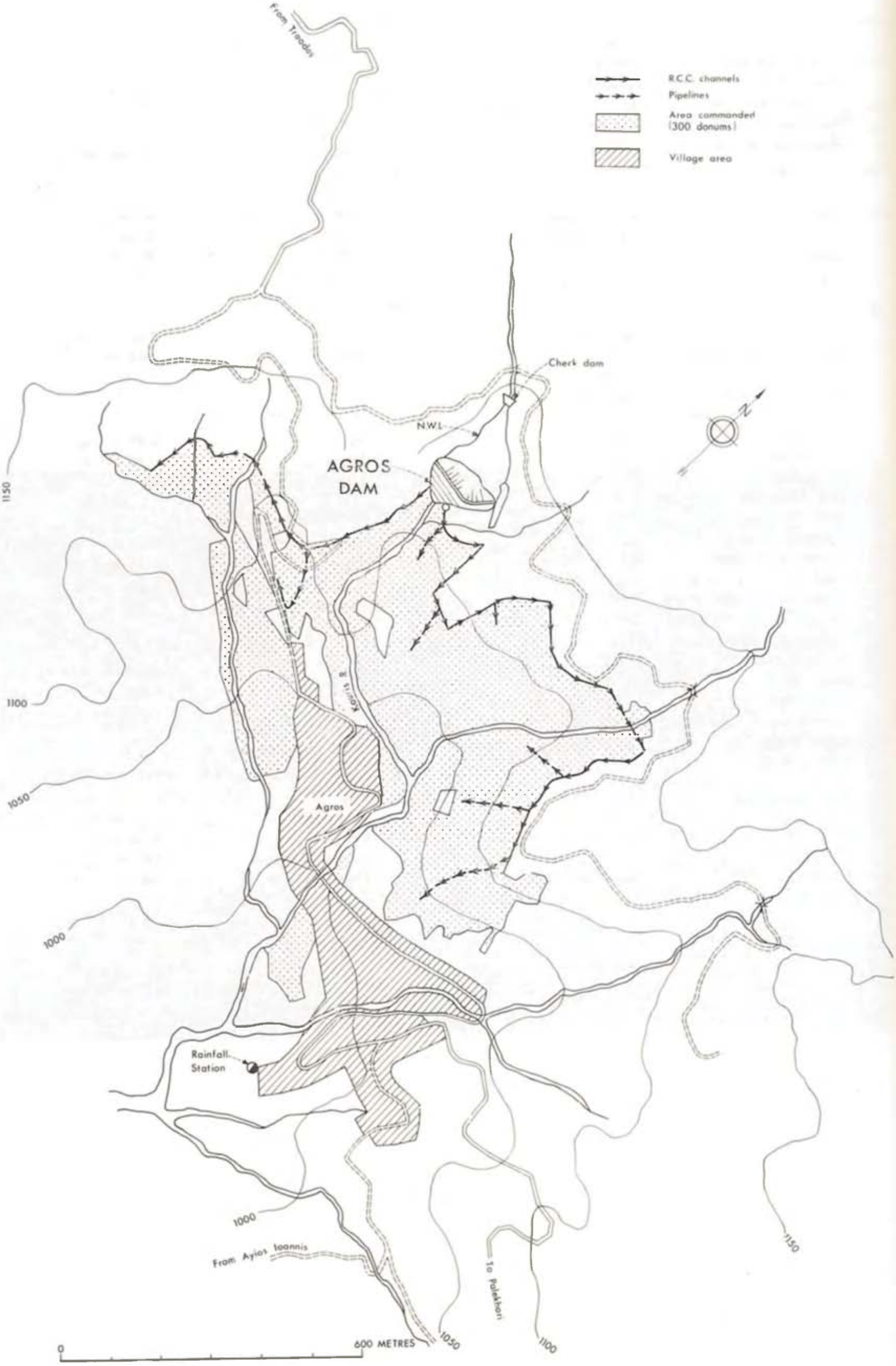
Size	0.075 to 0.10 m dia
Capacity	0.028 m^3/s
Length	1,380 m

xi. Pumping Scheme

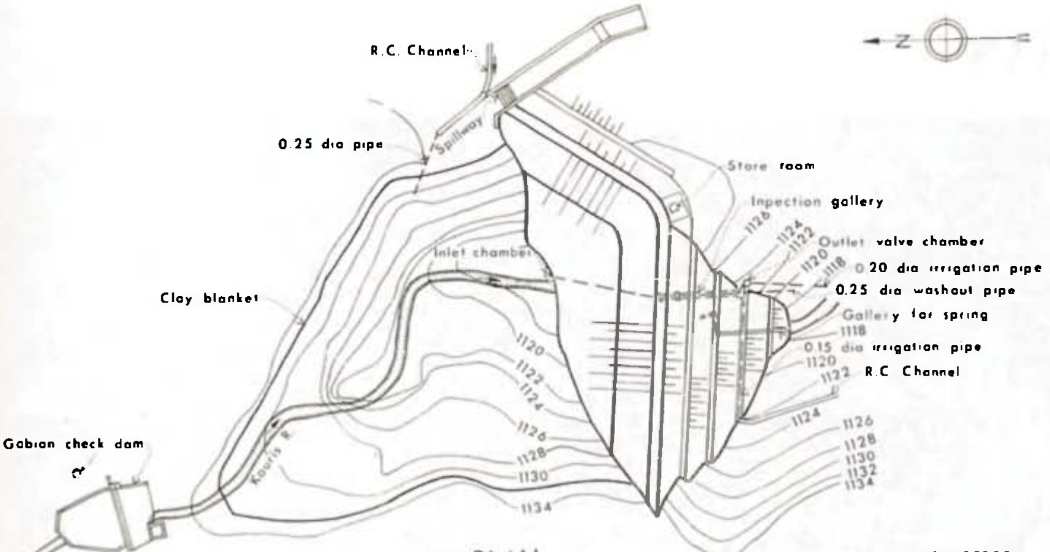
(i) Pumping main (steel pipeline)

Size	0.15 m dia
Capacity	0.02 m^3/s
Length	1,750 m
Pumping head	150 m
Pump Engine	100 hp

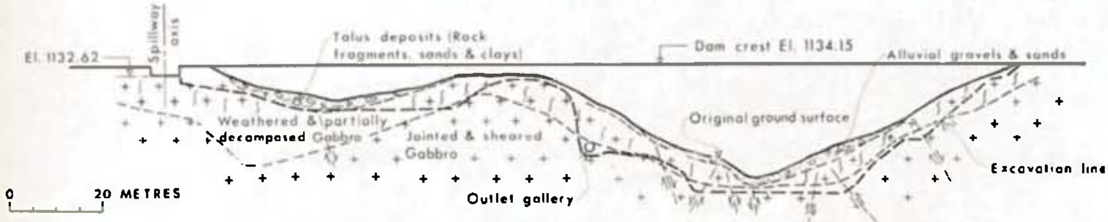
KOURIS-AGROS-DISTRIBUTION SYSTEM



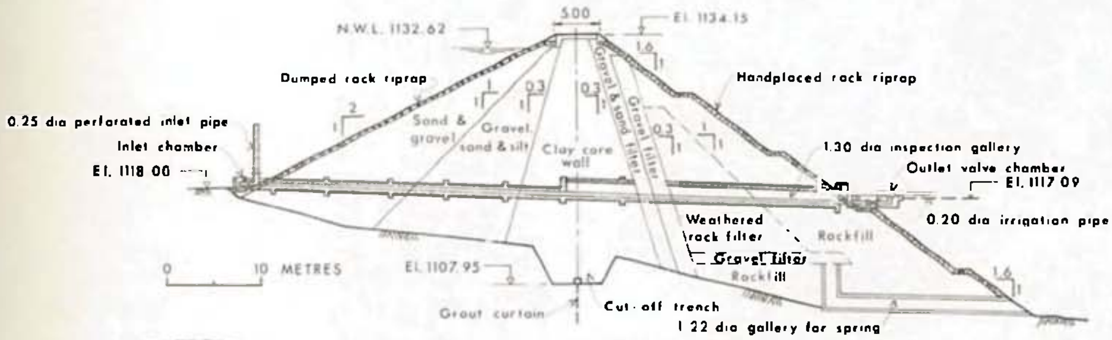
KOURIS - AGROS DAM



PLAN

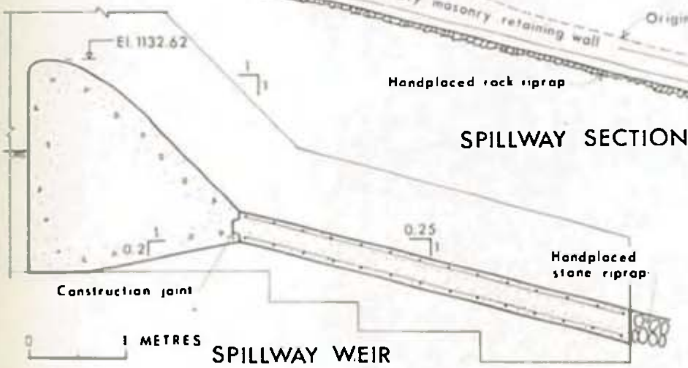


GEOLOGICAL SECTION ON DAM AXIS

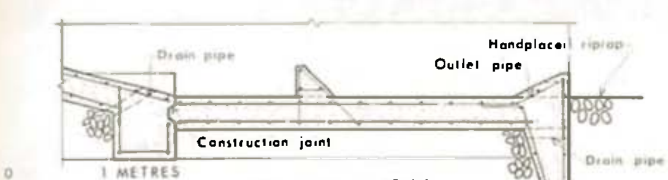


MAXIMUM SECTION

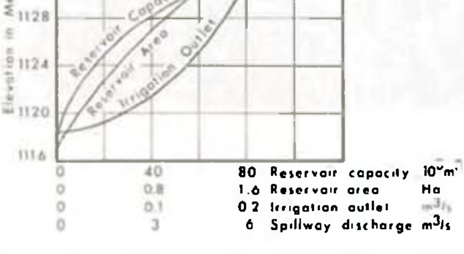
SPILLWAY SECTION



SPILLWAY WEIR

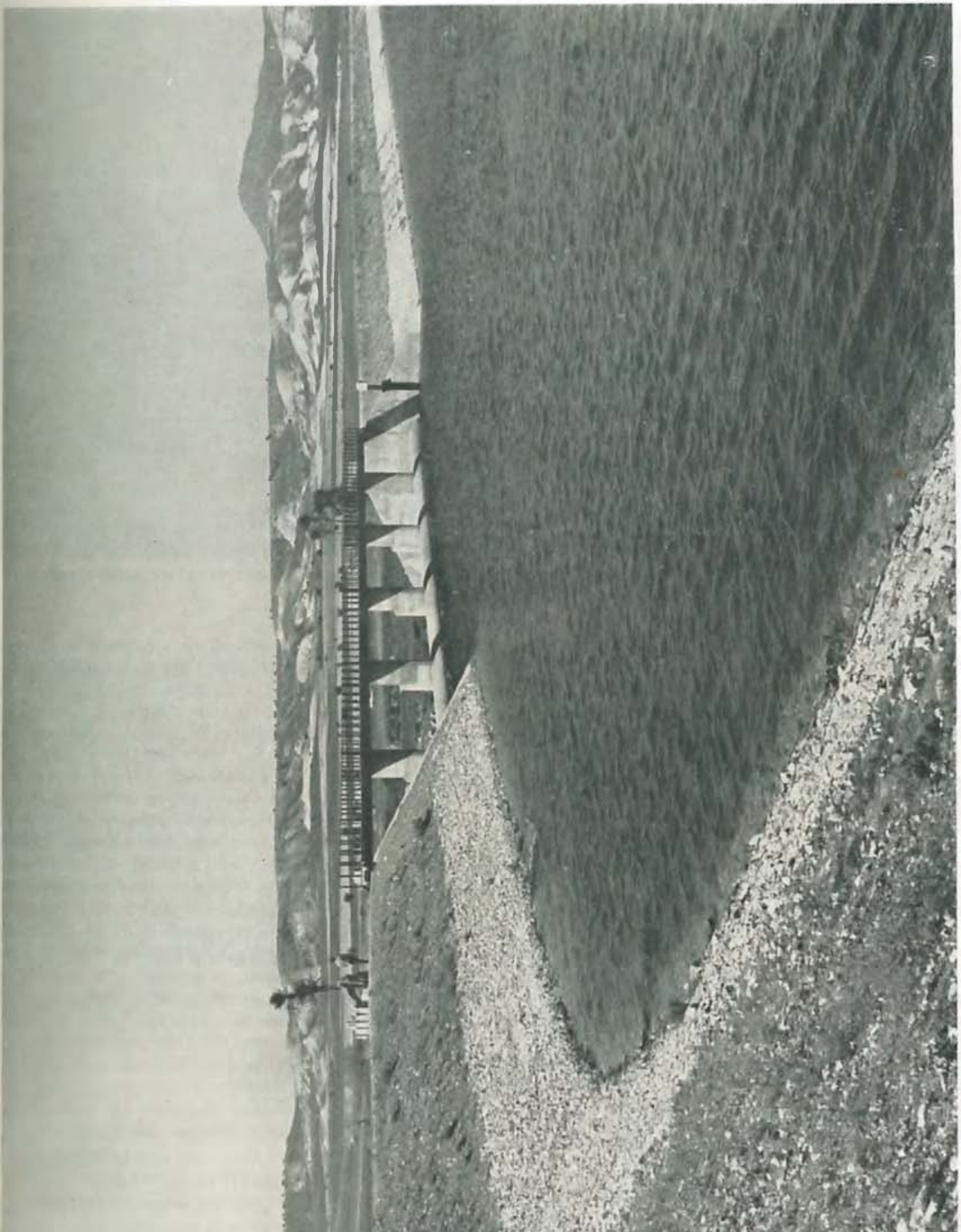


STILLING BASIN



CAPACITY-AREA-DISCHARGE CURVES

KITI



The Kiti dam.

TREMITHIOS — KITI PROJECT

1. PURPOSE

The depletion of the Pleistocene — recent alluvial aquifer between the villages of Kiti and Perivolia, was the result of over-pumpage in the region from 130 boreholes for the irrigation of 260 donums of citrus and 1,740 donums of artichokes, melons, potatoes and other vegetables. The salinity of the ground water increased considerably through sea intrusion and has polluted the aquifer up to 2.5 km inland in the region of Perivolia. At the same time the deficiency of the domestic water supply sources of Larnaca town from the Bekir Pasha chain-of-wells and from three boreholes just upstream of the dam as well as the deficiency of the domestic water supply of five nearby villages had necessitated the provision of additional water supplies.

The dam has been planned to serve both for direct irrigation with water conveyed from the dam to the irrigated areas with a consequent reduction of pumping from the aquifer and at the same time to benefit the groundwater table directly through lateral infiltration from the permeable sand and gravel abutments or through infiltration from the river bed which could be facilitated by specially constructed spreading grounds downstream.

2. LOCATION

The dam is located on the main Tremithios river, 3 km upstream of Kiti village in the district of Larnaca. It is situated 6.5 km upstream of the sea at an elevation of about 30 m asl.

3. PLAN

a. Water and Land

The surface flow was evaluated from automatic water level recorder measurements taken upstream and downstream of the dam site for the last 10 years, but as this period of surface measurements is not enough calculations for the runoff were mainly based on precipitation data in the catchment for which 55 years records are available. The average annual flow at the dam site has been calculated to be about 6 million m^3. The maximum flood for the spillway design at 1/1000 years frequency has been estimated to be of the order of 610 m^3/s.

The water use rights existing downstream of the dam site were evaluated according to existing legislation, but the owners abandoned their water rights in order to facilitate the construction of the dam.

The land commanded by the dam was selected to be the land belonging to the Irrigation Divisions of the area and which had the water use rights on the river. These Irrigation Divisions are the Kokkines — Ammi, the Stephanaki, the Megalon and the Bey. Most of this land are soils which are suitable for growing of all types of crops under the local climatic conditions. Provision was also made for the supply of water to Tersephanou and Meneou villages, both of which had intakes on the river. At the same time lands belonging to Perivolia village were incorporated in the project in order to reduce pumping and relieve the aquifer from further sea intrusion. The lands commanded for irrigation are 1,260 donums for Kiti, 630 donums for Perivolia and 210 donums for Tersephanou.

In general, most of the water of the dam is utilized between April and July, during which period the pumping of water from the depleted aquifer is reduced to the minimum. Thereafter, pumping is carried out mainly for citrus trees. Summer and Autumn non-permanent crops are discouraged. One important reason for the utilization of the water through a short period in Spring and early Summer is the considerable seepage from the permeable formations and the high evaporation losses from this rather shallow reservoir.

b. Geology

The general geology of the catchment is characterized by the Igneous volcanic rocks on the upper reaches, the Lapithos chalks (Cretaceous) in the middle, the Pakhna marls and sandstones (Miocene) a little upstream of the dam, whilst the damsite, reservoir and downstream area lie in Pleistocene-Recent alluvial deposits. A hidden old river valley is found on the West abutment of the dam which is overlaid by more recent deposits. Secondary limestones overlying the marls are to be found higher up the dam site and reservoir. The bed rock at the dam site is made up of Miocene marls which are found at depths of 6 m from river bed level. The permeable terrace material and the secondary limestone present leakage problems. However, as the downstream alluvial aquifer is in direct contact with the reservoir area, seepage is bound to replenish the aquifer, the depletion of which constitutes one of the main reason for building the dam.

4. MAIN FEATURES

a. Dam

The detailed design work for the dam was done by Consultants Il Nuovo Castoro of Florence, Italy, whilst that of the distribution system by the WDD.

The dam is earth-fill with a central clay core wall, on either side of which sand gravel filters were placed. Hand-placed limestone rip-rap protects the upstream shell. The spillway is made of concrete and is of uncontrolled ogee type intake structure connected to a trapezoidal concrete chute discharging on a stilling basin at river bed level.

Rectangular R.C. irrigation channel.

A positive cut-off to control excessive seepage was built to the depth of the marl which was found at a shallow depth in the river bed. The cut-off on either abutment did not reach the marl level.

Due to the existence of erosive sedimentary rocks in the catchment which are sparsely covered with vegetation, the silt problem was seriously considered. A relatively large concrete gallery was provided under the dam for this purpose which can discharge up to 11 m^3/s under free flow conditions. This gallery, was utilized during construction for the diversion of the river flow and is now used to discharge for irrigation the first Autumn floods which contain a high percentage of silt.

It is operated by means of a hydraulic cast iron slide gate. Three irrigation steel outlets are provided, two low level and one high level. The control is by means of hand operated downstream sluice valves. The two smaller intakes supply water rights whilst the larger one supplies the main asbestos-cement pressure pipeline conveyor, which in turn supplies the distribution system network.

b. Distribution System

The irrigation system has been designed by the WDD and commands the whole area to Perivolia. It is made up of prefabricated concrete canals and cast-in-situ concrete channels. Also some length of earth channels are still in use.

5. CONSTRUCTION

The construction was executed by the WDD between May 1963, and January 1965. No difficulties were met during construction. All materials utilized, except for the concrete aggregate were obtained from the axcavations in the reservoir. The concrete aggregate was obtained from the seashore. The compaction of the earth fill was done in 0.15—0.22 m layers by means of heavy pneumatic rubber type wheel rollers with 6-8 passes. The control of compaction was done by special laboratory equipment at the field. The total expenditure on the dam was £140,000.

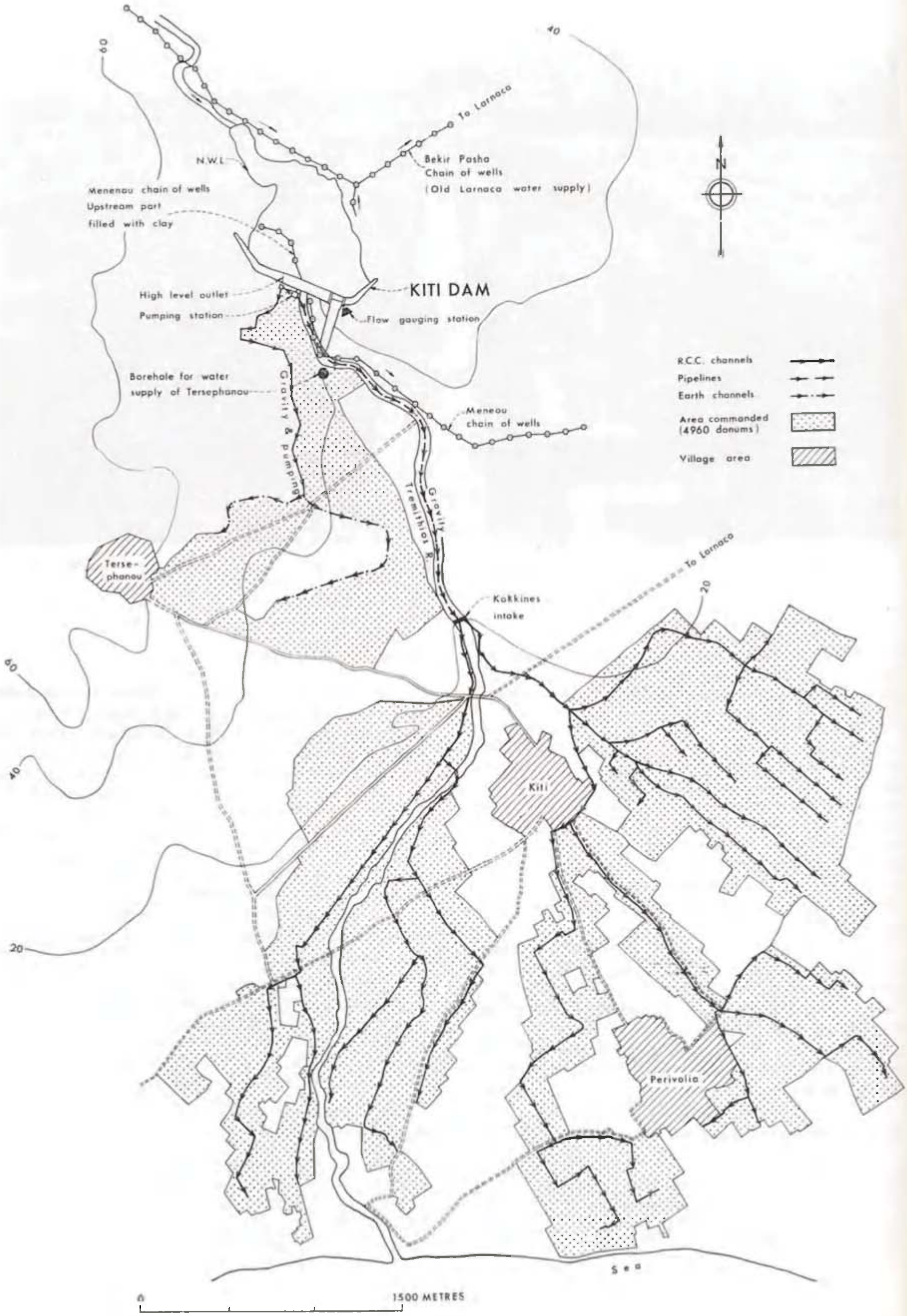
The Distribution System was carried out in various stages. The first stage included the main conveyor and cast-in-situ R.C. channels.

The second stage was of prefabricated concrete canal construction of parabolic shape. The total expenditure incurred until 1973 on the distribution system was £127,000. However, 30% of the land is not yet covered by lined channels and more extensions of the concrete lined system have to be made in the coming years.

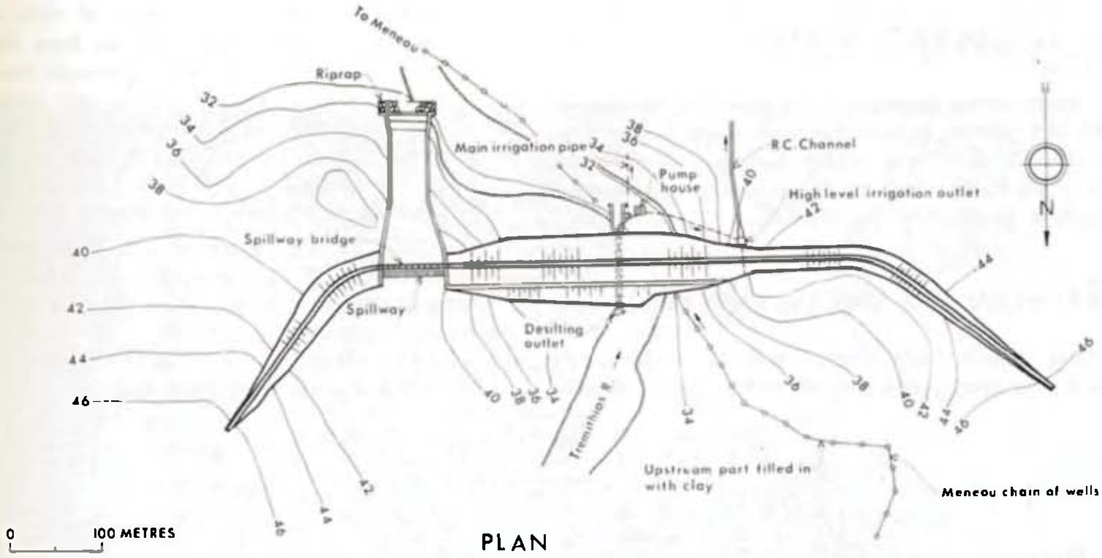
6. MANAGEMENT

The dam has been constructed at full cost to the Government and is administered by a Government Project Committee. Water is sold to the members of the Irrigation Divisions previously mentioned at 10 mils/m^3 which represents about 50% of the annual costs, inclu-

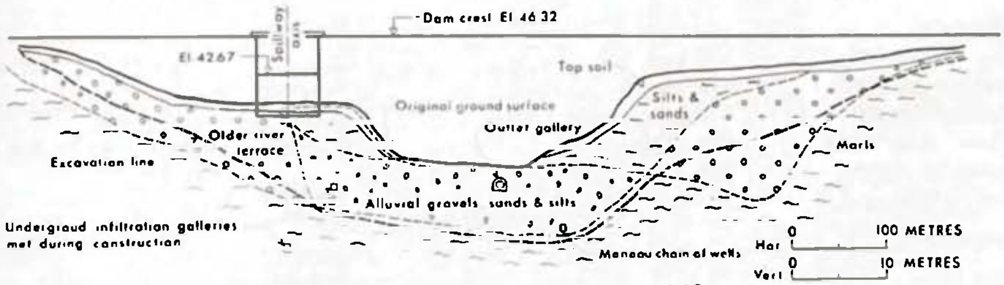
TREMITHIOS-KITI-DISTRIBUTION SYSTEM



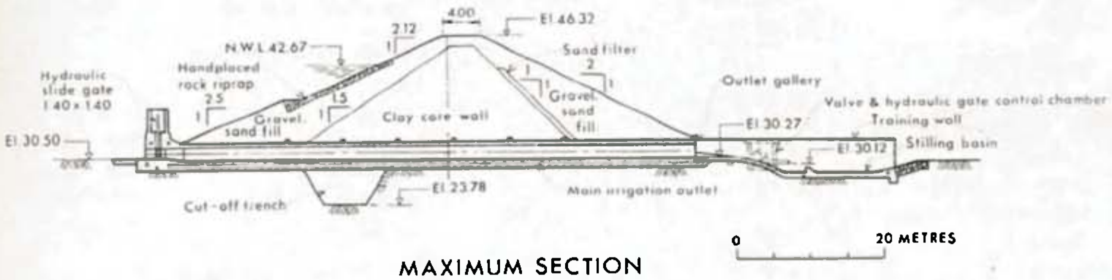
TREMITHIOS-KITI DAM



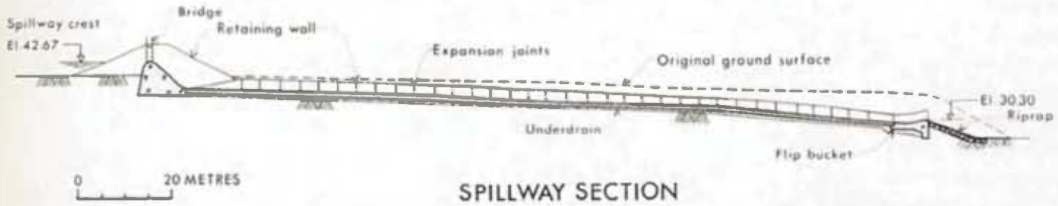
PLAN



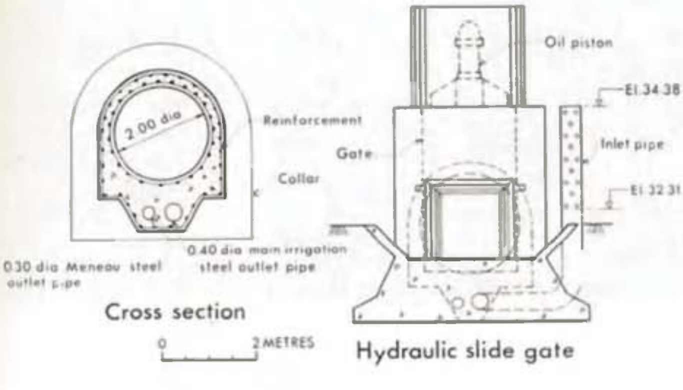
GEOLOGICAL SECTION ON DAM AXIS



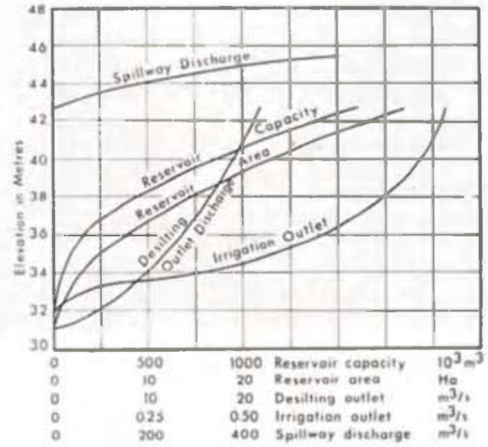
MAXIMUM SECTION



SPILLWAY SECTION



OUTLET GALLERY



CAPACITY-AREA-DISCHARGE CURVES

ding amortization of the total investment over 40 years at 7% rate of interest as well as operation and maintenance costs.

Two thousand donums of the land are commanded by the distribution system, the main crops being citrus, artichokes, melons and water melons, potatoes and beans. The maintenance of the dam and headworks is undertaken directly by the WDD.

7. PROBLEMS OF SPECIAL INTEREST

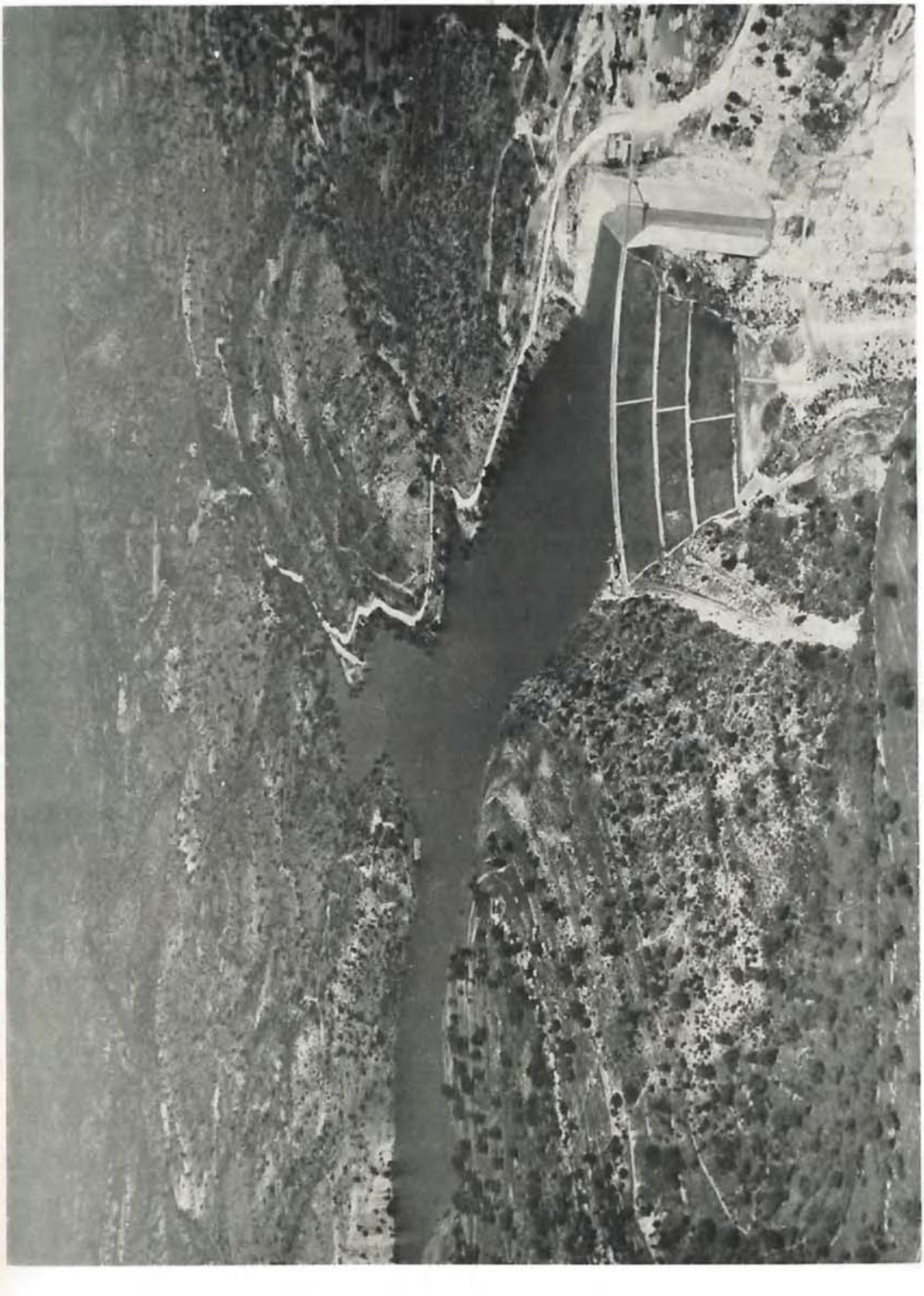
After impounding, several seepage problems developed, the most important of which were: Seepage through the "Bekir Pasha" chain of wells on the East abutment through surface overflow from the reservoir, as well as lateral infiltration through the permeable formations. This seepage was largely stopped by filling in the chain of wells with clay. Also seepage occurred under the spillway through an old chain of wells not previously explored. This seepage was also stopped by filling in the wells with clay.

Some seepage occurred also from the West abutment under the old covered river terrace which was not completely cut off. Leaching of salty soils upstream, especially in the West abutment, resulted to some accumulation of salts in the shallow groundwater table just downstream of the dam.

Data

i. Catchment	
Area	130 km²
Average rainfall	415 mm/a
Average runoff	6 million m³/a
1/1000 years flood	610 m³/s
Maximum height	600 m
Maximum length	27 km
ii. Reservoir	
Area	36 ha
Capacity	1.6 million m³
Live storage	1.5 million m³
Length	1,500 m
iii. Embankment	
Structural height	22.54 m
Height above ground level	15.82 m
Hydraulic height	12.17 m
Depth of foundation cut-off	6.72 m
Freeboard	3.65 m
Crest length	1,075 m
Top thickness	4 m
Base thickness	70 m
Upstream slope	1:2.5, 1:2.12
Downstream slope	1:2
Upstream core slope	1:1.5
Downstream core slope	1:1
Minimum core thickness	3 m
Maximum core thickness	34.5 m
Random fill	90,000 m³
Clay core	75,000 m³
Sand filter	5,200 m³
Rock rip-rap	2,300 m³
Total earth fill	172,500 m³
iv. Excavations	
Total	27,500 m³
v. Spillway	
Size	62x3.6 m
Capacity	610 m³/s
Length	150 m
Concrete	4,800 m³
vi. Gallery	
Size	2 m dia
Capacity	11 m³/s
Length	70 m
Concrete	450 m³
Operating gate size	1.30x1.35 m
vii. Outlets (Steel Pipes)	
Size	
Low level	0.38 m
Low level	0.3. m
High level	0.3. m
Capacity	0.28 m³/s
Length	71 m
viii. Distribution System	
(i) Main conveyor (steel pipes)	
Size	0.52 m dia
Capacity	0.33 m³/s
Length	2,400 m
ix. Distribution Network	
(i) R.C. cast-in-situ channels	
Size	0.32x0.25 to 0.85x0.38m
Capacity	0.008 to 0.28 m³/s
Length	10,850 m
(ii) R.C. Prefabricated canals	
Size	
Maximum width	0.62, 0.61, 0.46 m
Depth	0.46, 0.31, 0.34 m
Capacity	0.03 to 0.14 m³/s
Length	17,430 m

POLEMIDHIA



The Polemidhia dam.

GARYLLIS—POLEMIDHIA PROJECT

1. PURPOSE

This dam was constructed on the Garyllis river North of Limassol for the main purpose of supplying water to the region of the Akrotiri peninsula and more particularly to the Eastern part of it around the village of Zakaki. In the whole of that part of the peninsula, the extent of the citrus plantations was such that overpumping of the underlying aquifer resulted to sea intrusion which contaminated the groundwater supplies all along the coastal part of the aquifer.

In view of this sea intrusion which reached a conentration of up to 2,000 ppm NaCl, it became essential that additional fresh water should be imported to the region. For this reason the Polemilhia dam was built and a pipe conveyor was laid from the dam to the Zakaki region. Together with the construction of this dam a control on the groundwater extraction has been enforced so that the intrusion would be controlled. This means that the equivalent amount of groundwater extraction reduced by the control measures has to be supplied directly from the dam to each individual farm.

2. LOCATION

The dam was built on the Garyllis river at a distance of about 8 km upstream of the sea and at an elevation of about 100 m asl.

3. PLAN

a. Water and Land

The flow of the Garyllis river is estimated to be of the order of 7 million m^3/s with a maximum flood of a frequency of 1/1000 years of about 400 m^3/s.

Water had been used from this river downstream of the dam through the Polemidhia village intake situated a few hundred metres downstream of the dam. As the river flow ceased immediately after the winter floods, it was possible to irrigate through this intake only a few hundred donums of cereals. Therefore, with the construction of this dam a certain quantity of water had to be provided to satisfy the intake rights.

The total area commanded by the Polemidhia dam project at the region of Zakaki all of which are existing plantations of citrus, is about 9,270 donums. A limited extent of vegetable farms has also been incorporated into the Polemidhia dam distribution system.

b. Geology

The damsite itself is made up predominantly of alternatively laid beds of chalky marls, chalks, and calcareous sandstones of the Pakhna Formation. Also blocks of loose limestone can be met on the East abutment. Generally, the top of the formation is covered by a secondary limestone 'Kafkalla'.

From the characteristics of the rock structural features and the rock formation it has been concluded that a fault exists along the riverbed which caused the sinking of the East abutment. This abutment, due to the tectonic deterioration is composed of deteriorated sediments. The rocks on this abutment are sunk by falling and afterwards separated blocks the interspaces of which were filled by secondary materials. The degree of fissuration of the broken sandstones zone of the East abutment indicated that a considerable extent of grouting would be required.

The tunnel was situated on the West abutment, and it was considered that the low strength of the sediments allowed easy excavation but the marls would require the concrete lining of the excavated profile without delay because of danger of swelling and local sliding. The spillway itself was sited on the East abutment mainly for topographical reasons. The foundation conditions of the Pakhna formation are equivalant to the state of decomposed clay cemented by chalk materials. The base of the chute of the spillway discharging the overflow water to the level of the riverbed is made of sedimentary rocks of thick bedded sandstones and thin bedded chalks and marls, as well as of a part of decomposed chalky clay. Owing to the relatively poor physicomechanical properties of these rocks, the whole of the spillway chute had to be lined in concrete to avoid the erosion of these rocks.

The investigations carried out at the site showed that the riverbed level itself contained only shallow sediments of river deposits made up mainly of gravel sand and silt overlying the marl and therefore a relatively cheap cut-off would be required with very little grout consumption. However, the investigation on both the East and West abutments revealed soft layers of chalks, marls and sandstones and in many instances wide open channels could be traced through these beds, which indicated that the grouting requirements especially on the East abutment would be extensive.

4. MAIN FEATURES

a. Dam

The detailed design for this dam was carried out by Consultants EnergoProjekt of Yugoslavia.

Due to the foundation condition of the abutments, an earth fill type of dam was chosen. The basic material used for filling was the alluvial deposits from the riverbed and the excavated material from the spillway and the cut-off The clay for the central clay core was brought from a borrow area located near the Limassol road at about 2.5 km from the damsite. The transition layers of the dam were made from selected gravel and sand obtained from the alluvial terrace at the damsite. The upstream side of the dam is protected by rip-rap made of sandstones obtained locally.

The outlet tunnel excavated in the West abutment served for diversion purposes during construction, whereas after the completion of the dam, the diversion tunnel operates as a bottom outlet. The 0.38 m dia supply outlet for irrigation is also housed into this tunnel leading water from the upstream side to the downstream intake.

The spillway was excavated on the East abutment and is designed for a maximum head of five metres including freeboard. The overflow weir structure is of the gravity type. A layer of sand, gravel and pipes of 0.20 m dia for drainage are provided under the chute to relieve any hydrostatic pressure.

The chute channel ends with a bucket structure at riverbed level which is designed to protect it against erosion caused by jetting water.

b. Distribution System

The design of the distribution system was carried out by the WDD. It included a main asbestos pressure pipe 0.52 m dia, for a length of 8 km to reach the Zakaki citrus plantations. Within the Zakaki area an asbestos pipe pressure distribution network was provided.

5. CONSTRUCTION

The work was executed by the Contractors Mowlem-Ridgeway on a cost-plus basis. The reason for this is that the Contractors were also the financiers for this project and the cost-plus basis was part of the financing agreement. The profit on the job was agreed to be 8% and the overhead charges 5.82%. The work started in 1963 and completed in August 1965.

The specialized sub-contractors Soil Mechanics — Soletanche were employed to carry out the grouting works.

The supervision of the work was done jointly by the Consulting Engineers, EnergoProjekt of Belgrade, Yugoslavia and the WDD.

The cost of the dam was £809,000 including £200,000 for the grouting works carried out by the sub-contractors, and £28,000 spent by the Department on additional grouting works after the completion of the Contract.

The distribution system from this dam which was started in 1967 and completed in 1968 consisted of a main pipeline conveying water from the dam to the Zakaki area where it was distributed by A C pipes to the various farms of the region. The total cost of this scheme was £159,000 and was all constructed by the WDD.

The work on the embankment, tunnel, shaft and spillway did not encounter any important problems. Many caverns were met during excavations, especially on the East side, which were openings in the solution channels of the calcareous rocks and it became evident that the most important problem on the dam was the permeability of the abutments especially the East one where grouting works were expected to be extensive.

The foundation strata in the West bank was of uniform thin bedded sandstone and chalky marl, its permeability varying between 25 and 70 lugeons. After the completion of two stages of grouting the permeability was reduced to a maximum of 25 lugeons. In general, the permeability was found to have decreased to more than ten times.

On the East abutment the foundation strata consists of two zones of varying permeabilities. The upper zone consists of a thick bed of heavily broken sandstones with open cracks and partly filled up with chalky clay. The lower zone is composed of thin bedded sandstones and chalky marls which are generally of low permeability being less than five lugeons. The permeability of the upper zone was tested before grouting to be up to 1600 lugeons.

The whole grout curtain from the West to the East abutments was divided into six zones, but as the amount of expenditure on grouting became high, especially on the East abutment it was decided to stop the contracted grouting works as far as zone 4 only, the remaining work from part of zone 4 to zone 6 to be executed by the WDD.

The grouting works carried out by the WDD were commenced in August 1969 and completed in October 1970. Core drills and pipe casings of 75 mm dia were used. Stage grouting was adopted in lengths of between 1.5 to 7.5 m depending on the conditions of the rock. A variety of grout mixtures have been used depending on the condition of each separated stage and were composed of water, cement, clay, sand and bentonite.

The grouting was usually separated with thin mix ture 6:1 water-solids ratio except where the water tests and other informations indicated big cracks and cavities in which case the start was made with 2:1 mix. Gradually the mixture was thickened up, adding clay and sand and by changing the rate of water and solids until pressure could be created.

It was considered that a stage of grouting was finished when the consumption was less than 0.03 m^3 of grout mix for:-

20 minutes with a pressure less than 3.5 kg/cm^2
15 minutes with a pressure between 3.5 — 7 kg/cm^2
10 minutes with a pressure between 7 — 10.5 kg/cm^2
5 minutes with a pressure of over 10.5 kg/cm^2

The grout pressures which were used differed for the first and for the second phase. In the first phase the pressures were based on 0.07 kg/cm^2 per 0.3 m and in the second phase 0.14 kg/cm^2 per 0.3 m were applied.

Leather and rubber packers were used, placed on the top of the hole for the primaries and on the top of each stage for the secondaries.

It was originally decided to carry out the grouting in stages by drilling with overburden rig using casing down to the bottom and then withdrawing the casing in 1.5 to 4.5 m lifts and injecting the hole from the bottom to the top of the curtain. This was tried in two holes but it was not possible to withdraw the casing after a quantity of grout had been injected. With the packer placed at the bottom of the casing this was probably due to leakage of the grout upwards between the casing. So it was decided to carry out the grouting work in descending stages. The average grout takes were about 1,250 kg/m, but within the upper 1.5 m zone the average grout takes were appreciably higher reaching up to 10,000 kg/m.

A summary of the grouting results is as given on Table 12.

amount and the deficit which results out of this control for the irrigation requirements is allocated from the water of the dam.

7. PROBLEMS OF SPECIAL INTEREST

As already indicated the main problem of this dam was the excessive leakage from the East abutment which was due to the big cavities in the Pakhna For-

POLEMIDHIA DAM — GROUTING TABLE 12

Zone No.	Number of Boreholes	Number of Stages	Drilling for grouting Metres	Drilling through over-burden Metres	Total Drilling Metres	Cement consumption Kg	Bentonite consumption Kg	Clay consumption Kg	Sand consumption Kg	Total consumption of dry material Kg	Average grout takes Kg/m
I	56	433	1,342	1,271	2,613	655,944	55,157	537,954	—	1,249,055	930
II	41	198	594	1,016	1,610	193,000	17,000	85,000	—	295,000	498
III	45	498	1,610	220	1,830	1,570,000	128,573	1,132,000	—	2,830,573	1,755
IV	37	273	1,414	179	1,593	1,392,000	110,372	1,003,000	—	2,505,372	1,770
V	105	764	2,860	1,620	4,480	962,000	11,000	329,000	340,000	1,642,000	575
VI	15	49	309	414	723	43,000	1,031	3,969	—	48,000	150
Total	299	2,220	8,129	4,720	12,849	4,815,944	323,133	3,090,923	340,000	8,570,000	1,054

6. MANAGEMENT

The operation of this project is under the control of the Government through a Project Committee made up of the District Officer as Chairman, and the Engineer of the WDD and the District Agricultural Officer as members.

The maintenance of the dam itself and the main conveyor system is under the responsibility of the WDD. Water is sold to the farmers of the region at 15 mils/m³ for the existing citrus plantations and at 10 mils/m³ for vegetables and vines. The surplus water from the dam during winter time is sold at 5 mils/m³ for surface irrigation and at 2 mils/m³ for the recharge of wells. The actual cost of water considering the investment cost of the dam and distribution system amortized over 40 years at 7% rate of interest plus operation and maintenance costs is about 42 mils/m³.

About 400,000 m³ of water is left as compensation water for the Polemidhia village intake which used to derive water from Garyllis River before the dam was constructed. The area that is commanded at Zakaki by the distribution network is a total of 1,700 donums out of which 720 are citrus, 560 vines and 420 vegetables. The selling of water to the farmers of the region is done parallel with the application of the special Measures Law through which the extraction of water from the wells of the aquifer is controlled to a restricted

mation. The Contractor had to be stopped from going on with the grouting works on a cost-plus basis and it was decided that the WDD should do all additional grouting works as would be required. Before going on with these additional works, a number of observations for the leakages were decided.

From the piezometer records taken and the hydraulic gradient through the East abutment the total leakage and average permeability of the rock of the abutment was estimated to be about 1,000 lugeons. Since the water flow was taking place through localized zones only, the actual permeability of these zones was expected to be higher. The seepage through the West abutment before the extension of the grouting works was carried out by the WDD was considerable and varied tremendously as the reservoir level rose to within the last 7 m from the spillway level, to such an extent that one could say that there was an underground spillway operating on the West abutment.

The leakage generally started when the reservoir elevation was about 17 m above river bed level and increased uniformly for the next 6 m. For the last 7 m up to spillway level, the rate of increase was much greater. It was observed that there was a time lag of 3 days for the water to seep through the abutment from the reservoir.

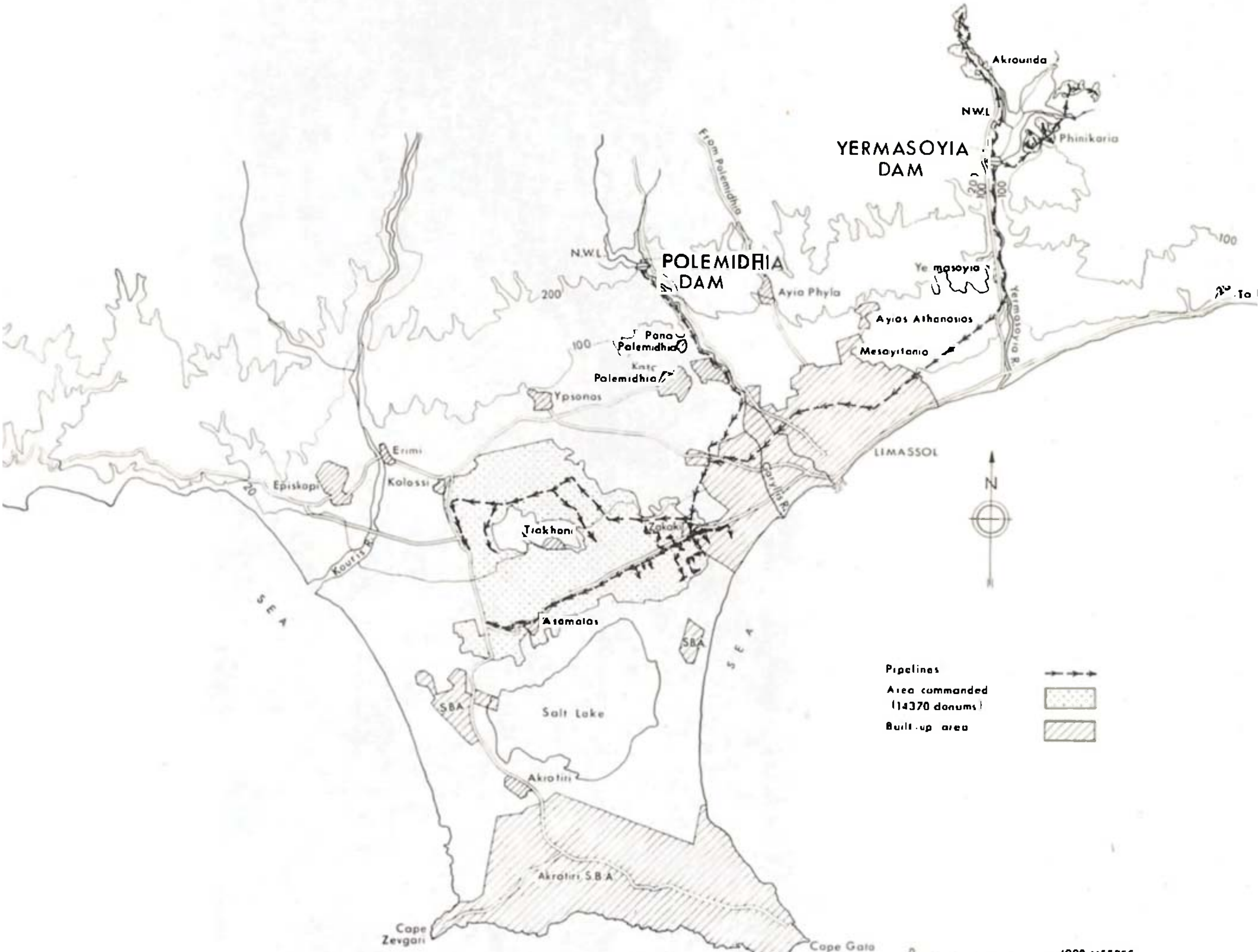
A programme of investigations were then carried out in 1968 which were made up of 7 exploratory boreholes drilled and grouted in 1.5 m stages up to 4.5 m into bedrock. This exploratory work served for determining the relatively impermeable bedrock and for designing the extension of the grout curtain.

As a result of these investigations a single row of grout curtain was drilled starting the grouting from top water level. The grout mixtures and method of grouting has been already described.

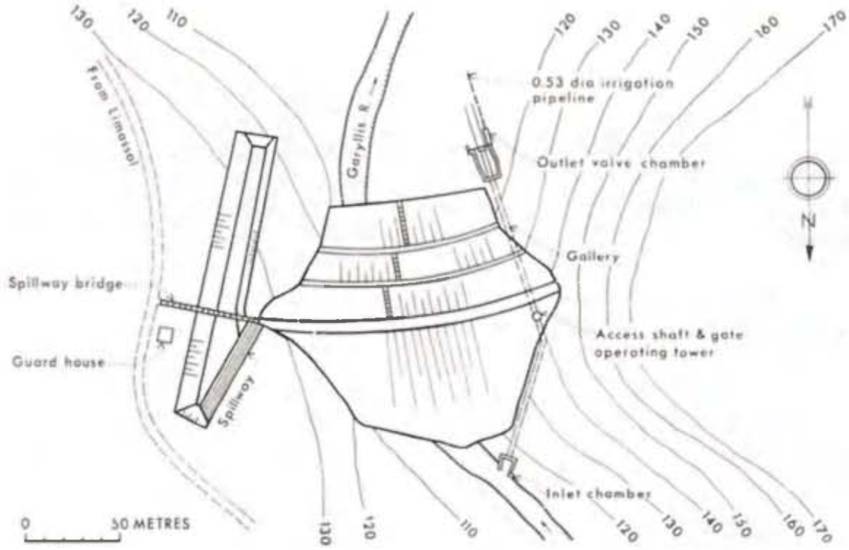
After the completion of grouting in primary and secondary holes, water tests were carried out in stages of 4.5 to 6 m with packers placed at the top of each stage. With pressures between 8 and 11 atmospheres the permeabilities were found to be 15 lugeons as compared with 1,600 lugeons before grouting. It was concluded that for the time being the grouting curtain was satisfactory and it had now to be tested under natural reservoir conditions which since the completion of the grouting work early in 1970 have not materialized as there has not been sufficient runoff to fill more than half the reservoir. From the records of seepage available it was also observed that there was a decrease of seepage in time for the same water level which may be attributed to the silting up of the reservoir and clogging the path of flow. It is therefore necessary that the new grout curtain be tested for some time before it can be decided to invest more money on additional grouting works.

Extensive seepage with landsliding on the East abutment.

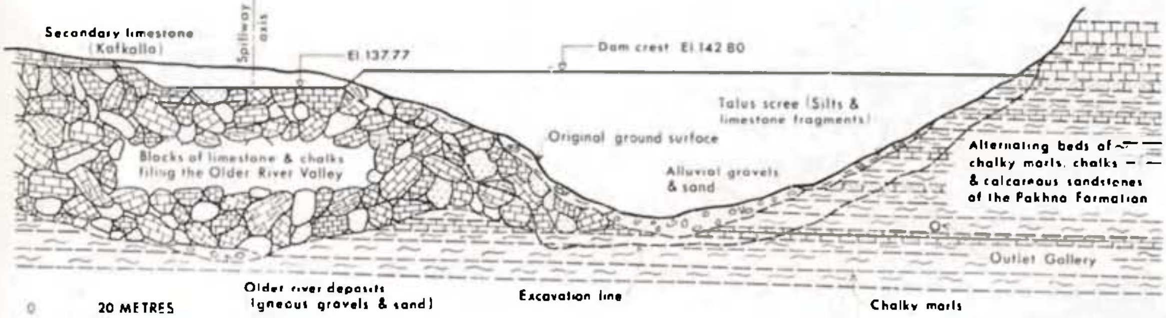
YERMASOYIA-YERMASOYIA & GARYLLIS-POLEMIDHIA DISTRIBUTION SYSTEM



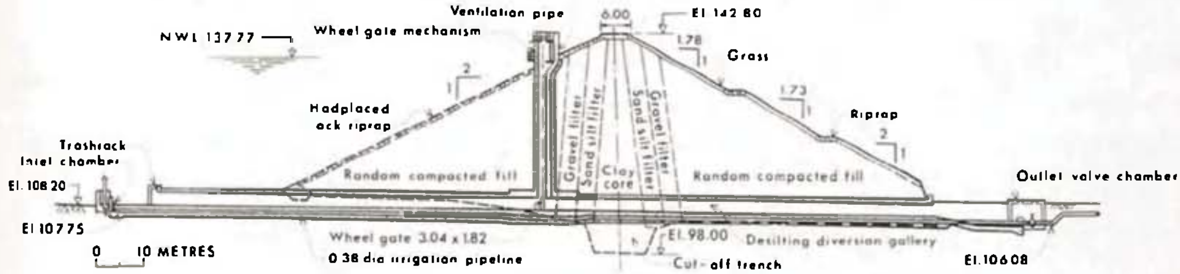
GARYLLIS - POLEMIDHIA DAM



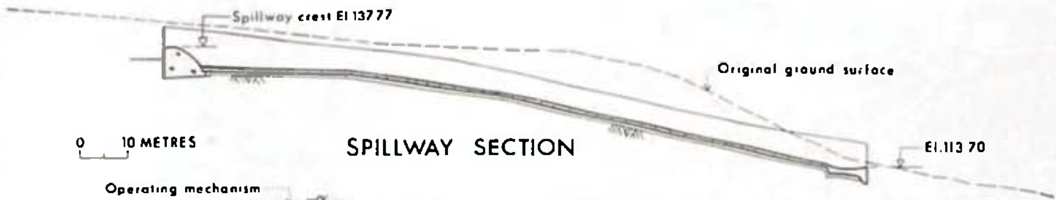
PLAN



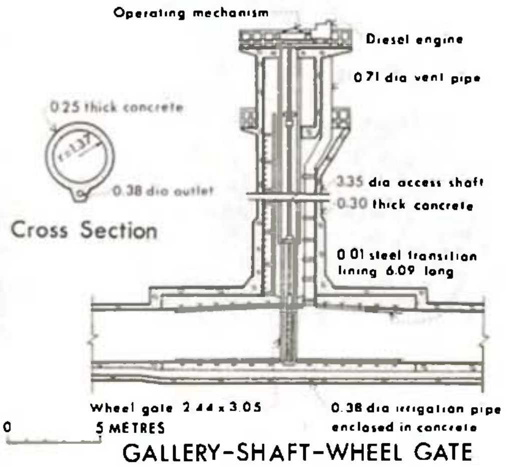
GEOLOGICAL SECTION ON DAM AXIS



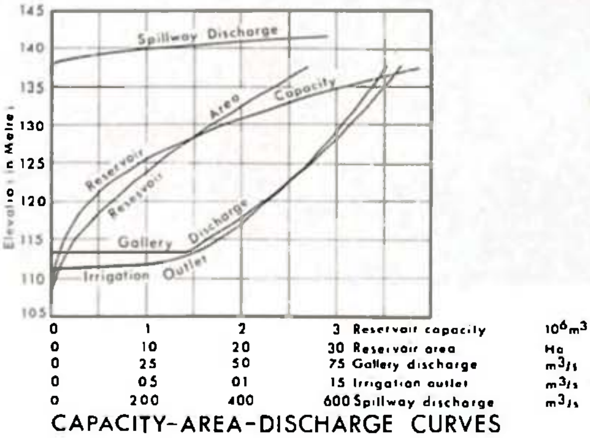
MAXIMUM SECTION



SPILLWAY SECTION



GALLERY-SHAFT-WHEEL GATE



CAPACITY-AREA-DISCHARGE CURVES

Data

i. Catchment

Area	75.6 km^2
Average rainfall	582.0 mm/a
Average runoff	7.00 million m^3
1/1000 years flood	400 m^3/s
Maximum height	740 m
Maximum length	17.9 km

ii. Reservoir

Area	11 ha
Capacity	3.86 million m^3
Live storage	3.7 million m^3
Length	1,920 m

iii. Embankment

Structural height	44.80 m
Height above ground level	34.60 m
Hydraulic height	29.57 m
Depth of foundation cut-off	10.20 m
Freeboard	5.03 m
Crest length	170 m
Top thickness	6 m
Base thickness	131 m
Upstream slope	1:2
Downstream slope	1:1.78, 1:1.73, 1:2
Upstream core slope	1:0.1
Downstream core slope	1:0.1
Minimum core thickness	3.04 m
Maximum core thickness	10.66 m
Random fill	135,000 m^3
Clay core	23,500 m^3
Gravel, sand filter	52,500 m^3
Rock rip-rap toe and parapet walls	4,000 m^3
Total earth fill	215,000 m^3

iv. Excavations

Total	110,000 m^3

v. Foundation Treatment

Total drilling depth for grouting	12,849 m
Total grout take	8,570,000 kg
Average grout take	1,054 kg/m

vi. Spillway

Size	45.70x3.65 m
Capacity	580 m^3/s
Length	134.10 m
Concrete	4,600 m^3

vii. Gallery

Size	2.74 m dia
Capacity	92 m^3/s
Length	167.63 m
Concrete	830 m^3
Operating gate size	3.04x1.82 m

viii. Access Shaft

Size	3.35 m dia
Depth	32 m
Concrete	150 m^3

ix. Outlet (Steel pipes)

Size	0.38 m dia
Capacity	1.77 m^3/s
Length	182.87 m

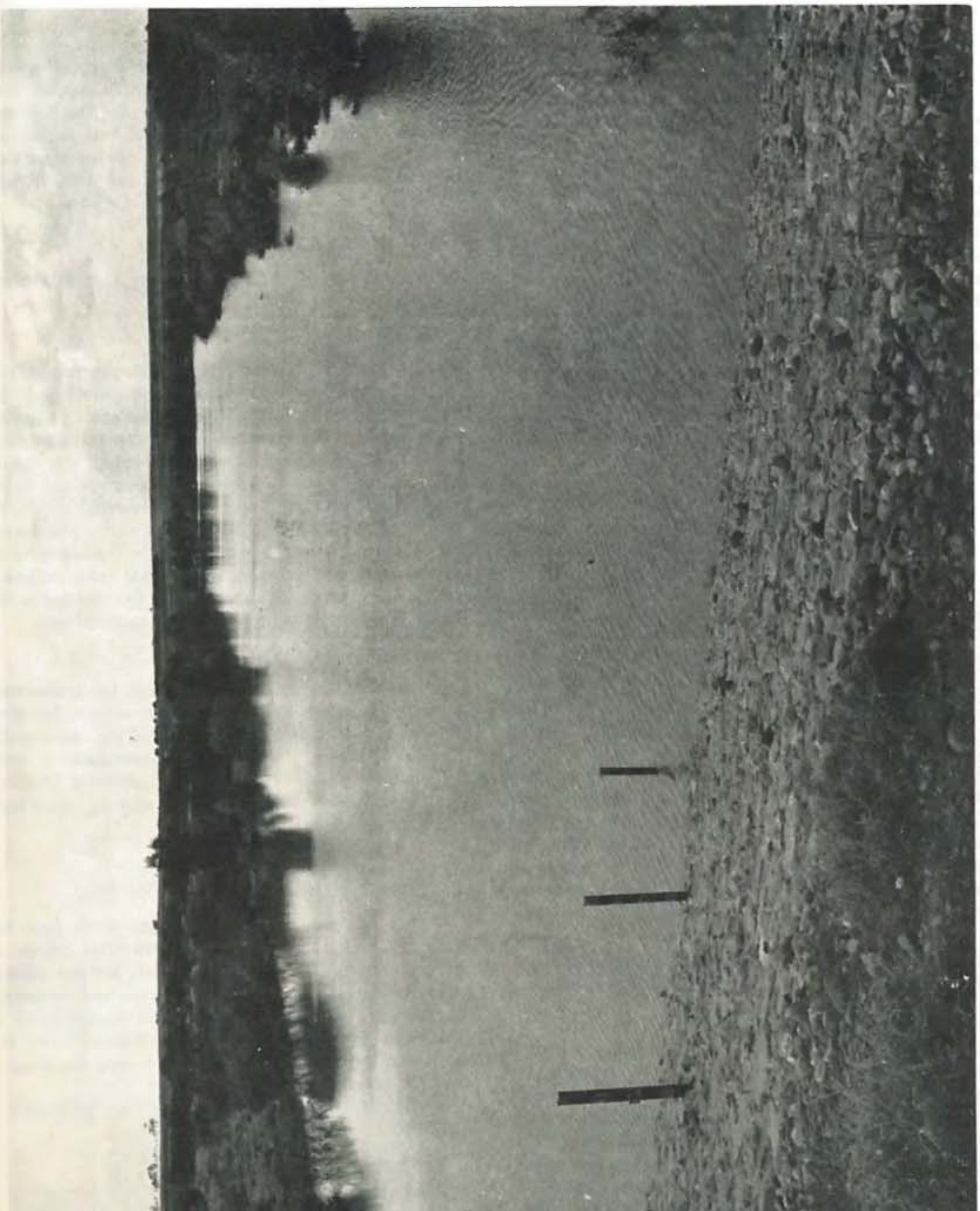
x. Distribution System

(i) Main conveyor (AC pipes)

Size	0.60 m dia
Capacity	0.56 m^3/s
Length	7,792 m

(ii) Distribution Network (AC pipes)

Size	0.15 m dia
Capacity	0.02 to 0.1 m^3/s
Length	11,222 m



The Liopetri dam.

POTAMOS — LIOPETRI PROJECT

1. PURPOSE

The purpose of constructing this dam was for groundwater recharge of the sandstone aquifer. Excessive pumpage from numerous boreholes has resulted to a drop of the water table at an average rate of 0.8 m annually. The sandstone aquifer outcrops at the dam and is continuous from the surface right down to the water table. Therefore, the construction of the dam which would impound water in direct contact with the sandstone outcrop, would enable the infiltration of water into the aquifer. Potamos is the main stream of the region and under heavy floods which yield significant quantities of water, infiltration could materially benefit the groundwater resources. The distance of the dam from the sea, the water table which is below the sea level, and the numerous boreholes in between, are advantages for the prevention of sub-surface losses to the sea from the dam.

2. LOCATION

The dam is situated on the Potamos stream about 2.5 km upstream of the sea and 3 km South of the village Liopetri. It's altitude is about 12 m asl.

3. PLAN

a. Water and Land

From readings of an automatic flow recorder which was installed on the tributary of this stream since 1956 and from rainfall data, the average estimated runoff is of the order of 0.16 million m^3/a. The water of this stream was not used before the construction of the dam being in the form of floods which last for a few hours in the year. Under such runoff conditions the only way to utilize the flow is by storage.

The irrigation practised in the region from groundwater is mainly for potatoes, carrots, other vegetables and some citrus plantations, through sprinkler systems.

b. Geology

The dam and reservoir lie on the Athalassa—Nicosia Formation which is made up of alternating beds of sandstone and marls. The marl forms the impermeable base of the aquifer whilst the sandstone forms the aquifer and in most cases outcrops at the reservoir. The depth of the aquifer reaches up to about 60 m at the damsite and is well below sea level. The danger of sea intrusion exists and it is one more reason for the importance of groundwater recharge of this region. The permeability of the sandstone outcrops in the reservoir is quite good. However, the silt transport in the reservoir is high due to the soft rocks and the little vegetation cover of the catchment and the clogging of the sandstone constitutes a serious problem. It is therefore required to scrape the reservoir from the silt once every 2 or 3 fillings, in order to maintain a high rate of infiltration. The flat topography of the reservoir and the fact that the silt can easily be transported and brought to nearby fields makes the scraping of the reservoir an economic scheme.

4. MAIN FEATURES

The design work for this dam and distribution system was done by the WDD.

a. Dam

The dam is made up of a homogeneous random fill embankment with a downstream gravel and sand filter. On the upstream, the embankment is protected by a hand placed rip-rap rock. The embankment is keyed into the sandy marl where the cut-off was formed. The concrete lined spillway has been provided on the right bank with an earth chute reaching down the river bed level. A 0.30 m dia steel outlet pipeline has been embedded in the dam from the upstream to the downstream and encased in concrete with collars at suitable intervals. The operation of the pipeline is done through an outlet valve at the downstream side.

b. Distribution System

Although this is primarily a dam for groundwater recharge purposes yet an irrigation scheme for direct supply of water from the dam to the surrounding gardens is under study, the implementation of which will help in the greater and more efficient utilization of the water of the dam during periods when water is required for irrigation.

5. CONSTRUCTION

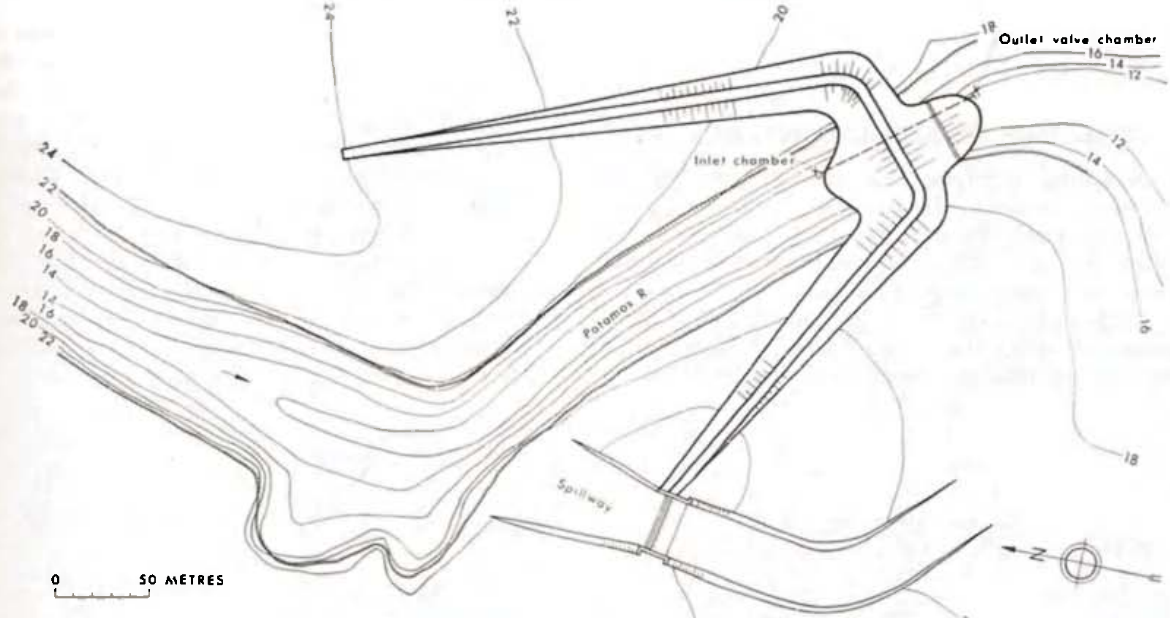
The dam was constructed by the WDD from July 1964 to April 1965. No serious difficulties cropped up during construction and all materials for the embankment were obtained from borrow areas and excavations at the site. The compaction of the homogeneous fill was done by pneumatic rollers in 0.20 m layers at a dry density of 1,540 kg/m^3 and at an optimum moisture content of about 22%.

The total expenditure on the dam amounted to £30,000.

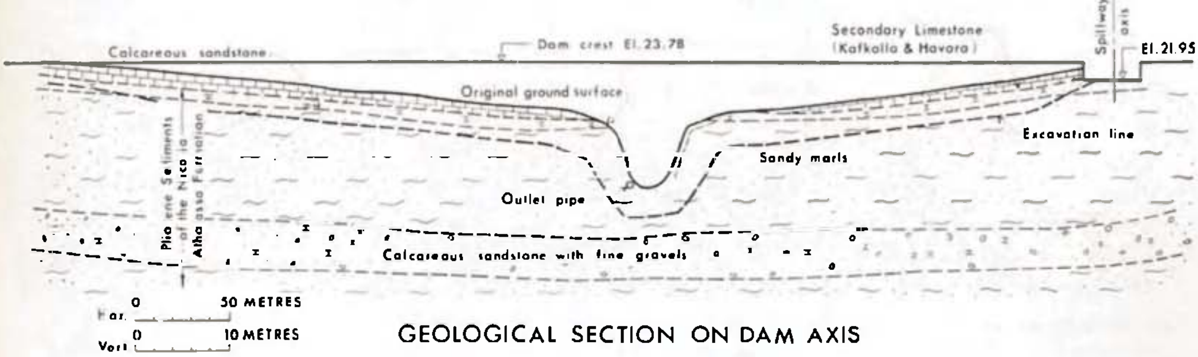
6. MANAGEMENT

This project is managed by the Liopetri Irrigation Division which has undertaken to contribute 25% of the total cost having obtained a loan from the Government payable in 10 years and at a rate of interest of 5%. The beneficiaries are about 140 owners of boreholes who use their own water supplies for the irrigation of their gardens. These people formed an Irriga-

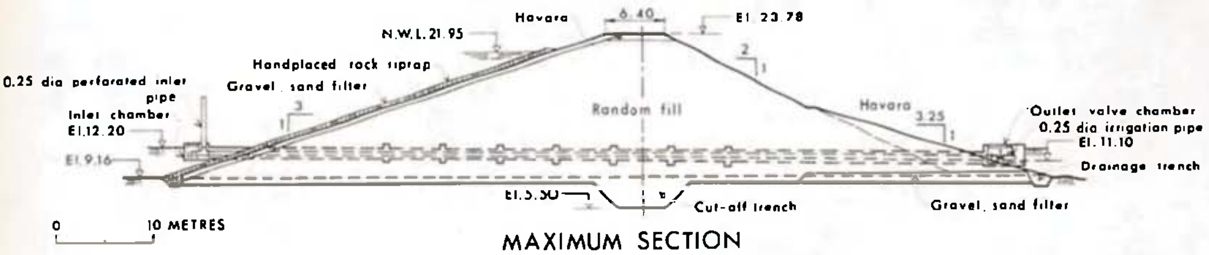
POTAMOS-LIOPETRI DAM



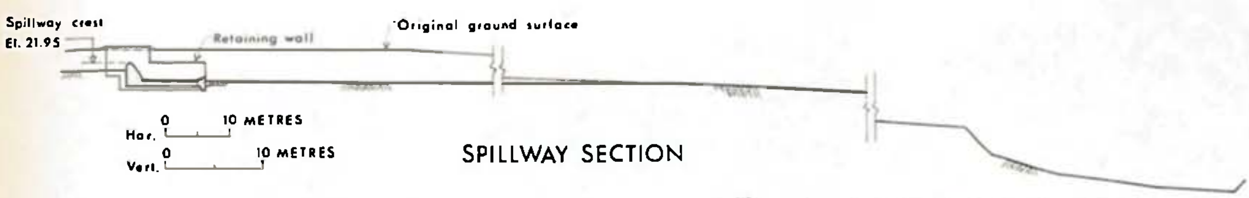
PLAN



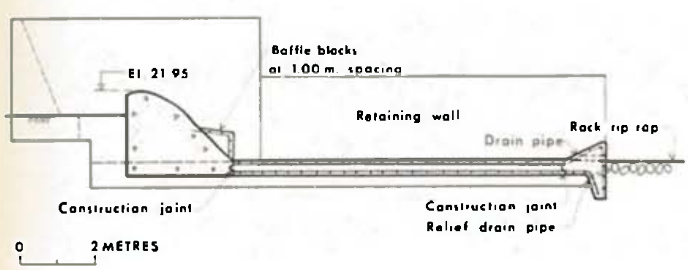
GEOLOGICAL SECTION ON DAM AXIS



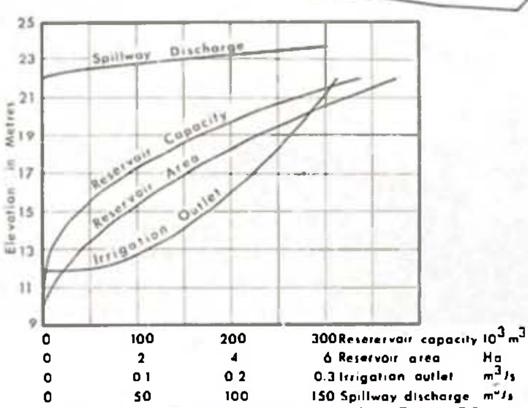
MAXIMUM SECTION



SPILLWAY SECTION



SPILLWAY SECTION



CAPACITY-AREA-DISCHARGE CURVES

tion Division which undertook the management of the dam.

7. PROBLEMS OF SPECIAL INTEREST

Regarding the dam itself there are no problems of special interest. However, the interesting points to be considered are the effects of recharge on the groundwater table after the construction of the dam. Before the construction of the dam the water table was dropping at about 0.8 m per annum, whilst almost immediately after the impoundment of water in the dam early in 1965 the water table rose by about 1 m as far as 300 m away from the dam and was reduced to a rise of 0.2 m at about 900 m from the dam. This recovery is largely attributed to the dam, for it can be seen from previous records that before the construction of the dam the water table was dropping continuously even under heavy rainfall conditions. Records of the water table are taken from a number of observation boreholes situated in the area. The rate of recharge from the dam during the first month of January 1965 was 4.500 m^3/day whilst in February it dropped to 3.150 m^3/day due to the lowering of the water level in the reservoir and the clogging effect from silt deposition.

Data

i.	*Catchment*	
	Area	36.5 km^2
	Average rainfall	384 mm
	Average runoff	0.16 million m^3/a
	1/1000 years flood	90 m^3/s
	Maximum height	80 m
	Maximum length	5.0 km
ii.	*Reservoir*	
	Area	7.4 ha
	Capacity	0.325 m^3
	Live storage	0.320 m^3
	Length	375 m
iii.	*Embankment*	
	Structural height	18.28 m
	Height above ground level	14.62 m
	Hydraulic height	12.79 m
	Depth of foundation cut-off	3.66 m
	Freeboard	1.83 m
	Crest length	540 m
	Top thickness	6.40 m
	Base thickness	90 m
	Upstream slope	1:3
	Downstream slope	1:2, 1:3.25
	Random fill	49,000 m^3
	Gravel, sand filter	2,500 m^3
	Rock rip-rap	3,000 m^3
	Total earth fill	54.500 m^3
iv.	*Excavations*	
	Total	22.000 m^3
v.	*Spillway*	
	Size	30x1.8 m
	Capacity	90 m^3/s
	Length	12 m
	Concrete	375 m^3
vi.	*Outlet (Steel pipes)*	
	Size	0.30 m dia
	Capacity	0.14 m^3/s
	Length	90 m

KALOPANAYIOTIS

The Kalopanayiotis dam

MARATHASA — KALOPANAYIOTIS PROJECT

1. PURPOSE

Kalopanayiotis is a village of Marathasa mountain region of Troodos and the villagers rely for their income mainly on growing deciduous trees. Small diversion weirs and distribution pipes or channels were used for the irrigation of limited areas of land.

The construction of the dam aimed at storing winter flow which is in abundance for utilizing it in summer when the flow is only just enough for the existing plantations. Therefore this dam was utilized solely for the expansion of irrigation to cover an area of about 435 donums on either side of the river bed downstream, with deciduous fruit trees, such as apples, pears, peaches and plums.

Kalopanayiotis and the other Troodos mountain villages require water for irrigation in order to enable the inhabitants to have sufficiently good income to stay in their villages. Although such schemes in mountainous regions are expensive due to the steep topography which requires relatively expensive dams and expensive land levelling for cultivation, yet the Government builds these projects because they help the people stay in their villages.

2. LOCATION

The dam is located on the Marathasa river at a distance of about 16 km from the sea or 7 km upstream of the Lefka Dam and at elevation of about 550 m asl.

3. PLAN

This dam is part of the plan for the development of the Troodos region where it is the policy to build small storage schemes for the extension of irrigation. A lot of such projects have already been built the major and most expensive one being the Kalopanayiotis project.

a. Water and Land

The catchment area of this dam at 26 km^2 is estimated to yield an average of about 11.6 million m^3/a of water. A great number of small diversion weirs have been built on this river with a major irrigation project at Lefka for which the Lefka dam was built downstream. To satisfy these and other water use rights a diversion pipeline has been placed in the reservoir taking water from upstream of the reservoir top water level.

The land for irrigation is situated on either side of the river downstream of the dam reaching a distance of up to 6.5 km and at an elevation of about 500 m asl. This land is quite steep and was, prior to the construction of the dam, mostly cultivated in vines used for the production of wine. This land lying on igneous rock and having been cultivated in vines, has enough soil cover but in many cases when the rock was exposed through land levelling it is easily crushed to soil during cultivation in a matter of 2 or 3 years.

b. Geology

The area covered by the reservoir and damsite is entirely occupied by diabase which consists of a series of parallel multiple dykes of andesitic or basaltic composition. The diabase is recognized by its sheeted aspect which has become more pronounced by a set of well developed joints running parallel to the strike of dykes.

On the Eastern flank there are remnants of old river terrace deposits consisting of gravels, cobbles and boulders embedded in silt.

In other places of the two flanks the diabase is covered by angular diabase fragments embedded in fine matrix referred to as talus.

The dominant strike of the dykes ranges from 130°—170°SE and they dip 30°—45°NE.

The rock of the area exhibits various degrees of weathering. The whole area is cut by numerous narrow discontinuous lense-shaped shear zones which are parallel to the strike of the sheets thus giving alternating bands of blocky and fractured rock.

In addition to the shear zones a major fault has been observed in the area. The major fault was found during the excavation for the cut-off trench near the West abutment. It crosses the dam axis at an angle of about 60°.

As a result of the complex geological setting of the damsite serious engineering problems were faced before, during and after the construction such as extensive grouting, deep cut-off trench and appearance of cracks due to sliding on the Western flank of the dam site.

About 21 boreholes, 5 test pits and one cutting were made in order to obtain information about the thickness of the overburden and the permeability of the rock.

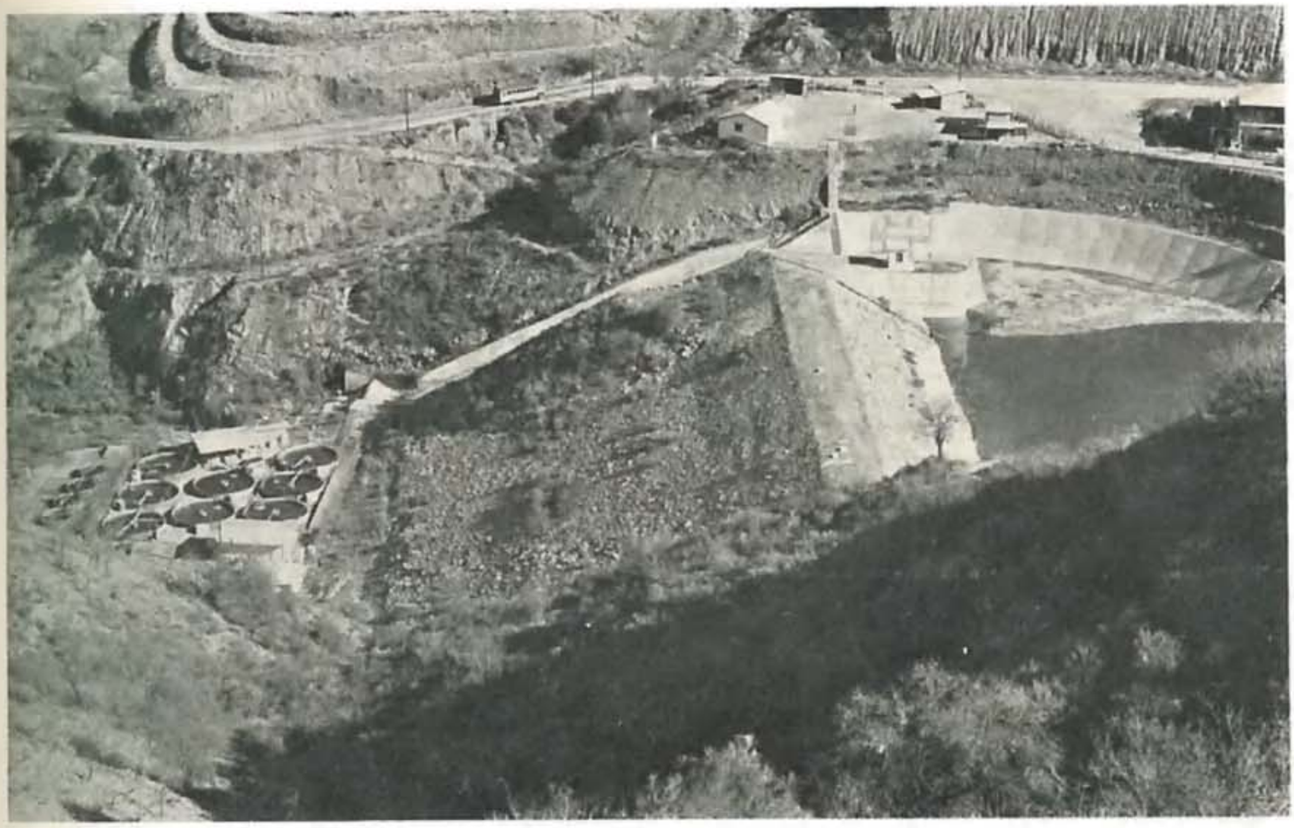
Some other boreholes and test pits were drilled in the clay borrow area to explore the soil characteristics and the quantities available for fill materials. Also tests in the WDD laboratory were conducted in order to obtain data for the design.

4. MAIN FEATURES

a. Dam

A detailed design for this dam was made by Messrs Howard Humphreys and Sons of London whilst the distribution system was designed by the WDD.

The embankment is made of random fill with rock from the excavated quarries. A central clay core is provided protected by filter zones on either side. A coffer dam to operate for the diversion of the flood during construction was embedded in the dam embankment.



Fish (trout) cultivation D/S of the dam.

A grout curtain was also provided for the rock underneath the river bed and the abutments, the weakest zone being in the left abutment.

An ogee overflow chute spillway with crest weir parallel to the dam axis was provided on the right abutment. Vertical retaining walls were provided on either side. A flip bucket was built at river bed level. The approach channel to the spillway is formed by a long circular wall on the right side and at the left side by a small semicircular wall extending into the embankment.

The outlet works structure is located on the East abutment. A forebay in front of the valve chamber has a channel about 23 m long and 4 m wide with retaining walls up to 55 m high. A valve chamber was provided to protect the gate of the tunnel and the inlet of the irrigation pipe. Trash racks made of steel protect the inlet to the tunnel. The tunnel is of reinforced concrete being 2.3 m dia the thickness of the lining being 0.45 m.

The entrance to the tunnel is controlled by a cast iron gate 1.21x1.21 m. This penstock is operated by an oil hydraulic cylinder installed 3.3 m above the centre of the gate plate. The cylinder is connected by hydraulic pipes to a hand operated oil pump which lies in the control house on the embankment crest and at an elevation of 16.5 m above the hydraulic cylinder. The capacity of the hand operated pump is of 1.4 l/m working at 130 atmospheres. Parallel to the hydraulic pipes a galvanized pipe 0.15 m dia was provided for the ventilation of the inlet to the tunnel.

An outlet irrigation pipe of 0.30 m dia is embedded in the right side of the concrete base of the tunnel and conveys water from the valve chamber to the outlet chamber at the downstream end of the tunnel. At the outlet chamber an irrigation meter is provided for measuring the flow.

Another 0.20 m dia pipe is embedded in the left side of the tunnel extending upstream through the reservoir to the top water level in order to divert water free of storage to the downstream side of the dam for water right use.

b. Distribution System

The distribution system provides for steel conveyor pipes of 0.10 to 0.30 m dia starting from the dam outlet and reaching the furthest point at a distance of 8 km downstream. A network of distribution pipes provides for the supply of water to each plot all along the narrow valley downstream of a total area of 435 donums of land. The maximum difference in elevation from the dam outlet to the lowest point is 250 m.

5. CONSTRUCTION

The construction of the dam was performed by direct labour under the supervision of the WDD and the Consulting Engineers. Grouting operations were partly executed by Foundation Engineering Ltd., of London, in co-operation with contractors Joannou and Paraskevaides of Nicosia, and partly by the WDD. The work started on the dam in September 1964 and completed in December 1966.

A coffer dam was built for the diversion of water through the tunnel which was excavated on the left abutment. The foundation of this coffer dam in the alluvium was grouted in order to avoid an extensive cut-off.

The excavation of the main cut-off was done mostly on the left abutment because of the weathered nature of the existing rock. No proper cut-off was possible on that side and the rock was finally covered with a concrete blanket 0.80 m thick. To excavate the foundation under the main river bed section dewatering had to be done through a number of pumps in order to keep the excavations dry. A fault was revealed during excavation and the rock was found to be much better on the East side than on the West side of the fault. The fault area was covered with a concrete blanket of 0.60 m thickness. The old river terrace on the East abutment below the spillway was completely excavated up to rock level for the first 8 m by mechanical means and then below that level up to a maximum depth of 34 m by hand. The cut-off excavation by hand was of 1.5 m width and was supported by timber shuttering all the way down.

The clay material for the core wall of the dam was obtained from the borrow area at the East flank of the damsite. It was made up of decomposed igneous rock soil containing angular fragments of diabase. The compaction was done in 0.15 m layers by heavy pneumatic rollers loaded to weigh 47.5 tons giving a pressure of 4.72 kg/cm^2. Twelve to sixteen single passes were applied. The maximum dry density of the clay was 1,940 kg/m^3 when the moisture content was 14.8%. The permeability of this clay as tested in the field was found to be 1×10^{-7} cm/s.

Transition zones were laid upstream and downstream of the clay core. The maximum size of the sand and gravel filter used was 40 mm with only 2% passing the BS 200 sieve. The vibrating roller was used to compact this material. For the upstream filter zone, material was obtained from the reservoir area whereas for the downstream filter zone and the 0.60 m thick horizontal filter layer upstream, material was brought from the Morphou sea shore. The material for the fill was obtained from the cut-off excavations and from the borrow area in the reservoir. Compaction of this random material was done by vibrating rollers.

Rock boulders for the rip-rap and the rock toe were obtained from the excavations by selecting the large pieces. The rip-rap was placed by hand.

For the spillway floor longitudinal and transverse 0.10 m dia tiled drains were provided, covered by gravel filters. A 0.30 m thick gravel drain blanket was placed along the back of the spillway side walls to drain the natural back slopes behind. All joints of the side walls and the spillway were sealed with PVC water stops and covered with flint coat. The steel foot bridge above the spillway was erected after completing the concrete works.

The excavation for the tunnel was in rock, most of it sound enough not to require any supports during excavation. Grouting rings were provided at 4 m spacings. The 0.30 m dia steel irrigation pipe and the 0.20 m dia steel diversion pipe were placed in the tunnel embedded in concrete on the floor level. The concrete in the tunnel was placed by means of a con-

Hose-basin irrigation of deciduous trees planted on terraces.

crete pump through a pipe 0.10 m dia.

In Addition to this protective measure, drainage works were carried out above the elevation of the crack to convey rain water away from the crack zone.

The Contractors did most of the grouting work required on the East abutment including the main river bed section. The WDD completed the grouting work in the East abutment after the contractor left and carried out also the grouting works on the West abutment. Drilling and grouting was performed in primary holes spaced at 4 m centres with 3 m stages followed by drilling and grouting of secondary holes midway between the primary ones. Then necessary tertiary holes were drilled in — between primary and secondary holes for testing and supplementing grouting. All holes were drilled down to rock where they were keyed. The pressure for injection of the grout varied according to the depth below surface. The pressure in pounds per square inch were approximately made equal to the depth in feet. The grouting mixture started with a ratio of 1:8 cement water and was thickened depending on the consumption up to 1:1 with the addition of one grouting.

In the case of the tunnel, the total grouting drilling depth was 145 m and the total grouting take 114 tons of cement.

The grouting in the tunnel was carried out on 21 rings along the length of the tunnel through steel pipes which were built in concrete during construction. On the three central rings 8 holes were drilled on the circumference and grouting proceeded in 3 m stages. For each of the other 18 rings. 4 holes were grouted only.

The construction of the distribution system was all carried out by the WDD and was completed in December 1967. The total cost of the dam was £236,000 including £50,000 on grouting works. In addition £64,200 was spent on the pipe distribution system.

All expenditure was incurred by the Government.

6. MANAGEMENT

The operation of the project is done through a Project Water Committee made up of the District Officer as Chairman, the Regional Engineer of the WDD and the Regional Agricultural Officer as members. The maintenance is under the direct responsibility of the WDD.

Water is sold and measured through a water meter to each farmer at 13 mils/m^3. Water is used very efficiently through an aluminium pipe network supplying a hose basin system made up of plastic hoses which in turn supply water to each basin. The irrigation efficiencies reached here are about 80% of the water supplied. There is a great interest by the farmers on this project and more water is demanded for the extension of crops.

GROUTING WORK CARRIED OUT — TABLE 13

Zone	Number of Boreholes	Number of Stages	Drilling thro'clay concrete or overburden Metres	Drilling thro'rock for grouting Metres	Total Drilling Metres	Cement consumption Kg	Bentonite consumption Kg	Sand consumption Kg	Total consumption Kg	Average grout takes Kg/m
East abutment including spillway weir block	73	127	530	388	918	18,850	—	—	18,850	48
River bed	82	366	542	1,728	2,270	173,500	22,100	7,000	82,600	106
West abutment	46	313	1,180	1,030	2,210	303,000	24,350	29,000	356,350	346
Total	201	806	2,252	3,146	5,398	495,350	26,450	36,000	557,800	177

part in weight of sand. Bentonite was also added in order to increase the viscosity. Grouting was continued until the hole could take grouting at the rate of less than 0.028 m^3 for the following pressures:-

3.5	kg/cm^2	in 20 minutes
3.5—7	kg/cm^2	in 15 minutes
7—14	kg/cm^2	in 10 minutes and for pressures exceeding
14	kg/cm^2	in 5 minutes

The West abutment was mostly drilled and grouted from the crest top after the completion of the embankment. A total of 5 rows were grouted, a number of them inclined at 60° to the vertical. Stage grouting was done in the West abutment from the crest and the maximum depth reached was 70 m. The existing cavities in the rock and the shear zones were in such a bad state that the thickest mix could not fill them even with the addition of sand. Also the presence of groundwater in the West abutment had an adverse effect on the

7. PROBLEMS OF SPECIAL INTEREST

By the end of the construction period in 1967 two severe problems were encountered.

The first one was a rather excessive leakage mainly through the Western abutment of the dam and the second was the development of a large crack also on the Western abutment.

a. Leakage

The problem of leakage was realized before construction with the presence of three springs near the West abutment. The minimum flow of these springs before construction was 11 l/s and reached a maximum of 40 l/s when the reservoir filled with water in the Winter of 1967. The increase was relative to the water level and it was noted that at 7 m below full reservoir level the flow of the spring became 18 l/s.

In addition to the flow of these springs a major spring appeared on the East abutment which flowed only when water was impounded in the reservoir and which reached 30 l/s during the winter of 1967 when the reservoir was filled with water. Furthermore leakage became apparent through the river bed itself, which possibly found its way through the fault zone. The flow varied from 50 l/s at maximum reservoir water level to 15 l/s at 7 m below full reservoir level.

In an effort to see whether the seepage of the West abutment could be reduced by further grouting works, considerable investigations were carried out through drilling, coring, water testing and test grouting. From the cores recovered, it became evident that the rock was broken in most parts and shear zones exist right down to a depth of 70 m from the crest.

The permeability of the rock in lugeons varied from 4 to 106. Due to the extensive shear zones it appears that better rock can be found only in considerable depths below what can be grouted economically and it therefore seemed improbable that any further grouting works could be justified.

Thus a weir was built just downstream of the dam to collect the seepage along the main river channel which was then diverted in the irrigation pipeline. The three springs of the West abutment could not be collected in the weir as they are at a lower elevation.

b. Slides

During December 1966, immediately after a heavy rainfall when the reservoir was filled with water, small cracks started appearing on the West abutment at about 0.3 km away from the dam and at an elevation of 200 m higher. By March 1967 this crack reached 0.60 m vertical displacement whilst at the same time many minor subsidiary cracks developed. A detailed examination followed which brought to light that the rock is composed of a series of parallel dykes and that there are distinctly one intact and one detached unit. The detached unit appears to be the result of a gravity slide. The cracks were first observed in the talus immediately after the first impoundment. The major crack can be easily traced for about 650 m without any breaks in its continuity with a maximum vertical movement of about 1 m. A number of trenches were excavated across the major fault which revealed that the hard diabase was overlain by 0.15—0.30 m of fault breccia made up of angular fragments indicating movements under high pressure. A clay slip surface with slickensides was met at the junction of the fault breccia with the talus which indicates movement in the direction of the dip, with clear indication that the talus moved in respect to the underlying diabase by sliding on the clay slip surface. The question arose whether the movement which occured was associated with the

Drilling and grouting on the West abutment. Weathered and sheeted diabase rock can be seen.

Crack on the overburden overlying the diabase of the Western flank.

talus alone or whether it also involved a portion of the detached unit.

In favour of the first view that the talus moved alone the following arguments have been put forward:

i. The excavations carried out on the West abutment of the reservoir are small compaired with the mountain mass and it would be highly unikely that this excavation could disturb the equilibrium of the sheets.

ii. The flat surfaces of the mountain mass should not be attributed to sliding but to old river erosional terraces.

iii. The dips are generally conformable and are in the same direction and equal to those of the supposed slided block.

iv. It was considered that the shear zones are not deep and are wedging out.

v. It is difficult to explain the accommodation of the movement of the rock mass in the fault zone under the dam because it appears that the direction of the dip and the strike of the fault is the same as that of the sheets, in which case the accommodation of the movement cannot be explained.

The arguments in favour of the view that the rock mass has moved and the talus movement is due to this are:

i. The remarkable continuity of the dominant crack.

ii. The movement first started at the top.

iii. There is a strong correlation between the levels of both reservoir and rainfall with the history of movements.

iv. All movement is mainly in the direction of the underlying sheeting plane.

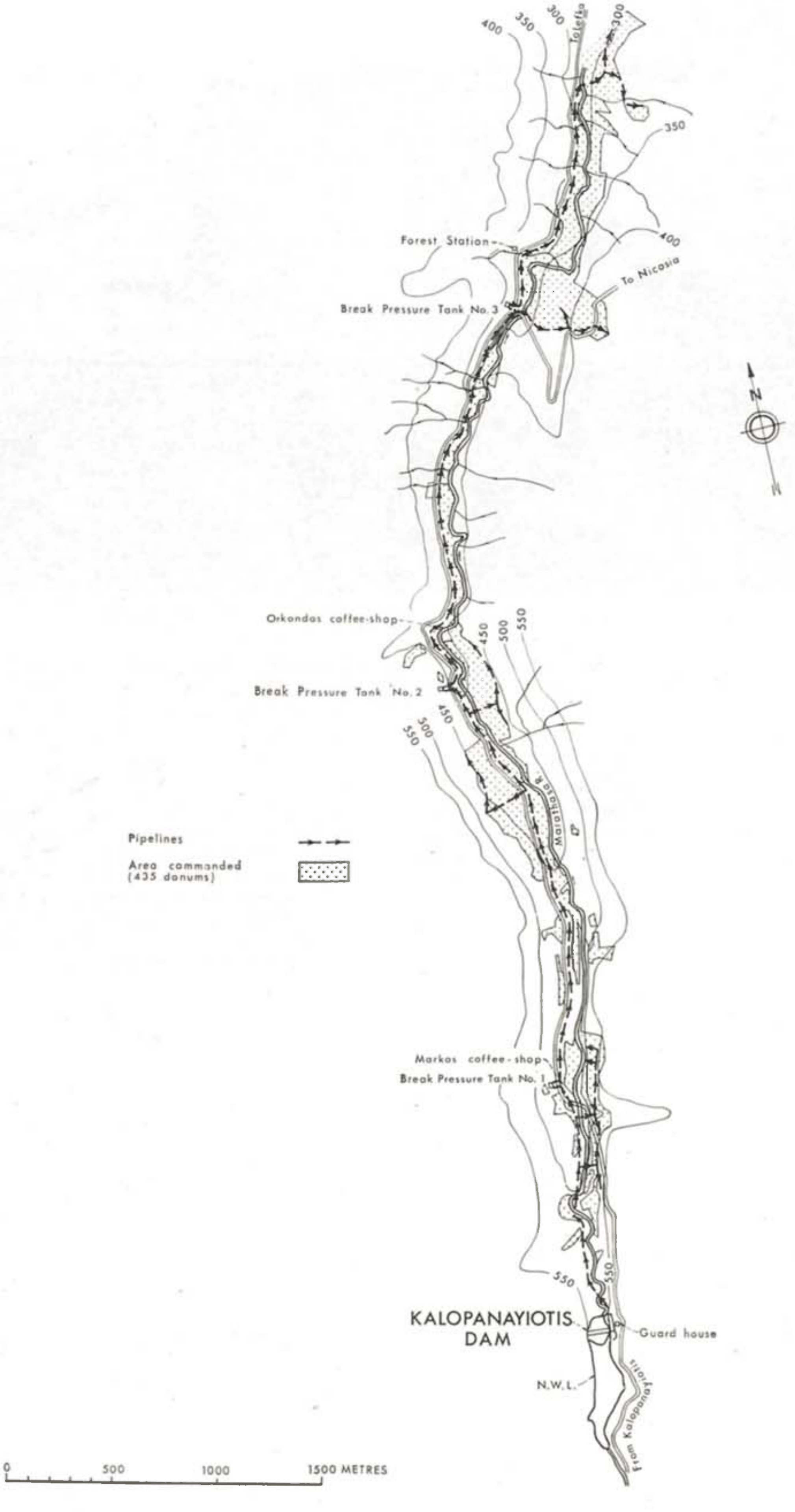
v. The physiographic features of the hillside and the presence of the fault breccia beneath the detached unit reveal a pre-existing zone of considerable weakness.

Although it was concluded that the excavations on the West abutment could not have triggered slides of such magnitude yet the reservoir impounding itself changed the groundwater regime of the detached unit and in general induced an increase in the pore pressures for a given amount of infiltration. Therefore the effect of impounding would decrease the factor of safety of the mountain mass by causing the groundwater to back up within the mass. Such a situation is more emphasized under heavy rainfall conditions which would cause uplift pressures in the rock mass. The fault zone under the dam is considered to be compressible enough to have accommodated the movement without any surface features to appear.

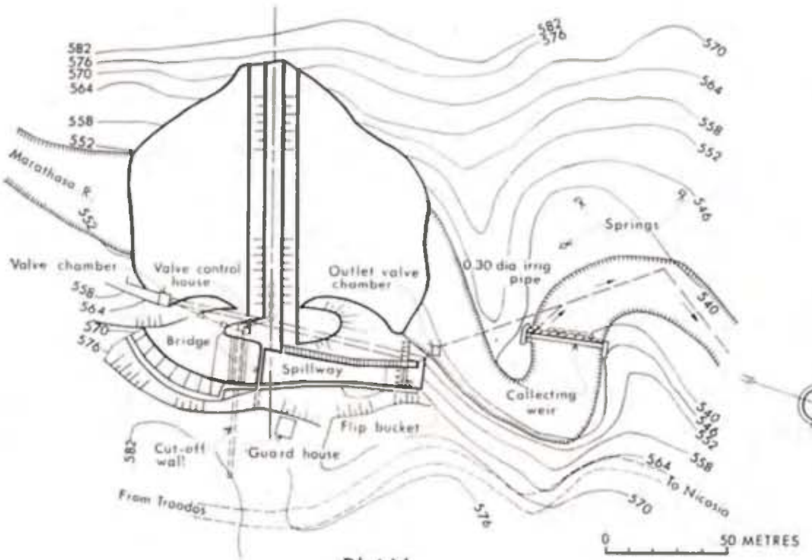
It was decided that as a protective measure, the reservoir should be kept empty during winter rainfall conditions so that the drainage of the mountain mass would occur at river bed level. This situation gives a more stable condition on the mountain. Since the Marathasa river flow is far in excess of the storage requirements, leaving the reservoir empty, is not only an advantage from the abutment stability point of view but also from the desilting point.

A series of monuments were set up in order to measure the movements both of the rock and of the talus material. Nine monuments were also set up on the dam and on the East abutment. Of these 9 monuments three were fixed on the crest, three on the

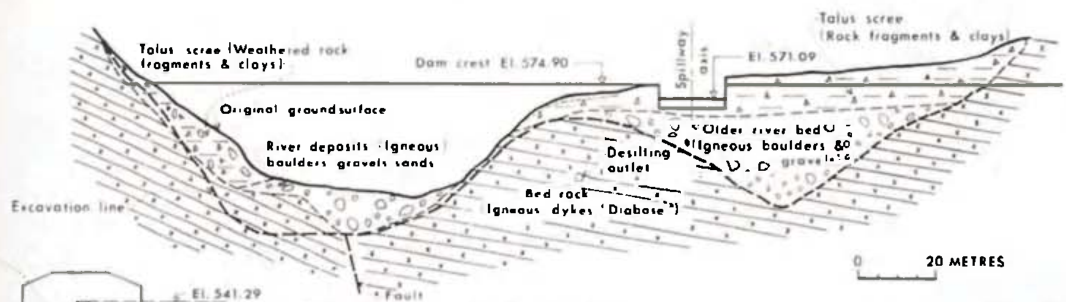
MARATHASA-KALOPANAYIOTIS-DISTRIBUTION SYSTEM



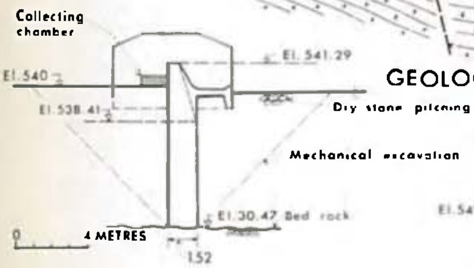
MARATHASA-KALOPANAYIOTIS DAM



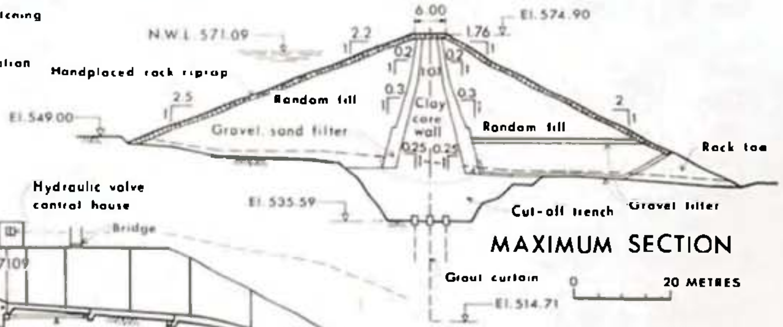
PLAN



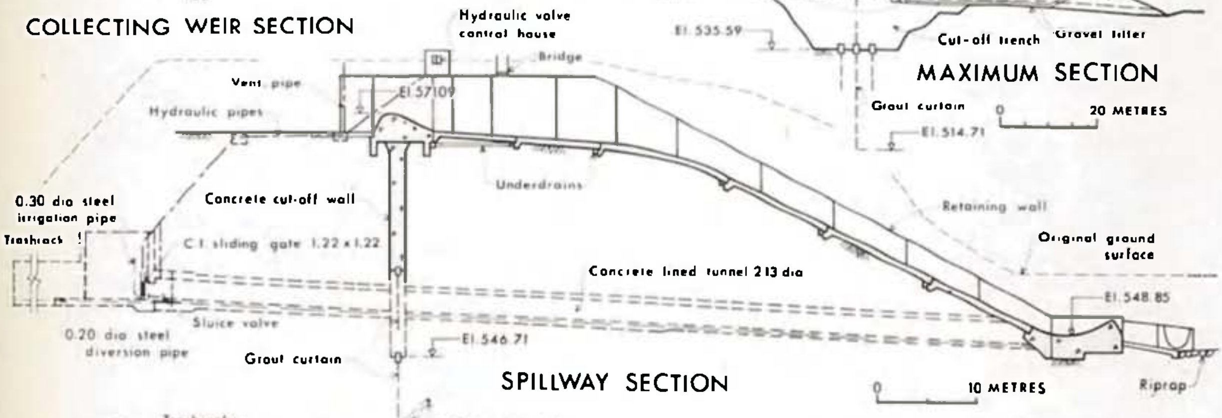
GEOLOGICAL SECTION ON DAM AXIS



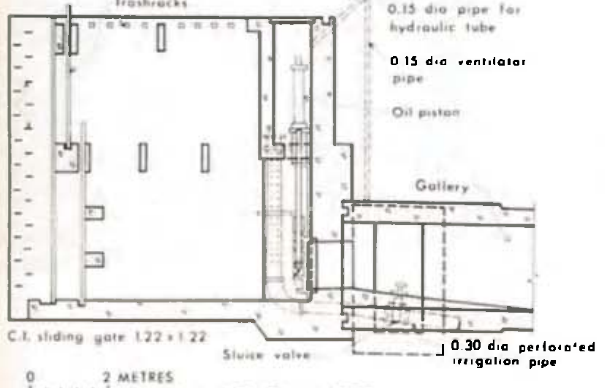
COLLECTING WEIR SECTION



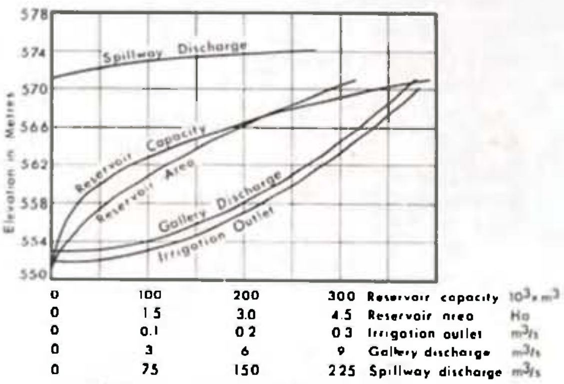
MAXIMUM SECTION



SPILLWAY SECTION



VALVE CHAMBER



CAPACITY-AREA-DISCHARGE CURVES

downstream berm of the dam and three on the East abutment. Measurements taken since 1966 indicate that the region of the dam near the West abutment has mainly an upward vertical displacement of up to 57 mm whilst the region near the East abutment has a downward displacement of up to 9 mm.

Regarding the horizontal displacements, the monuments show that the region near the crest has mostly a displacement towards the reservoir of 35 mm maximum, whilst the downstream region of the dam has mostly a displacement away from the reservoir of 170 mm maximum. Although the historic records show a relationship of these movements with impoundment and rainfall, yet it is not possible to determine exactly whether the displacements of the dambody bear any correlation with the mountain mass movements on the West abutment.

Regarding the movements of the mountain mass itself a total of 16 stations were set up on talus and on rock with movement observations taken both for horizontal and vertical directions. Observations taken from 1967 onwards indicate the following:-

All stations are moving both on talus and on rock.

The order of magnitude of the movements is the same for both talus and rock stations.

From 1967 to 1971, the movements vary between 0.13 to 0.3 m the maximum having occurred on stations nearest to the reservoir.

The movements are progressively reducing since 1967, and in fact they have been very little since 1969. The maximum movement occurred immediately after heavy rainfall followed up the first impoundment early in 1967. Heavy rainfall is associated with the movements of 1969 also.

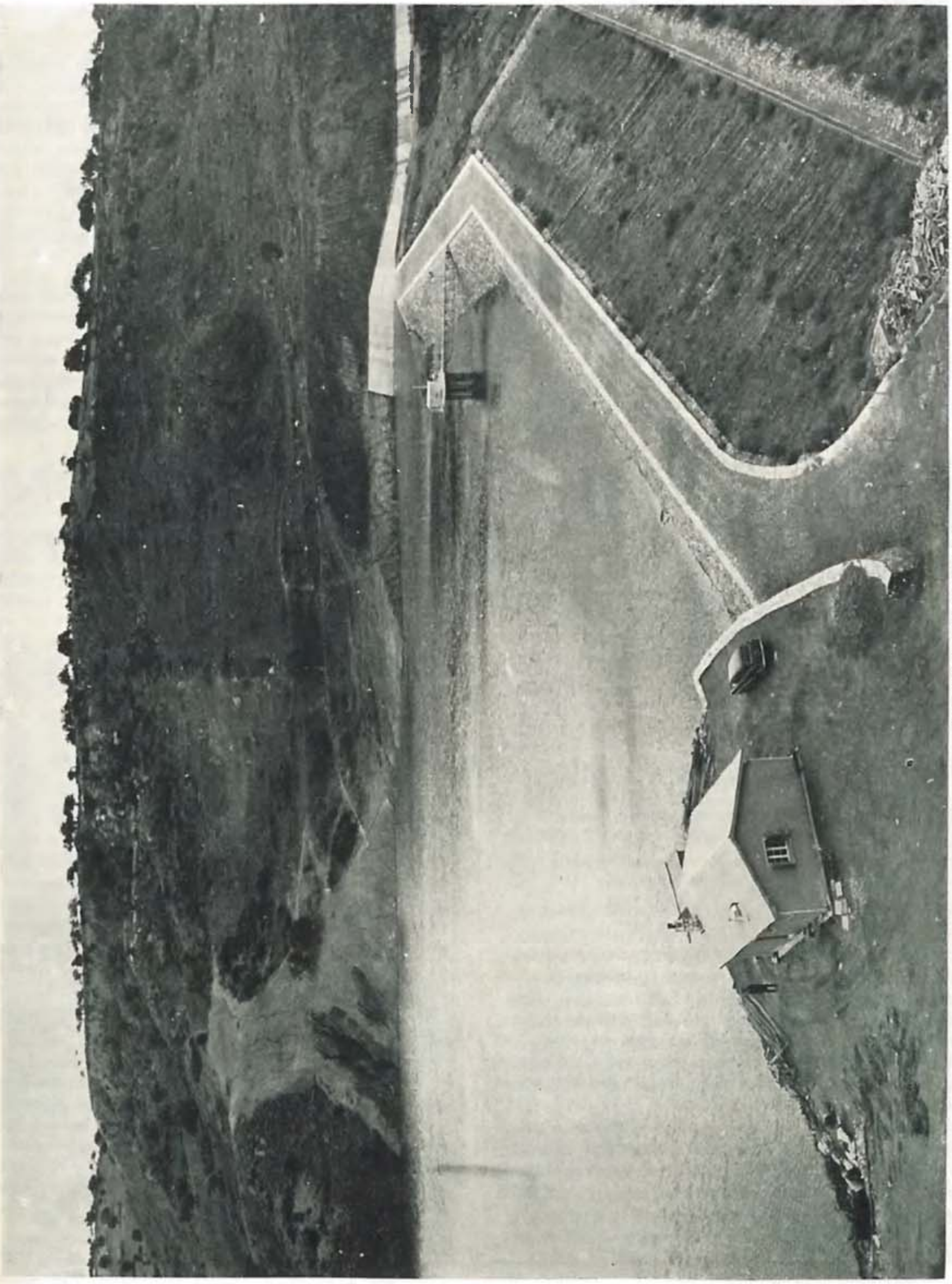
The resultant direction of the movements appears to be generally at right angles to the reservoir and for those stations downstream of the dam at right angles to the river bed.

From the up to date observations it appears that movement of the rock has taken place. However, whether this movement has now stabilized is not certain for although there has been very little movement since 1969, the fact is that very little rainfall has occurred since then. Therefore, more observations have still to be taken especially under more adverse weather conditions before we can conclude definitely about the type of movements taking place, their extent and their effect on the safety of the reservoir.

Data

i.	*Catchment*	
	Area	26 km²
	Average rainfall	900 mm/a
	Average runoff	11.6 million m³/a
	1/1000 years flood	173 m³/s
	Maximum height	1640 m
	Maximum length	7,260 m
ii.	*Reservoir*	
	Area	4.7 ha
	Capacity	0.39 million m³
	Live storage	0.38 million m³
	Length	555 m
iii.	*Embankment*	
	Structural height	39.31 m
	Height above ground level	25.90 m
	Hydraulic height	22.09 m
	Depth of foundation cut-off	13.41 m
	Freeboard	3.81 m
	Crest length	137 m
	Top thickness	6 m
	Base thickness	128 m
	Upstream slope	1:2.5, 1:2.2
	Downstream slope	1:2, 1:1.76
	Upstream core slope	1:0.25, 1:0.1
	Downstream core slope	1:0.25, 1:0.1
	Minimum core thickness	3.04 m
	Maximum core thickness	14.5 m
	Random fill	104,160 m³
	Clay core	31,179 m³
	Rock toe	4,050 m³
	Gravel, sand filter	8,100 m³
	Horizontal gravel filter	2,674 m³
	Rock rip-rap	6,037 m³
	Total earth fill	156,200 m³
iv.	*Excavations*	
	Total	47,000 m³
v.	*Foundation Treatment*	
	Total drilling depth for grouting	3,146 m
	Total grout take	557,800 kg
	Average grout take	177 kg/m
vi.	*Spillway*	
	Size	18.28x3.80 m
	Capacity	204 m³/s
	Length	77.70 m
	Concrete	2,100 m³
vii.	*Gallery*	
	Size	2.13 m dia
	Capacity	11.3 m³/s
	Length	88.4 m
	Concrete of outlet works	870 m³
	Operating gate size	1.21x1.21 m
viii.	*Outlet (Steel pipes)*	
	Size	0.30 m dia
	Capacity	0.38 m³/s
	Length	113 m
viii.	*Distribution System*	
(i)	*Main conveyor (steel pipes)*	
	Size	0.20 to 0.30 m dia
	Capacity	0.105 to 0.39 m³/s
	Length	7,120 m
(ii)	*Distribution network (steel pipes)*	
	Size	0.07 to 0.15 m dia
	Capacity	0.009 to 0.021 m³/s
	Length	13,170 m

MAVROKOLYMBOS



The Mavrokolymbos dam.

MAVROKOLYMBOS—MAVROKOLYMBOS PROJECT

1. PURPOSE

The purpose for constructing this dam was to supply enough water for the irrigation of early summer crops for the lands of the villages of Khlorakas, Emba and Kissonarga, as well as to the Government farms at Potima. Early summer crops grown in this region since a number of years are predominantly cucumbers and tomatoes which are grown under glass or polythene cover so that maturing is achieved in February. The profits from these crops at this time of the year are very high and in many cases the farmers of the region develop lands by transporting suitable soils and laying it over rocky ground. Water was extracted from shallow wells in a calcareous sandstone aquifer the yield of which is rather poor and could not meet the demands for irrigation. Further more, there are big stretches of good land in the neighbourhood of the 4 villages in the coastal region which could easily be cultivated and produce early vegetables if water was to be made available. Therefore a decision was taken to build a dam on the Mavrokolymbos river which lies at the extreme North boundary of the Project area and from which the water could easily gravitate to all lands in the region. This project is part of the overall plan for the development of the SW Paphos coastal region between the Ayios Yeoryios cape at Peyia up to the Khapotami river involving the building of more dams and groundwater extraction in the larger rivers of the region.

2. LOCATION

The dam has been built at a distance of about 2.5 km upstream of the mouth of the river and at an elevation of about 70 m asl.

3. PLAN

a. Water and Land

The flow of the Mavrokolymbos river is estimated to average 3.9 million m^3/a with a maximum flood at a frequency of 1/1000 years of 227 m^3/s.

Irrigation rights on this river belong to the Government owned Potima farm of an area of about 700 donums situated downstream of the damsite and receiving water from an intake and a canal constructed just downstream of the dam.

The total area of land commanded by the project between Potima and Khlorakas village is of the order of 3,355 donums of good soil most of which is suitable for the cultivation of all types of crops. Very little of this land was previously irrigated.

b. Geology

The Mavrokolympos dam and reservoir lie in the so called Mamonia Formation of the Igneous Complex. This Formation consists of diabase cherts, radiolirites. sandstone clay stones, and shales, marls, limestones, pillow lavas and serpentines. The essential feature of these rocks is that they are all extraordinary folded and fractured so that everything is so overturned and interwoven that it must be treated as a unit whose lithological members cannot be separated owing to very frequent lithological changes both vertical and lateral. This complex resembles the diabase chert series found all the way from Yugoslavia to Greece and Turkey. It is considered that the rocks of the diabase chert were formed in the deepened part of the dinario geosynclinc where frequent undersea intrusion eruptions and outflows of ultrabasic lavas occurred during sedimentation process whereas the material was variously alternated and interwound. Also it is considered that cold intrusion of serpentine masses occurred through overlying rocks along fractures and faults.

Owing to the extensive disturbances of rocks and relatively high degree of saturation, slides often occur similar to those in clays. Serpentine masses within those formations are generally very strongly fractured but not completely crushed although some times when they are completely crushed they turn into serpentine talc schist, but only in places where they are subject to extraordinarily strong tectonic strains.

In general the dam is founded on a sound serpentine rock mass. The serpentine mass where the dam is built is embraced by sedimentary members of the Mamonia series and small masses of pillow lavas which are probably under-sea outflows of basic lavas which occurred in the course of sedimentation. The only well defined tectonic contact of the serpentine mass and the Mamonia sediment series is a local fault whose strike generally corresponds to the strike of the system of faults in the Troodos magna complex. Along this fault the serpentine is not damaged any more than it generally is in the area. Generally however, the rock is fractured in an irregular way but such damages are not inter connected and have different strikes. It may be concluded that the serpentine mass as a whole is sufficiently bounded and stable enough for the construction of a dam 30—40 m high. The rock proved to be hard enough during excavation including the tunnel which was left unlined without any supports for one year.

The geology of the storage reservoir itself is made up of the sedimentary series of the Mamonia complex predominantly composed of thin bedded and schistous clays, sand stones, cherts and conglomerates. There are old slides in the higher levels of the reservoir and large blocks of pliocene sandstones can be seen lying almost horizontally over the schists of the Mamonia series.

The soil is in general fairly saturated with water and some pools of water are formed in depressions with some springs appearing along the fault between the serpentines and the schisty agrillaceous series even at fairly high levels above the storage reservoir.

An extensive exploratory work was carried out before construction, including drilling and laboratory testing to determine the best dam site and the location for the tunnel and spillway.

The rock along the cut-off was found sufficiently impermeable not to warrant any grouting works.

4. MAIN FEATURES

The detailed design of the dam was produced by EnergoProjekt of Yugoslavia, whilst that of the distribution system by TechnoExporstroy of Bulgaria.

a. Dam

The embankment was formed of random fill upstream and downstream of a clay core. Between the random fill and the clay core there are filter transition zones. On the upstream face, rock rip-rap has been provided whilst downstream the random fill is exposed with two berms of rip-rap along the slope.

A 2.74 m dia scouring tunnel has been built through the left bank operated through a shaft by a wheel gate 3.96x2.13 m. A steel irrigation outlet pipe 0.38 m dia is provided through the tunnel operated by a valve at the downstream. The spillway weir made of mass concrete on the left abutment is of the overflow type.

The lateral walls on the chute channel made of reinforced concrete are partly of a slope of 1:2 and partly vertical ending to a bucket structure at the river bed made of reinforced concrete. The spillway design was based on a hydraulic model set up in Beograd Yugoslavia.

b. Distribution System

The main conveyor is of rectangular reinforced concrete lining whilst the network is made up of asbestos cement pressure pipes supplying water in most areas by pressure.

5. CONSTRUCTION

The work for the dam was carried out by the Contractors CYBARCO of Nicosia between June 1964 and November 1967.

Work started first on the tunnel. For most of its length the serpentine rock was not sound enough and it had to be supported by steel frames although at intervals it was hard enough to require blasting. Near the exit the rock was clay and 16 m of overlying clay collapsed. Several other minor falls occurred before final cutting through.

The concreting of the tunnel and shaft was done mostly by compressed air pump.

Grouting of the tunnel was carried out at 3 m spaced sections, each section having 4 holes of 0.67 m depth. Grouting was first done at the lower boreholes of the section by cement - water mixture of cement - water ratio of 1:3 at the beginning and thickening to 1:0.7. The maximum pressure used was 2 atmospheres. The excavations carried out upstream of the tunnel entrance caused slides in this region which necessitated the extension of the lined tunnel upstream to a safer position of entry.

A 0.38 m dia irrigation outlet pipeline was laid welded and concrete covered in the lower part of the tunnel. This pipe is operated by two hand slide valves placed at the downstream end.

To enable construction of the embankment a cofferdam was built to divert the river flow into the tunnel. For the cofferdam, fine random fill from a nearby borrow pit was used.

The excavation of the cut-off went through good sound serpentine rock and it was decided that grouting was not required.

The clay used for the core was transported from borrow areas near the coast and was compacted by sheepfoot roller with 10 passes and 0.20 m maximum thickness of layers before compaction. At optimum moisture content of 18% the optimum dry density of this material is 1,690 kg/m^3. The random fill obtained from borrow pits in the reservoir area and from spillway excavations was compacted by sheepfoot roller and pneumatic tyred roller with 6 passes at optimum moisture contents 10—13% and optimum dry density of 1,850—1,930 kg/m^3. Upstream and downstream of the core, transition filter zones of gravel from borrow areas located along the coast were compacted using vibration rollers with 6 passes and maximum thickness of layers 0.40 m before compaction.

Handplaced rip-rap protection was laid on the upstream face followed by a gravel filter made up of 0.3 m fine gravel placed immediately next to the random fill. The boulders for the rip-rap were selected from the river bed both upstream and downstream of the dam.

Materials for concrete aggregate were obtained from the coast at the mouth of the Mavrokolymbos river.

A rubble curb was built on the upstream side of the dam crest

Concrete drainage channels at the toe and around the dambody on the downstream face, were constructed.

Settlement monuments on the crest were fixed for settlement observations.

Instrumentation made by Galileo of Italy for measurements of pore and total pressure were placed in two sections as follows:—

At river bed level six pore pressure and five total pressure cells. At the same section at 13 m higher two pore pressure cells. At the second section on the right abutment and at an elevation of 13 m above river bed level, six pore pressure and three total pressure cells, whilst in the same section and at 12 m higher two more pore pressure cells were fixed. Thus a total of 16 electroacoustic manometers for pore pressure measurements and 8 tensiometric cells for total pressure measurements were used.

All cables connected with the instruments were placed through steel pipes leading up to the permanent guard house where they are connected to a control box, thus enabling the measurements to be taken from the house.

Piezometric observations on the right abutment.

The problem of locating the spillway, on a sound foundation was very important to avoid slides, whilst proper protective works had to be made. After excavation and trimming for the spillway concreting, gravel was spread over the surface in the places where clay foundation existed for drainage purposes. In the gravel blanket, perforated asbestos cement pipes 0.25 m dia were laid to facilitate drainage. Also, on the side adjoining the hillside on the top of the retaining wall and throughout its length a drainage trench was excavated lined in concrete to drain the surface runoff away from the spillway leading it into a manhole from which a 0.45 m dia pipe conveys the water to river bed.

All surfaces of construction joints on the spillway were coated with two coats of 'flinkote'. Also rubber water stops were used in most of the joints protected by a layer of asphalt.

The main part of the hydromechanical equipment, that is the gate, the gate frame, the lifting mechanism, and aeration pipes were constructed and supplied by the firm Drakos—Polemis of Greece. The steel transition lining, guides, trash racks, bends, steps and other minor steel components were made at the WDD workshop. The hydromechanical equipment was erected by the Contractor.

Because of the marginal stability of the Mamonia clayee soils and excessive excavation of the tunnel and spillway structures, landslides occurred which necessitated the carrying out of a number of remedial works and the setting up of an observation network as is described under "Problems of Special Interest". All these remedial works were completed by November 1967.

All works on the dam including £8,000 cost of the landslide remedial measures was £335,000.

The distribution system composed of a main conveyor channel from the dam and piped distribution network started early in 1966 by the Department and was completed by the end of 1970.

The total cost of the distribution system reached £135,000 whereas some tertiary distribution pipes to the value of £30,000 has still to be laid awaiting the finalization of land consolidation in the region.

Thus the total cost for dam and distribution is £500,000.

6. MANAGEMENT

A Project Water Committee made up of the District Officer as Chairman, the Regional Engineer of the WDD and the Regional Agricultural Officer, is responsible for the management of the irrigation works. The maintenance works are under the direct responsibility of the WDD. Water is sold to each farmer volumetrically through a water meter at the rate of 10 mils/m^3. The main crops planted are early vegetables such as cucumbers and tomatoes, although a number of banana plantations are also grown especially in the Mavrokolymbos valley downstream of the dam. The farmers come from the villages of Khlorakas, Emba and Kissonerga owning their own land or renting land at the Government Potima farm in the Mavrokolymbos valley.

7. PROBLEMS OF SPECIAL INTEREST

The most interesting problem which became apparent during construction and immediately after, is the instability of the Mamonia Formation. Two problems were of concern, the first as regards certain zones of weakness at the damsite itself and the second as regards the instability of the natural slopes in the reservoir which was aggravated by the excavation for fill material.

Investigations made of the problems that appeared after the first impoundment, resulted to a series of remedial works to be carried out and observation stations to be established.

Regarding the first problem at the immediate vicinity of the damsite itself, it was concluded that the dam was resting on a sufficiently large block of serpentine continuous from the one abutment to the other. The presence of shear planes on the serpentine with talc and clay of low frictional resistance are sources of weakness. The most critical alignment of the thrust shears with respect to the dam is when their strike is parallel to the centreline of the dam. In this condition instability would occur if the shear planes were horizontal. Fortunately, however, the shear zones dip upstream at inclinations of over 25° and the talcy surfaces to an average inclination of 40° which give considerable extra resistance to sliding.

The second problem was the instability of the reservoir sides and the slides which were triggered by heavy rainfall in February 1966 and undercutting of the toes for fill material. The examination revealed that at the right bank of the reservoir, old landslide areas exist.

Sample tests showed liquid limits as high as 90%. The residual strength parameters of the clay were determined by direct shear strength tests carried out on block samples containing slip surfaces. The conclusion from these tests is that the residual cohesion intersept should be taken as zero and the residual angle of shearing resistance as 16°.

Also for the purpose of studying the water level movement and its relationship with landsliding, a series of piezometric and surface displacement measurements were taken during the winter 1967—1968.

The conclusions of the study were:—

That the old slides on the right abutment are the results of weathering of the underlying shales, and that these slides were reactivated by the oversteepening of their toe by under excavation during construction and by water pressure due to the rainfall of 1965—1966. The natural slopes of the ground being 10° to 16° give a factor of safety against sliding of 1.2 which would decrease to 1 under wet conditions. So with the undercutting of the toes and an increase in the moisture content, the surfaces become very unstable. Although such landslides within the reservoir do not alter the capacity, yet the possibility of triggering more landslides from outside the reservoir was estimated that it might result to about 340,000 m³ or 15% reduction of the reservoir capacity.

Although a major remedial works programme was proposed, it was finally decided to go on by stages after due consideration of the results being taken from

Reservoir slide area on the right abutment.

piezometric and displacement instruments.

As immediate measures, however, the works carried out were:—

The tunnel entrance was extended upstream to be saved from blocking by landslip materials. On the left abutment where the spillway is situated, the surface was trimmed to a flatter slope, and planted with acacia trees and a peripheral diversion channel was built on the top conveying rainwater into the reservoir above the landslide areas.

On the right abutment extensive smoothing and flattening of the abutment near the dam was made and diversion channels were built above these surfaces taking rainwater into the reservoir clear of the landslide surfaces. Also at the toe of the right abutment, in the reservoir, a drainage embankment was built to facilitate drainage.

At the same time the programme of observations continues both on the dam and in the reservoir area, so that if necessary additional remedial measures would be carried out. For the dam, movement monuments on the crest have been placed by which the horizontal and vertical movements are determined, and for the dambody tensiometric cells for pore water and total pressure observations.

For the reservoir area, movement monuments have been placed for measurements of the horizontal and vertical displacements and piezometers for measuring the water table levels.

The conclusions of the various observations are as follows:—

The dam crest movement monuments showed that a slow settlement started just after the dam was completed in 1967 with a maximum sudden drop in the winter of 1969 just after the first impoundment. The slow settlement continued thereafter being 0.27 m total by the end of 1971. Similarly the horizontal movements showed a maximum upstream displacement of 0.150 m in the winter of 1969 with the first impoundment. Thereafter the total displacement was reduced to 0.100 m and remained more or less steady.

For the porewater and total pressure on the foundations and dambody the porewater and total pressure cells indicated the following:—

Regarding the total pressure cells, three of them have not functioned at all whilst the remaining 13 have shown a pressure relative to the overburden above. As the overburden during construction increased, the pressure increased until full height was reached when the pressure was stabilized. When impoundment of water occurred in January 1969 the pressures were affected downwards for a few days but were then back to more or less their original pressures before impoundment.

Regarding the porewater pressure cells, one of them stopped functioning as soon as the water table reached it. Of the two in the foundations just downstream of the core, the one showed no change at all with impoundment, whereas the other one's pressure increased a few months after impoundment to about 25% of the hydrostatic head. The cells placed upstream of the core showed an increase within only a few days after water filling started which reached full hydrostatic head and followed closely the reservoir level thereafter. The cells placed downstream of the core showed from no increase up to maximum of 25% of the hydrostatic head having reached this maximum value at about one and a half years later.

Regarding the reservoir area, five lines of a total of 29 observation stations were placed parallel to the length of the reservoir.

The three nearest lines to the reservoir showed movements only away from reservoir, whereas the two most distant lines showed movements both away and towards the reservoir. The maximum displacement towards the reservoir reached 1.3 m in winter 1969 just after impoundment and thereafter it remained constant. Similarly the maximum displacement away from the reservoir reached 0.3 m by the end of 1969 having stabilized since then.

Finally, three lines made up of 24 piezometers placed at right angles to the reservoir were under observation. Their water levels showed increase in relation to the water level in the reservoir and some of them have shown variations in level of up to 5 m. It has been concluded that the water table in the mountain side has moved relative to the reservoir level.

Data

i. Catchment

Area	37.84 km^2
Average rainfall	624 mm/a
Average runoff	3.9 million m^3/a
1/1000 years flood	227 m^3/s
Maximum height	530 m
Maximum length	12,450 m

ii. Reservoir

Area	17.5 ha
Capacity	2.18 million m^3
Live storage	2 million m^3
Length	945 m

iii. Embankment

Structural height	45.10 m
Height above ground level	35.40 m
Hydraulic height	31.14 m
Depth of foundation cut-off	9.70 m
Freeboard	4.26 m
Crest length	183 m
Top thickness	6 m
Base thickness	174.33 m
Upstream slope	1:3, 1:2.5, 1:2
Downstream slope	1:1.75, 1:1.84
Upstream core slope	1:0.5
Downstream core slope	1:0.4
Minimum core thickness	3.04
Maximum core thickness	7.39 m
Random fill	202,000 m^3
Clay core	22,000 m^3
Filter zones	67,000 m^3
Rock rip-rap, toe, curbs	11,000 m^3
Total earth fill	302,000 m^3

iv. Excavations

Total	161,000 m^3

v. Spillway

Size	35.34x5.18 m
Capacity	366 m^3/s
Length	284 m
Concrete	7,000 m^3

vii. Gallery

Size	2.74 m dia
Capacity	68 m^3/s
Length	235 m
Concrete	2,200 m^3
Operating gate size	3.96x2.13 m

vii. Access Shaft

Size	3.55 m dia
Depth	36 m
Concrete	200 m^3

viii. Outlet (steel pipes)

Size	0.38 m dia
Capacity	0.70 m^3/s
Length	270 m

ix. Distribution System

(i) Main Conveyor (RC channels)

Sizes	1.11x0.71, 1.06x0.71 9.90x0.60, 9.90x0.81 m
Capacity	0.28, 0.39, 0.42 m^3/s
Length	4,605 m

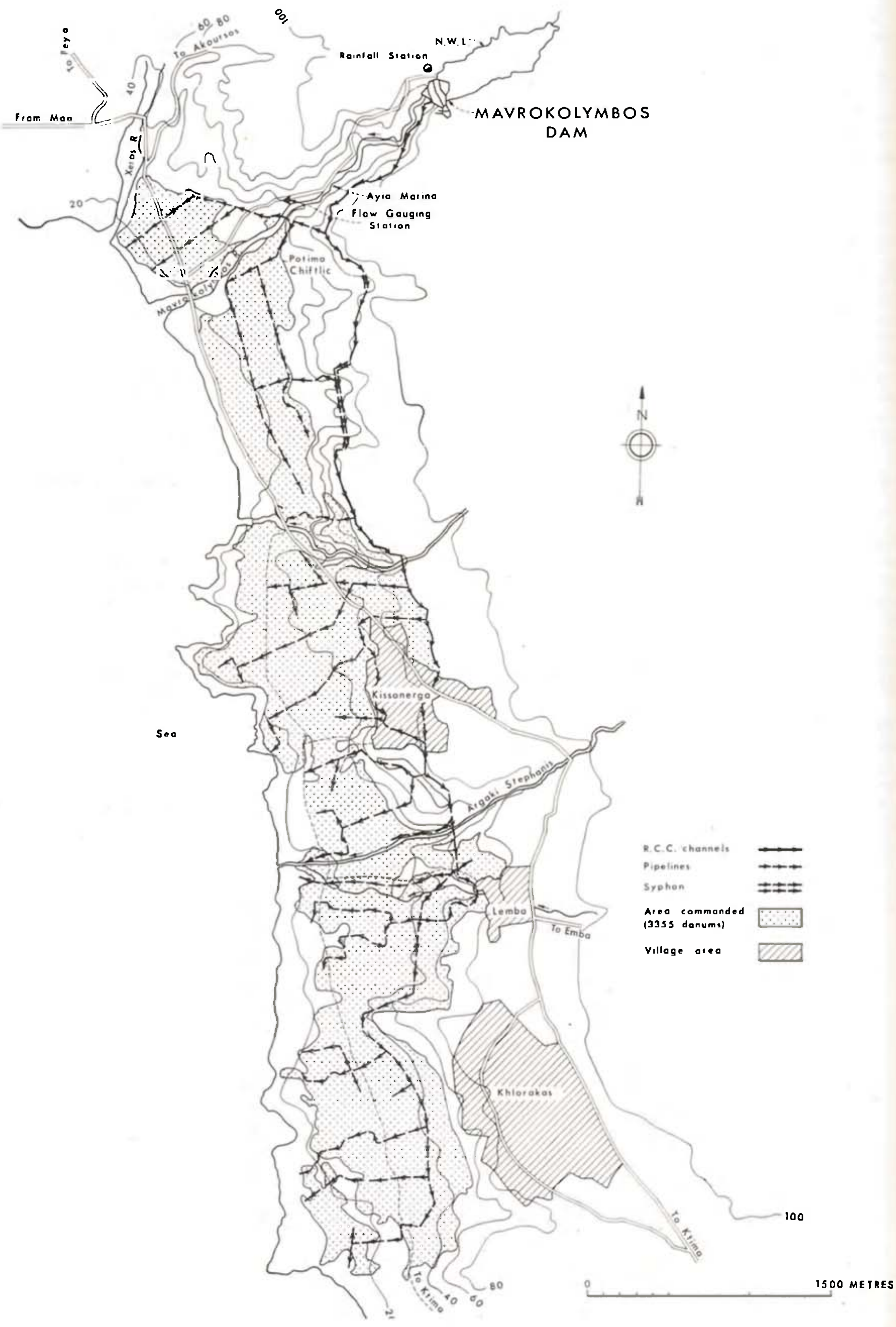
Piped Conveyors

Size	0.2 — 0.6 m dia
Capacity	0.22 m^3/s
Length	3,000 m

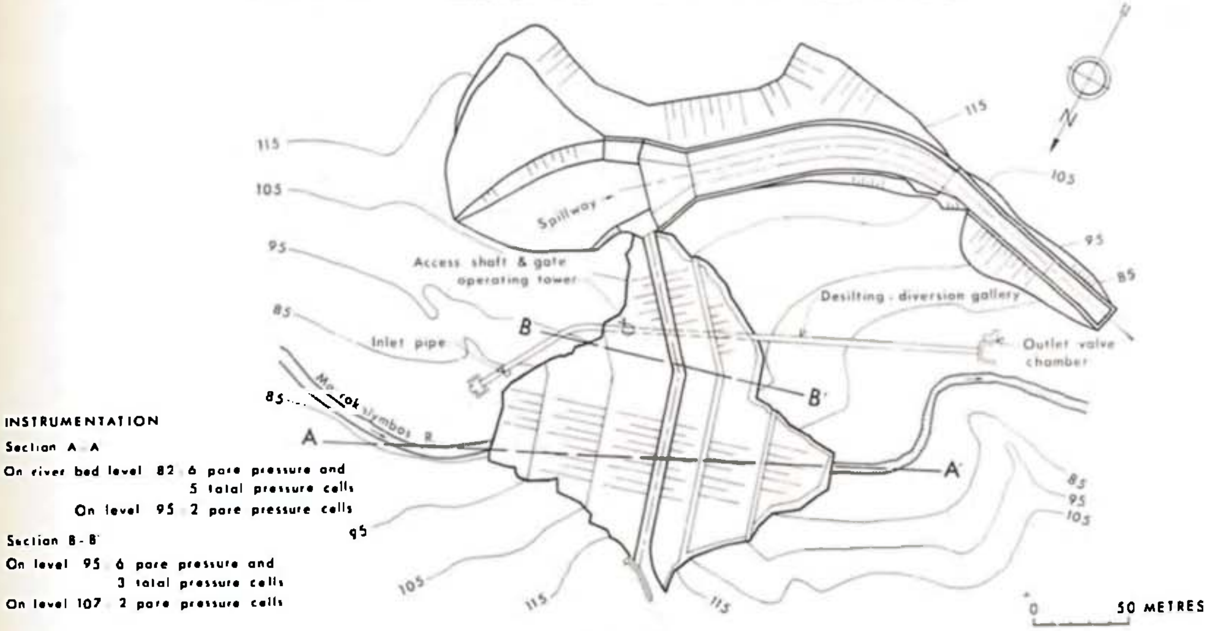
(ii) Distribution Network
(Pipes A C classes B and C)

Size	0.10 — 0.30 m
Capacity	0.14 m^3/s
Length	17,500 m

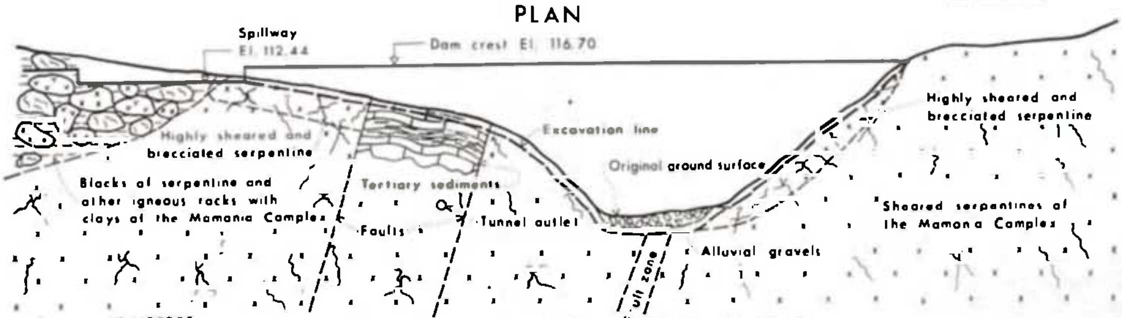
MAVROKOLYMBOS - MAVROKOLYMBOS-DISTRIBUTION SYSTEM



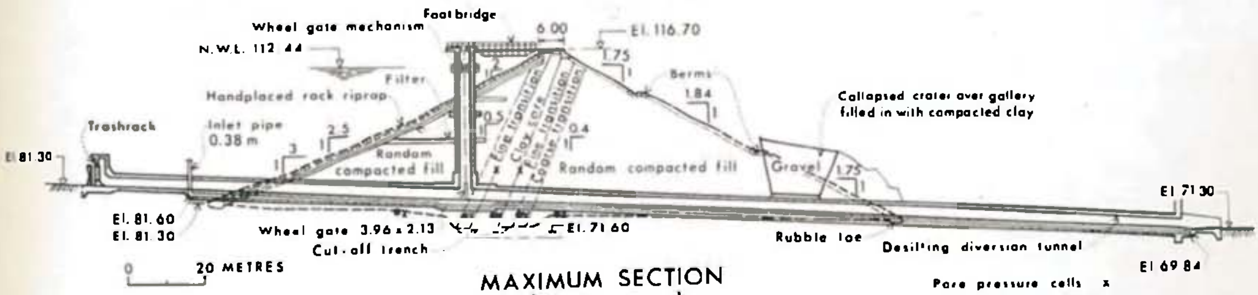
MAVROKOLYMBOS - MAVROKOLYMBOS DAM



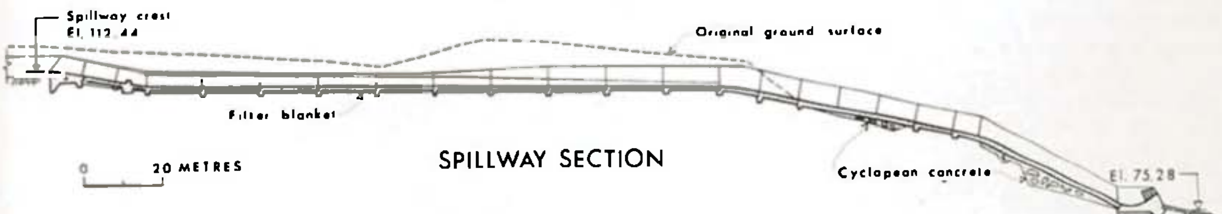
PLAN



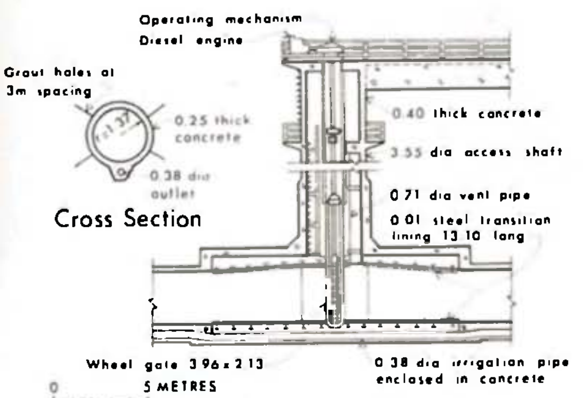
GEOLOGICAL SECTION ON DAM AXIS



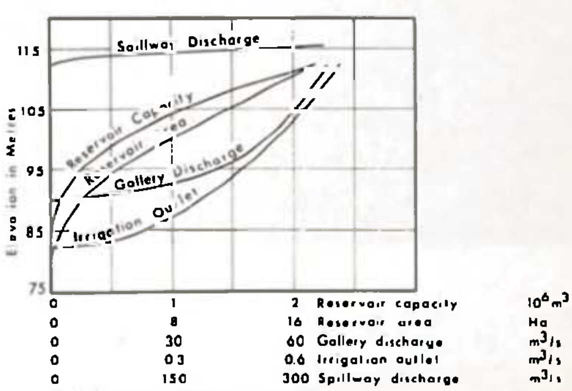
MAXIMUM SECTION
(Section A-A)



SPILLWAY SECTION

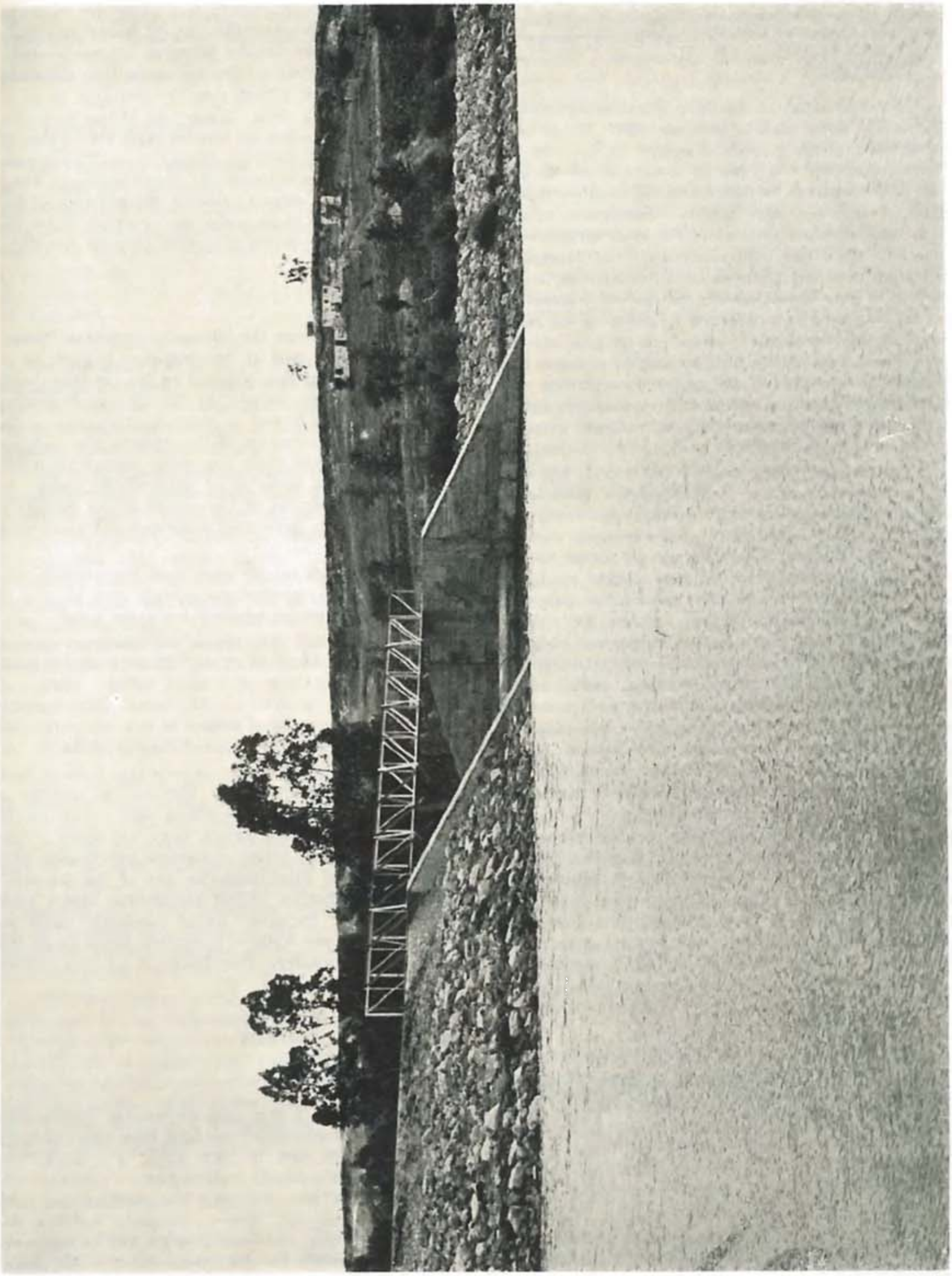


GALLERY - SHAFT - WHEEL GATE



CAPACITY-AREA-DISCHARGE CURVES

SYNGRASI



The Syngrasi dam.

MERIKEROS — SYNGRASI PROJECT

1. PURPOSE

This dam is one of the three oldest constructed in Cyprus and dates back to the year 1897. It has been constructed for the purpose of storing the water of the Merikeros stream which has its sources at the Kyrenia hills and discharges its flow to the sea North of Famagusta. From this dam several downstream intakes have been supplied with water for spate irrigation of cereals in the plains between Syngrasi and Famagusta. Irrigation practiced when water is available in the reservoir is only supplementary to that of rainfall. It occurs that some years there is no water in the reservoir due to low rainfall. After emptying of the reservoir which is usually done at about April, the land flooded by the water of the reservoir is planted with summer crops such as melons and cotton which receive their water requirements from the moisture retained in the soil.

The original dam lasted until 1941 when it was abandoned because of several breaches which occurred on the embankment and was considered very costly to repair it. Until 1967 the reservoir area was used for agricultural purposes only. A very bad erosion of the banks occurred from the big floods of 1949 which had a maximum peak of 153 m^3/s and washed away the remains of the old embankment. In July 1967 the rebuilding of the reservoir started, the purpose of which was for improved irrigation of the cereals downstream, for flood protection of the downstream lands, for reclamation of the irrigated lands in the bed of the reservoir on which summer crops could be cultivated and more important, for groundwater recharge of the Lapaphos aquifer. The need for the groundwater recharge of the Lapathos aquifer expedited the re-construction of this dam.

The groundwater recharge became necessary because of the depletion of the Lapathos aquifer which is used for domestic water supply and irrigation purpose. Due to the excessive pumpage the water table level dropped 6 m from 1962 to 1965. Water for domestic uses is pumped from this aquifer to 12 villages having a population of 12,000.

2. LOCATION

The dam is located on the Merikeros stream at about a distance of 1 km West of Syngrasi village and about 2.5 km from the sea. It lies at an altitude of about 27 m asl.

3. PLAN

a. Water and Land

No automatic flow measurements were available from this stream but from the rainfall data it has been estimated that the average runoff at the dam site is about 0.8 million m^3/a. The maximum flow calculated at a frequency of 1/1000 years is estimated to be of the order of 164 m^3/s. Water use rights from this stream were exercised by farmers from the villages of Syngrasi, Ayios Yeoryios, Arnadhi and Spathariko having intakes on the river for the irrigation of about 4,000 donums of cereals. One of the purposes of the dam is to satisfy these water use practices under the the management of an Irrigation Division for every village.

b. Geology

This dam lies on the Nicosia — Athalassa Formation. The upper part of the catchment is made up of the Hilarion limestone, followed by the Lapithos chalks which are in turn followed by the Kythrea Formation and lastly by the Nicosia — Athalassa Formation at the damsite. The abutments of the damsite are made up of sandstone beds which are quite permeable, whilst the river bed is made up of impermeable marls. To determine the depths of the cut-off at the damsite a series of testpits have been excavated and many tests carried out.

The Lapathos aquifer starts from the Southern side of the Syngrasi reservoir and extends over an area of 34 km^2, its maximum length being about 5 km.

It is a rather flat region, its maximum ground elevation being about 60 m asl. There is only a small drainage in the form of a small valley South of Lapathos with a NW — SE trend. Dry farming accounts for the greatest portion of this intensely cultivated area. The main irrigated crop is citrus.

The area, is partly covered by a thin layer of hard limestone locally known as Kafkalla. It overlies a layer of soft secondary calcareous rock known locally as havara. Below the havara there are layers of fine to medium-grained porous calcarenite interbedded with sandy marls of Plio-Pleistocene age of the Nicosia— Athalassa Formation. These calcarenetic layers form the aquifer of the area. Under the sandy marls we meet the Miocene Kythrea Formation which forms the base of the aquifer. The thickness of the aquifer varies from 17 to 30 m.

4. MAIN FEATURES

a. Dam

The design of this dam as well as that of the ground water recharge scheme have been undertaken by the WDD. The dam is made up of a homogeneous single zone embankment protected on its upstream site by handplaced rip-rap rocks. The spillway has been lined in concrete right down to riverbed. A 0.45 m dia pipeline has been provided through the embankment supplying water to the downstream intakes and pumping plant. From the pumping plant water is pumped through a steel conveyor to the recharge wells. Before the water gets into the wells it is treated by the application of chlorine. Analyses of the reservoir water show that the water gets contaminated by bacteria whilst after many days of storage most of these bacteria die out in the reservoir without any application of chlorine.

5. CONSTRUCTION

The dam was constructed by the WDD between July 1967 and December 1967. There were no difficulties during construction as it was a simple straight forward homogeneous embankment work. The compaction of the fill was done in 0.25 m thick layers by means of pneumatic rollers at a maximum dry density of about 1,700 kg/m^3 and at an optimum moisture of about 20%.

The total expenditure on this dam was £24,000.

For the spate irrigation of 4,000 donums, earth channels of a capacity between 0.45 and 1.2 m^3/s are available since the dam was first constructed in 1897. Due to the infrequent flow it has not been considered economic to line these canals.

The Lapathos groundwater recharge scheme was constructed after a pilot project had been tested and proved successful. Based upon the results of the pilot project, the recharge scheme has been designed to infiltrate about 4,500 m^3/day through 13 wells reaching a depth of about 7 m. The scheme is made up of a pumping unit at the reservoir and an AC pressure conveyor pipeline of 0.10, 0.15, 0.20 and 0.25 m dia and 13 suitably selected wells in which the water is introduced for recharge. The cost of this recharge scheme reached £11,000.

6. MANAGEMENT

This project is under the management of the Government which is responsible to supply water to the donwstream intakes for irrigation and to the wells at Lapathos for groundwater recharge. The land commanded by the earth distribution system from the downstream intakes is about 4,000 donums of cereals. The administration is done through a Committee appointed by Government and known as the Eastern Messaoria Irrigation Works Committee which manages also some other irrigation works in this region.

The total extraction of water for domestic purposes from the Lapathos aquifer in 1967 reached up to 0.67 million m^3 from seven boreholes. For the irrigation of 1,400 donums of citrus and vegetables from 150 wells and boreholes the extraction was 1 million m^3. The total amount of water that was used for recharge in the year 1967 was 0.18 million m^3 and in the year 1969 it was 0.12 million m^3.

7. PROBLEMS OF SPECIAL INTEREST

The dam itself presents no problems of any particular interest. However, the observations on the Lapathos aquifer recharge give interesting results.

a. Experimental pilot recharge project

The recharge experiment was carried out between the 14th February until the 19th April, 1967 during which period 0.18 million m^3 of water from the Syngrasi reservoir area where a small storage dyke had been constructed were pumped through a 0.10 m dia steel conveyor and recharged through a number of wells and boreholes at Lapathos.

The silt content problem in the reservoir water was one important factor to be considered. The inflow floods into the reservoir were found to contain from 1 to 5 kg/m^3. However, in 6 to 12 hours the water in the reservoir was found to clear up. Water which contains up to 0.05 kg/m^3 of silt was found not to present any difficulties in infiltration and since this could easily be obtained with only a few days storage in the reservoir it was concluded that no filtration plant would be required.

The salinity of the water was also considered and was found to be very satisfactory. Although the salinity of the water in the reservoir rose from 32 ppm to 160 ppm after 2 months storage the salinity of the water in the Lapathos aquifer reached up to 150 ppm. However, the hardness of the water in both the reservoir and the aquifer was high being up to 620 ppm and 600 ppm respectively.

One other important factor which had to be considered, especially due to the fact that the Lapathos aquifer water is used for domestic water supply, was the bacteriological contamination risk. Whereas the first sample of water from the reservoir in February showed 800 coliforms per 100 ml of water, subsequent samples after one month storage showed in some cases coliforms absent. However, due to the domestic requirements it has been decided to use chlorine in the water at the entry into the recharge wells.

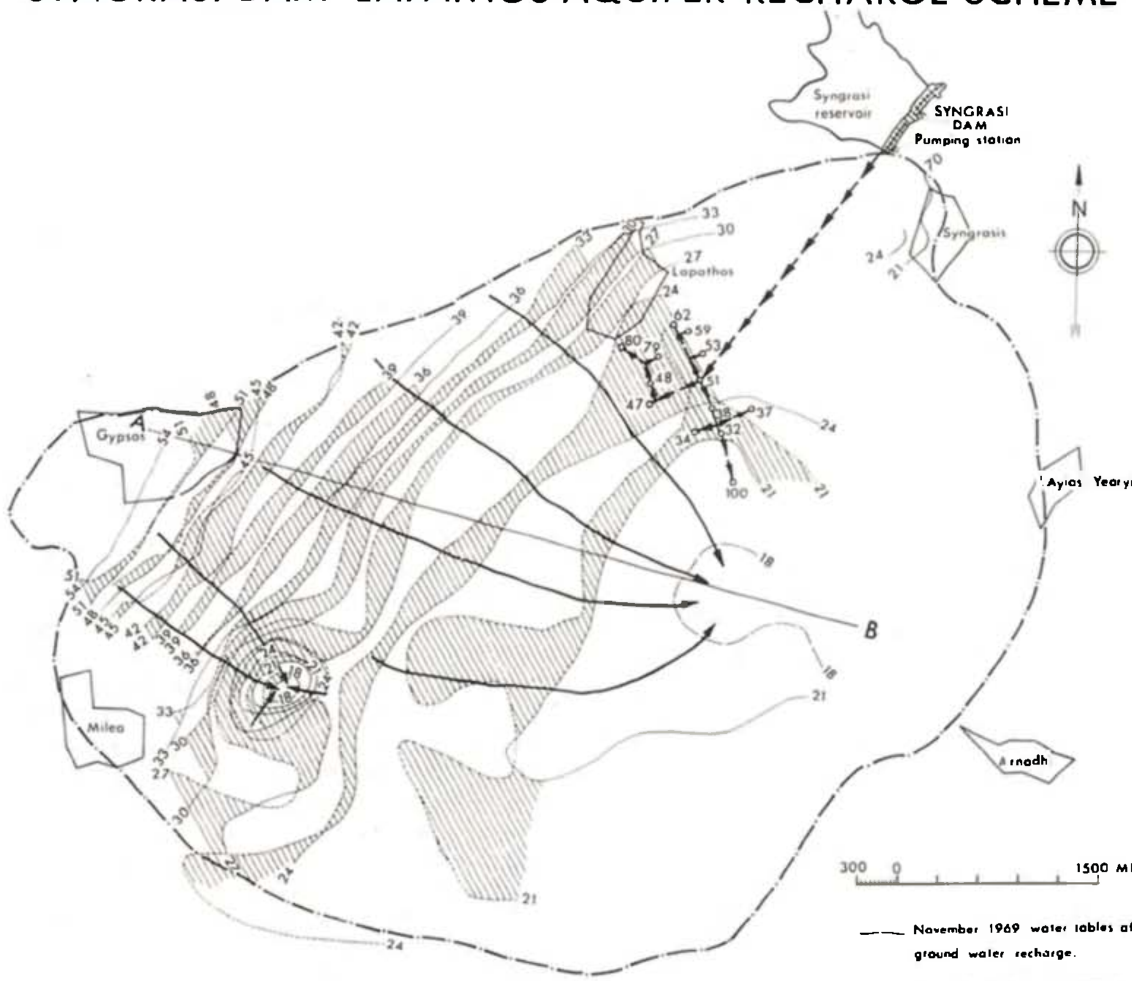
Frequent readings of the water levels in the recharge wells and in nearby wells were carried out to determine the effect. The rate of infiltration through the recharge wells varied from 1.4 to 14 l/s. Some of the wells showed no decline in the rate of infiltration but some in which silty water above 0.05 kg/m^3 was used, the rate was reduced up to half of the original.

In total about 0.18 million m^3 of water were recharged through 9 wells and water levels were raised up to a maximum of 2.5 m in an area of at least 1.1 km^2. Infiltrating water has been found to move in the aquifer at an average rate of 30 m/day but in certain directions rates of up to 75 m/day were observed.

b. Permanent Recharge Project

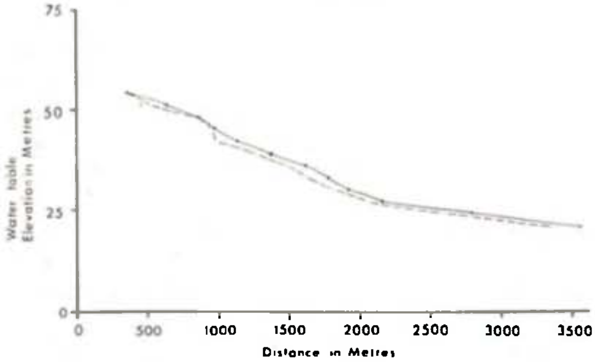
The permanent recharge scheme was put for the first time after construction into operation between the 7th January and the 11th April, 1969 and during this period about 0.12 million m^3 of water recharged through 13 wells in the area. Water levels in a region under observation of 2.5 km^2 rose up to 2.1 m by the end of the recharge period, the average throughout the area being 1.3 m. Sterilization by chlorine was constantly applied because bacteriological examination of the water in some wells showed up to 240 coliforms per

SYNGRASI DAM - LAPATHOS AQUIFER-RECHARGE SCHEME



- November 1969 water tables after ground water recharge.
- November 1968 water tables before ground water recharge.
- Ground water recharge conveyor pipeline.
- o51 Ground water recharge wells.
- Lapathos Aquifer boundary.
- Direction of ground water flow.
- Positive displacement of ground water contours due to recharge.

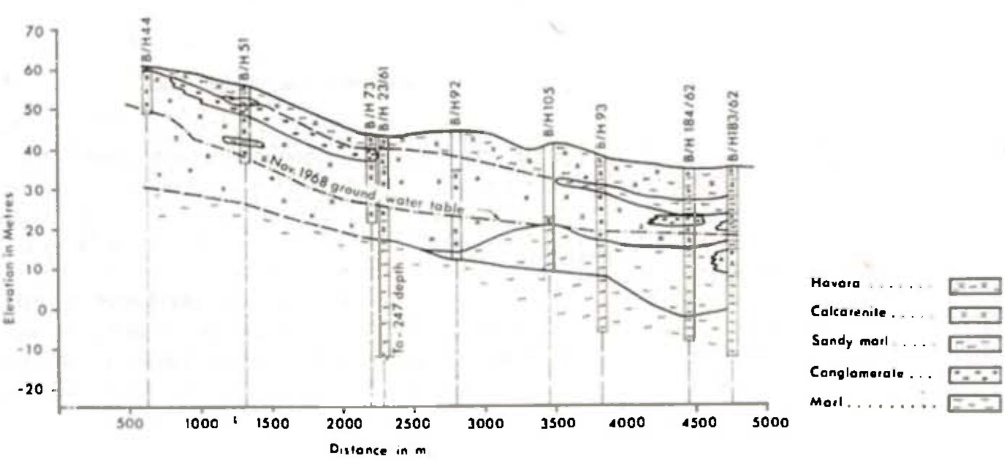
PLAN



SECTION A-B

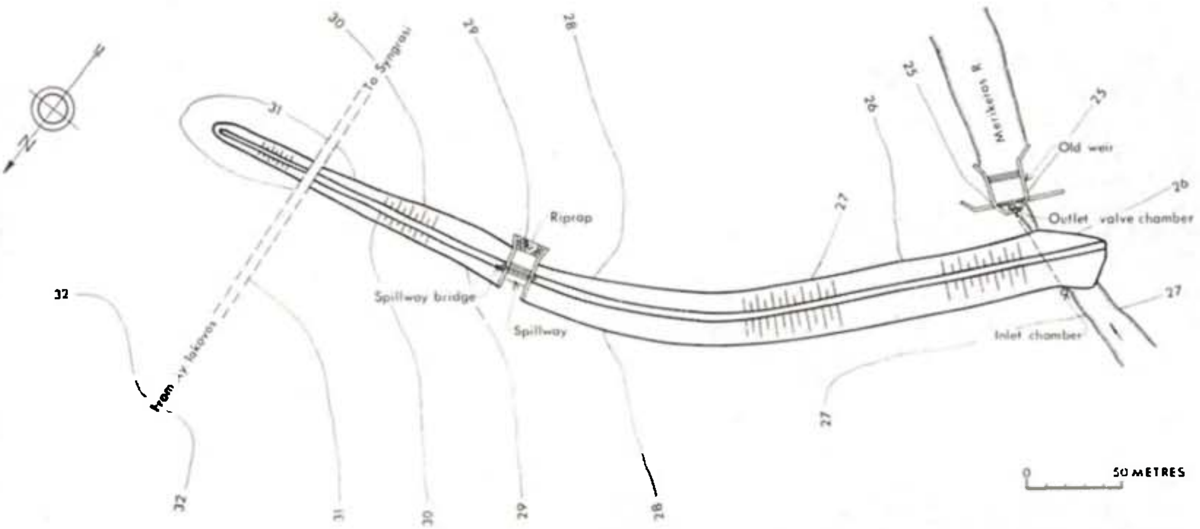
Note

4.2% of the rise is due to artificial ground water recharge and 95.8% is due to natural replenishment.

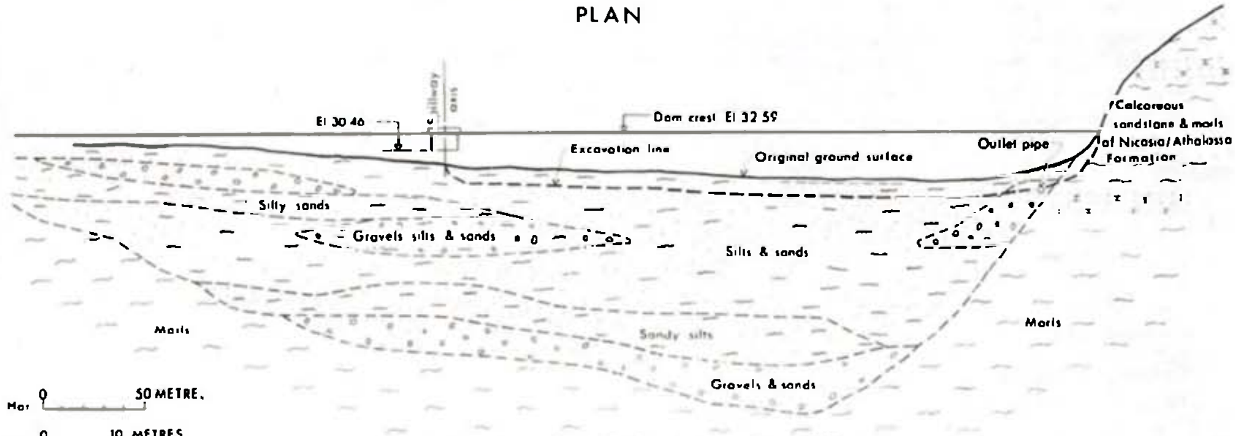


GEOLOGICAL SECTION A-B

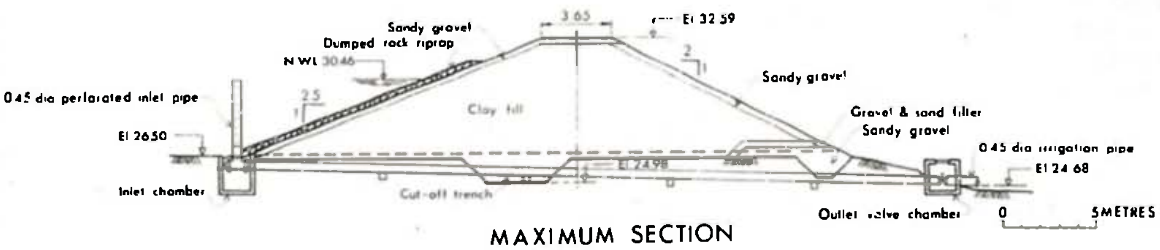
MERIKEROS-SYNGRASI DAM



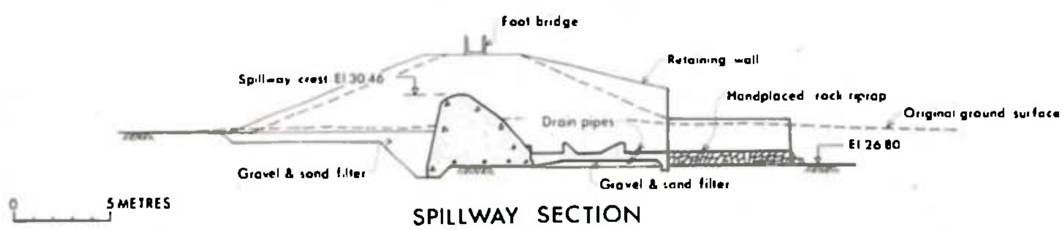
PLAN



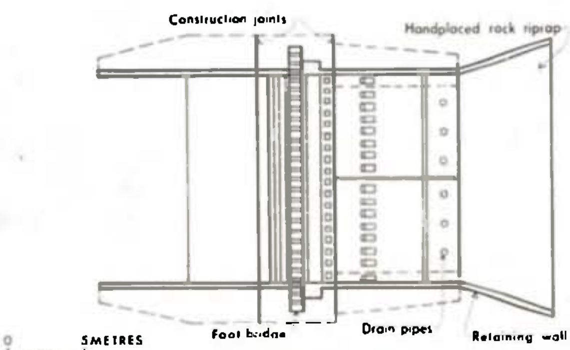
GEOLOGICAL SECTION ON DAM AXIS



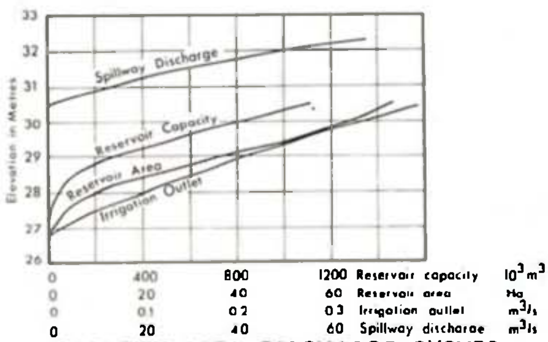
MAXIMUM SECTION



SPILLWAY SECTION



SPILLWAY PLAN



CAPACITY–AREA–DISCHARGE CURVES

100 ml. of water.

The 120,000 m^3 recharged represent only 4.2% of the total 2,700,000 m^3 recharge during the hydrological years 1967 — 1969, the remaining 2,580,000 m^3 representing 95.8% is accounted for by direct rainfall infiltration. On the other hand the extraction during the same period was estimated to be 2,124,000 m^3 for irrigation and 580,000 m^3 for domestic water supplies, that is to say a total of 2,704,000 m^3. This balance shows that the recharge of 120,000 m^3 made all the difference for balancing the extraction with the recharge during the period under reference.

To enable a better distribution of the recharge water it has been concluded that the recharge wells should be spread parallel to the flow lines and not accumulated on one side of the aquifer at right angles to the flow lines as had actually been placed.

For better determination and evaluation of the results, further work has to be carried out for determining more accurately the geometry of the aquifer. Test pumping has to be made to determine the aquifer constants and better measurements of the recharged water, quantitatively and qualitatatively have to be undertaken. The use of properly located observation boreholes is also of importance.

Data

i. Catchment

Area	44 km^2
Average rainfall	0.45 m
Average runoff	0.8 million m^3/a
1/1000 years flood	164 m^3/s
Maximum height	636 m
Maximum length	12 km

ii. Reservoir

Area	72 ha
Capacity	0.11 million m^3
Live storage	0.10 million m^3
Length	1,320 m

iii. Embankment

Structural height	7,61 m
Height above ground level	6.09 m
Hydraulic height	3.96 m
Depth of foundation cut-off	1.52 m
Freeboard	2.13 m
Crest length	489 m
Top thickness	3.65 m
Base thickness	31 m
Upstream slope	1:2.5
Downstream slope	1:2
Clay core	52,500 m^3
Sandy gravel	1,500 m^3
Gravel sand filter	2,500 m^3
Conglomerate fill downstream	5,600 m^3
Rock rip-rap (upstream)	1,100 m^3
Total earth fill	63,200 m^3

iv. Excavations

Total	12,700 m^3

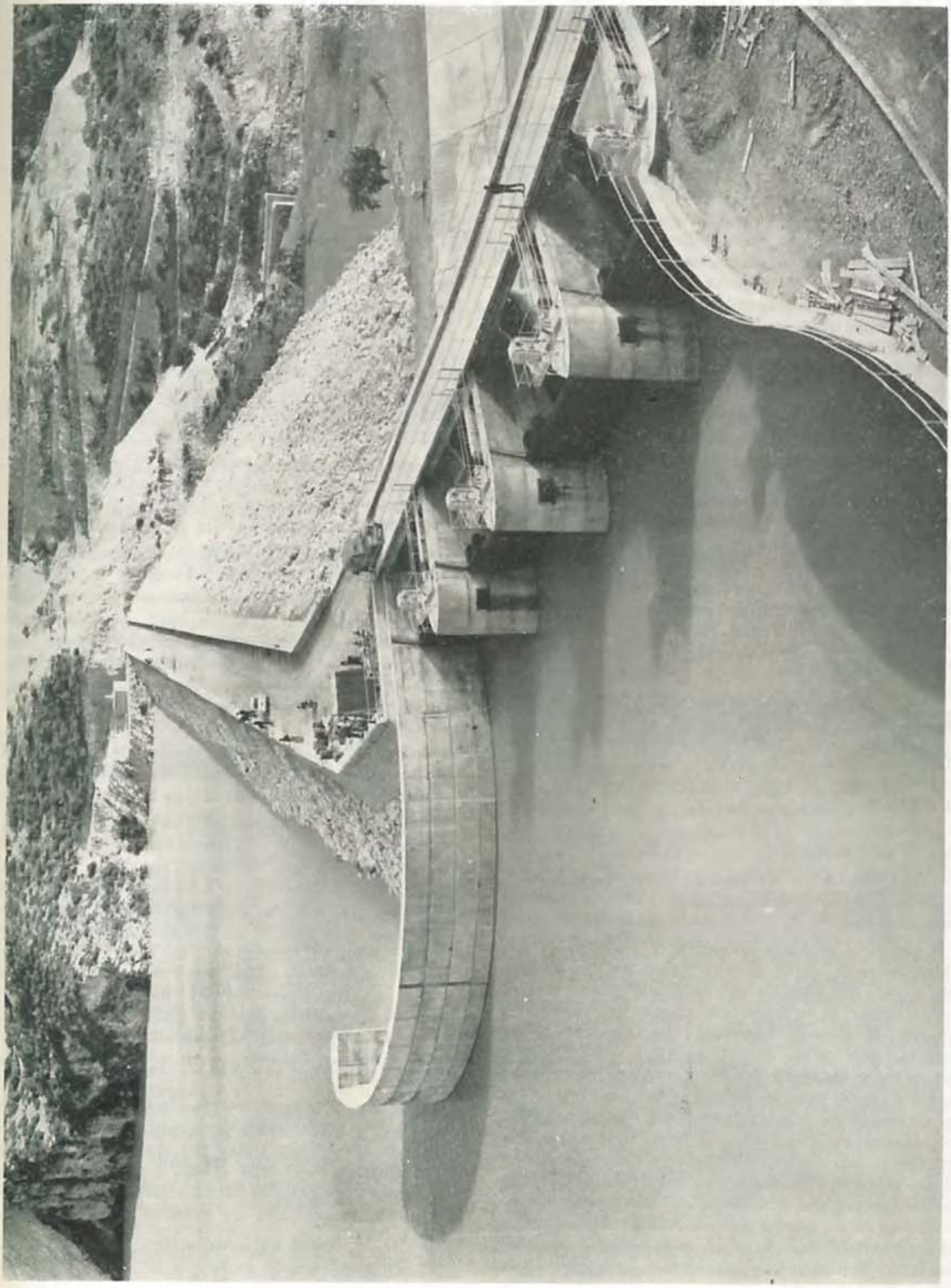
v. Spillway

Size	13.6x2.1 m
Capacity	164 m^3/s
Length	18.6 m
Concrete	335 m^3

vi. Outlet (Steel pipes)

Size	0.45 m
Capacity	0.4 m^3/s
Length	66 m

YERMASOYIA



The Yermasoyia dam.

YERMASOYIA—YERMASOYIA PROJECT

1. PURPOSE

This dam is part of the Master Plan for the development of the water resources of the Akrotiri peninsula, one source of which is the Yermasoyia river. With the overpumping of the groundwater resources to satisfy the existing plantations and the inadequacy of the Polemidhia dam to satisfy the deficiency, the Yermasoyia dam had to be built.

The conveyance of water to Akrotiri includes the expansion of the cultivation of citrus by another 4,000 donums in the same area neighbouring the present groundwater fed plantations. Also 920 donums of new lands will be irrigated at Akrounda and Phinikaria villages upstream of the dam to substitute lands that have been flooded by the dam.

Another important reason for building the dam is the need for meeting the water supply requirements of Limassol and the neighbouring villages whose demand is steadily increasing, as well as the domestic water supply requirements of the coastal region in the East of Limassol which is developing as a tourist and summer vacation centre. The contribution of the dam for the domestic supplies is in the form of groundwater recharge of the river gravel aquifer downstream from where water is extracted by pumping.

2. LOCATION

The dam is situated on the Yermasoyia river at a distance of about 4.5 km upstream of the sea and at an elevation of about 60 m asl.

3. PLAN

a. Water and Land

The flow of the Yermasoyia river is estimated to yield an average of 22.5 million m^3/a with an expected maximum flood of 1/1000 years frequency at 695 m^3/s.

The only previous utilization of water from this river upstream of the damsite before the construction of the dam has been through an intake which provided winter irrigation to lands belonging to the Yermasoyia village. The most important use had been the replenishment of the downstream river alluvial aquifer which yields water for domestic purposes for Limassol and the nearby regional villages. Just upstream of the reservoir two river intakes served for limited irrigation of seasonal crops cultivated in lands belonging to the villages of Akrounda and Phinikaria. A great part of these lands was inundated by the impounded water in the dam and had to be acquired. For several reasons including social, the Government decided to pump water above the flood line of the reservoir in order to supply water for irrigation to new lands belonging to the two affected villages.

b. Geology

The geological structure of the area around the Yermasoyia dam is formed of the Pakhna formation made up of alternating series of chalk, marly sandstone, chalky sandstone, chalky marl, marly sandstone and other similar transitional varieties, the thickness of each varying from 3 to 10 cm. Havara soil occurs as a crust which has recently covered the bedrock.

The right hand side of the valley seems to have been eroded by water, as the parent rock there is mostly washed away. On the left abutment, the sandstone face is vertical which indicates that this face was not subjected to active erosion. A tectonic action may be assumed with an inferred fault along the river valley justifying the steep topography to the left abutment as compared with the more or less flat topography on the right.

The left abutment includes a lot of cavities and solution channels met during construction and which in some cases go along the whole width of the dam whilst in other cases vertical solution channels exist reaching from river bed up to the top of the left abutment, a total height of the order of 50 m.

In the river valley, sediments of gravel and sand are deposited which reach a depth of up to 40 m. Several terraces can be differentiated by age with high terraces on the right abutment. Petrographically, the sand and gravel are essentially formed of igneous pebbles and to a lesser extent of sandstones, chalk and marl fragments.

The inferred fault along the river bed runs from near the left bank in a NE direction crossing the left side of the river valley.

Due to the flat topography of the right abutment the spillway structure has been sited there. The outlet tunnel has been sited on the left abutment, because of the flat topography and the more favourable geological characteristics such as better stability, the approximately horizontal position of the layers, the uniformity of the mechanical properties of the rocks and the slight tectonic distrubance. The soft rocks available would make excavation easy but on the other hand the presence of solution channels and many open joints would result in sliding and separation of rock pieces. These problems however, were not expected to be serious.

On the other hand, the great depth of the alluvium in the river valley with permeabilities ranging from 10^{-3} to 10^{-2} cm/s makes the problem of rendering this alluvium water tight quite a difficult task. A satisfactory permeability of the alluvium required after treatment should be of the order of 10^{-5} cm/s.

For the design and construction, extensive drilling and testing of the site as well as laboratory tests have been carried out.

4. MAIN FEATURES

a. Dam

The detailed design of this dam has been carried out by EnergoProjekt of Yugoslavia whereas the WDD carried out the preliminary plans.

b. Distribution System

The distribution scheme and the conveyance of water to the Akrotiri peninsula on the West of Limassol have been designed by the WDD. For this water distribution system many alternative designs have been studied in addition to the Akrotiri diversion which included the conveyance of water to the East and to the North of Limassol for irrigation, or to utilise, the dam water totally for the domestic water supply of Limassol or Famagusta.

Eventually, the conveyance of water to Akrotiri for the primary purpose of satisfying the water requirements of the existing plantations has been decided. The main conveyor has to pass through the town of Limassol and is a gravity AC pipeline of about 0.90 m dia being of a total length of about 12 km to reach the existing Polemidhia distribution pipe.

The distribution system provided for the Akrotiri expansions at Trakhoni comprises about 15,000 m AC pressure pipes of various diameters ranging from 0.10 to 0.45 m.

The distribution system including the conveyor to the lands of Phinikaria—Akrounda provides for about 12,000 m of AC pressure pipes ranging from 0.10 to 0.40 m dia.

All the distribution system is designed to provide for sprinkler irrigation.

For distribution network map see Garyllis—Polemidhia project.

5. CONSTRUCTION

The construction of the Yermasoyia dam commenced in January 1966 and completed in October 1968. The main Contractor on the dam was CYBARCO of Nicosia. For the foundation treatment works and the chemical grouting, Cementation Company of London has been employed as a Sub-contractor. For the hydromechanical equipment both for the bottom outlet and for the spillway gates A.S. Koupas of Greece have been employed. The WDD has carried out the preliminary works, access roads and other services.

The total cost of the construction works has been £930,000 the main items of which were:—

Main earthfill work	£470,000
Grouting sub-contract	£190,000
Land acquisition	£110,000
Auxiliary works carried out by the WDD at	£160,000

Construction materials such as clay for the dam core, filter material for the transition zones, random fill and rip-rap rocks were obtained from borrow areas within or near the reservoir.

Observation equipment used on the dam included pore water pressure cells for measuring stresses induced by the seepage water and total pressure cells for measuring stresses caused by the earthfill and water load. Also surface movement monuments for measuring horizontal and vertical displacements on the embankment and piezometers for recording underground water levels downstream have been provided.

a. Rock Grouting

The rock grouting was executed starting from bottom to top with packers placed on the top of each stage injected. In special cases where the rock was collapsing in the hole thus preventing fixing the packers, grouting was carried out from the top to the bottom, drilling and grouting stage after stage. In the case of big absorption the packer was not used but in all cases where the grout takes were not significant packers were used. Permeability under pressure was tested in most holes before grouting and certain stages were water tested after grouting. The grout mix used was 95% cement and 5% bentonite except in cases of considerable permeabilities when sand was added to the mix in the ratio of 1:1 to 1:3 cement sand. The permeability in the control holes was considered as satisfactory for permeabilities less than two lugeons.

Primary, secondary and tertiary rows of holes were used in rock grouting. The most extensive treatment was required in the East abutment extension.

The grout consumptions in each zone treated are as given in Table 14.

Regarding the grouting of the foundation of the spillway this was done through pipes left in the concrete during construction. Grouting was carried out without using the packer but by connecting the injector with the top of the pipe. For the grouting, maximum pressures applied were 2 kg/cm^2 for the first stage and 3 kg/cm^2 the second stage.

The tunnel and shaft grouting work included the inlet and outlet chamber concrete grouting, rock contact grouting and steel lining contact grouting. Contact grouting of any part of the tunnel was allowed to start only 14 days after placing concrete at the particular point to be grouted. Two stages were applied, the first one for grouting primary profiles on each second ring and the second stage on grouting secondary profiles between the primaries. Grouting of the fan holes has been made in two 3 m stages. In the inlet and outlet chambers and the shaft, the normal pressures under which the largest part of grouting was injected being 3 kg/cm^2 whereas the final grouting was done up to 6 kg/cm^2. The grout mix was of 3% bentonite and 97% of cement. In cases of high takes sand was added to the grout mix. The criterion of satisfactory grouting was that the consumption should be less than 2.25 l/s per minute of 15 minutes time, which if not fulfilled, the zone in question had to be regrouted. Generally the work was satisfactory when the degree of water tightness obtained ranged from 0 to 6.65 lugeons.

In addition to permeability tests, core recovery was made from these control holes. Geological description of these cores and core photographs were taken.

TABLE 14

	Primaries	Secondaries	Tertiaries
Zone 1			
Number of holes	14	12	15
Number of stages	83	70	89
Total length of stages (m)	476	374	474
Total consumption of cement (kg)	71,130	75,400	22,950
Total consumption of bentonite (kg)	1,930	2,130	635
Total consumption of sand (kg)	25,700	14,550	--
Total consumption of clay (kg)	4,960	2,010	1,975
Total consumption of all materials (kg)	103,720	94,090	25,560
Average consumption (kg/m)	207	252	54
Zone 2			
Number of holes	21	20	40
Number of stages	84	72	134
Total length of stages (m)	457	374	648
Total consumption of cement (kg)	26,201	19,219	23,515
Total consumption of bentonite (kg)	810	594	727
Total consumption of sand (kg)	—	—	—
Total consumption of clay (kg)	—	—	—
Total consumption of all materials (kg)	27,011	19,813	24,242
Average consumption (kg/m)	59	53	38
Zone 3			
Number of holes	22	21	39
Number of stages	79	74	139
Total length of stages (m)	392	377	702
Total consumption of cement (kg)	28,600	8,450	15,400
Total consumption of bentonite (kg)	1,375	374	740
Total consumption of sand (kg)	—	—	—
Total consumption of clay (kg)	83	74	222
Total consumption of all materials (kg)	30,058	8,898	16,362
Average consumption (kg/m)	62	21	23
Zone 4 and 5			
Number of holes	13	12	16
Number of stages	27	26	37
Total length of stages (m)	162	165	214
Total consumption of cement (kg)	19,450	12,000	12,300
Total consumption of bentonite (kg)	610	373	648
Total consumption of sand (kg)	4,940	7,200	200
Total consumption of clay (kg)	—	—	—
Total consumption of all materials (kg)	25,000	19,573	13,148
Average consumption (kg/m)	155	118	62
Right Extension			
Number of holes	12	9	16
Number of stages	46	35	61
Total length of stages (m)	286	218	376
Total consumption of cement (kg)	52,600	21,850	21,350
Total consumption of bentonite (kg)	1,425	585	570
Total consumption of sand (kg)	33,750	6,830	—
Total consumption of clay (kg)	—	—	—
Total consumption of all materials (kg)	93,775	29,265	21,920
Average consumption (kg/m)	318	134	48
Left Extension			
Number of holes	9	8	17
Number of stages	46	38	135
Total length of stages (m)	264	215	821
Total consumption of cement (kg)	202,503	131,261	162,056
Total consumption of bentonite (kg)	6,262	4,059	5,012
Total consumption of sand (kg)	240,391	163,368	—
Total consumption of clay (kg)	—	—	—
Total consumption of all materials (kg)	449,136	298,638	167,068
Average consumption (kg/m)	1,700	1,390	204
Grand Totals			
Number of holes	91	82	143
Number of stages	365	315	595
Total length of stages (m)	2,037	1,723	3,235
Total consumption of cement (kg)	400,484	268,183	257,571
Total consumption of bentonite (kg)	12,412	8,115	8,332
Total consumption of sand (kg)	310,781	192,028	200
Total consumption of clay (kg)	5 043	2 084	2,197
Total consumption of all materials (kg)	728,720	470,327	268,300
Average consumption (kg/m)	358	273	83

Excavation of tunnel showing solution channels in the sandstone.

Cut-off trench excavation with alluvial grouting.

b. Alluvial Grouting

The Yermasoyia valley itself has been formed by erosion and sedimentation of alluvial deposits in the form of gravel, sand and silts. These deposits reach a depth of up to 40 m below river bed level, and are saturated with water, the water level varying from complete saturation during the Winter and Spring months to a few metres below river bed level during Summer and Autumn months. Tests carried out showed the permeability of the alluvial in-situ to be about 10^{-3} cm/s and in some cases as high as 10^{-2} cm/s which indicates that the overall permeability of the alluvium appears to correspond to that of a clean medium to coarse sand. This permeability was found for the upper 20 m whereas the lowest portion was found to be more permeable.

In view of the depth of the alluvium and of the amount of seepage, it was decided that the best method for making a permeable foundation curtain would be by alluvial grouting using chemical grouts as well.

The Sub-contractor Cementation Co., of London, untertook certain tests on sand specimens which were grouted using two types of grout, one named Cemex A being a patent of the Company, and the other Silicate-bicarbonate.

The Cemex A is sodium silicate, oxalic-aluminum chloride mix. The gelling time is about 15—40 min. viscosity 1.3 centipoise.

The results show an increase of the permeability with time for the sodium bicarbonate grout and no increase for Cemex A. These results confirmed previous knowledge that mixtures of sodium silicate-sodium bicarbonate are unstable, deteriorating with time and losing their solidification.

In addition to the above, tests were made on two types of bentonite which established that a bentonite-water suspension having a minimum bentonite concentration of 2% will support cement particles whilst with 3½% concentration there will be practically no bleed. These characteristics vary slightly depending on the type of sodium bentonite used.

In view of the results obtained a decision was taken to use Cemex A for the chemical grouting.

Before proceeding with the actual grouting, four rows of eight holes were selected for making grouting tests, in order to plan the works. These control holes showed satisfactory results giving permeabilities after grouting not exceeding 10^{-4} cm/s which is considered satisfactory for this case.

The procedure for grouting was as follows:—

Four rows of holes were drilled at 1.4 m spacing between the holes and 1.2 m between the rows, and injected through "tube a manchette" sleeve tubes. Drilling of all holes was done up to 1.5 m into the rock using casing fixed to the top of the bedrock and plastic pipes of 37 mm dia with sleeves to withstand a pressure of 84 kg/cm² were lowered inside the casing. The drilling was performed, by overburden drill of 87.5 mm dia casing.

The advantage of the "tube a manchette" system is that each horizon can be grouted several times with progressively finer grouts.

Then a coating grout of cement-clay mix was used to provide a good sealing along the grouting pipe and to enable its breakage with relatively low pressures. The mix used for the coating was 15% of cement, 20% of clay and 65% of water, giving a strength in 14 days of 7 kg/cm². This mix was injected by means of a double ended packer through the lowest sleeve until it appeared at the top of the hole when the casing was withdrawn at the same time keeping the annulus grout topped up.

After the clay cement coating, water tests were carried out for determining the permeabilities.

TABLE 15

Grout	Unconfined compressive strength (kg/cm²)		Permeability (cm/sec)		
	7 days	28 days	7 days	170 days	360 days
Cemex A	1.97	1.89	$1x10^{-7}$	$1x10^{-7}$	
Sodium Silicate— Sodium bicarbonate	1.41	1.54	$2.6x10^{-6}$	$1.3x10^{-5}$	$2x10^{-5}$

The sand samples were prepared from a sand graded so that 100% passed the 100 B.S. sieve (max. grain size 0.152 mm), 10% passed the 200 sieve and nil passed the 300 sieve. The size of samples prepared for unconfined compressive strength measurements were 75x12 mm dia while those prepared for permeability measurements were 300x75 mm dia cylindrical vessels sealed at the base and filled with sand. The sand was compacted by vibration and any surplus grout appearing was syphoned off and measured. A load of 0.2 kg/cm² was applied through a porous plate. After consolidation the porosity was 39.5%. All grouts were designed to give a 40 minutes gel time and the silicate-bicarbonate grout to provide this gel time was as follows:

1.4 parts sodium silicate

1.8 parts water

3.1 parts sodium-bicarbonate (6.6% solution)

The permeability measurements were under an applied head of water of 6m corresponding to a hydraulic gradient of 20.

Table 15 shows the results of testing:

Following the water tests, injection was carried out, of cement-bentonite mix of 12% bentonite, 24% cement and 64% water, starting with the two outer rows. The grouting pressures for this operation were 7 to 10 kg/cm².

Regrouting with cement-bentonite grout was carried out for the holes which showed high permeability.

Lastly, for the holes which still showed high permeability, these were injected with chemical grouting using Cemex A.

The grouting pressures for this operation ranged from 7 to 12 kg/cm². In stages where the takes of grout were very low the pressures were increased upto 20 kg/cm² for cement-bentonite and upto 28 kg/cm² for Cemex A.

Finally, the two inner rows of holes were grouted with the same procedure as above.

For grouting with cement-bentonite mix the consumption varied from 220 to 450 l/stage and grouting was considered completed when the rate of pumping did not exceed 9 l/m.

For the chemical grouting, the consumption varied from 110 to 450 l/stage and grouting was considered completed when the rate of pumping did not exceed 9 l/m.

The overall width of the effective grout curtain may be assumed to be 4.8 m or about 15% of the depth of water in the reservoir.

A summary of the results of the alluvial grouting is given in Table 16.

The total amount of the component parts in the chemical grouting was as follows:—

Aluminium Chloride	5370 kg at £43/ton
Oxalic Acid	2560 kg at £187/ton
Silicate of Soda	2120 kg at £25/ton
Detarex	1210 kg at £750/ton

After grouting was completed, six check holes were measured for permeability values which gave permeabilities ranging from 6×10^{-6} cm/s minimum to 2.3×10^{-4} cm/s maximum.

Only in a few local pockets the permeability was found a little higher. By checking the amount of take in neighbouring holes, many suspected stages had been further reinjected with cement/bentonite grout once or even more times.

The total cost of the above grout work in 2,788 stages was £29,800.

As the area of the treated alluvium was 320 m² the unit cost is £93/m².

The cost of Cemex A was found to be 12 mils/l.

6. MANAGEMENT

This project was built at full Government cost and the same is being done for the distribution system under construction. Since this project is both for irrigation and groundwater recharge in the river valley downstream, it has been decided that it will be run by Government through a Project Board as for the other major dams. Of primary importance is the groundwater recharge of the aquifer from which the town of Limassol and several regional villages draw their water supplies. Also of some importance is the regulation of the dam for flood control against damages of inhabited areas downstream.

7. PROBLEMS OF SPECIAL INTEREST

Many problems were encountered and overcome during the construction, such as dewatering of the foundations, and the diversion floods.

Of course, the most important problem was the foundation cut-off and in particular the chemical grouting which is the first of its type in Cyprus.

The grouted curtain after completion gave permeabilities ranging from 10^{-6} to 10^{-4} cm/s which show a

TABLE 16

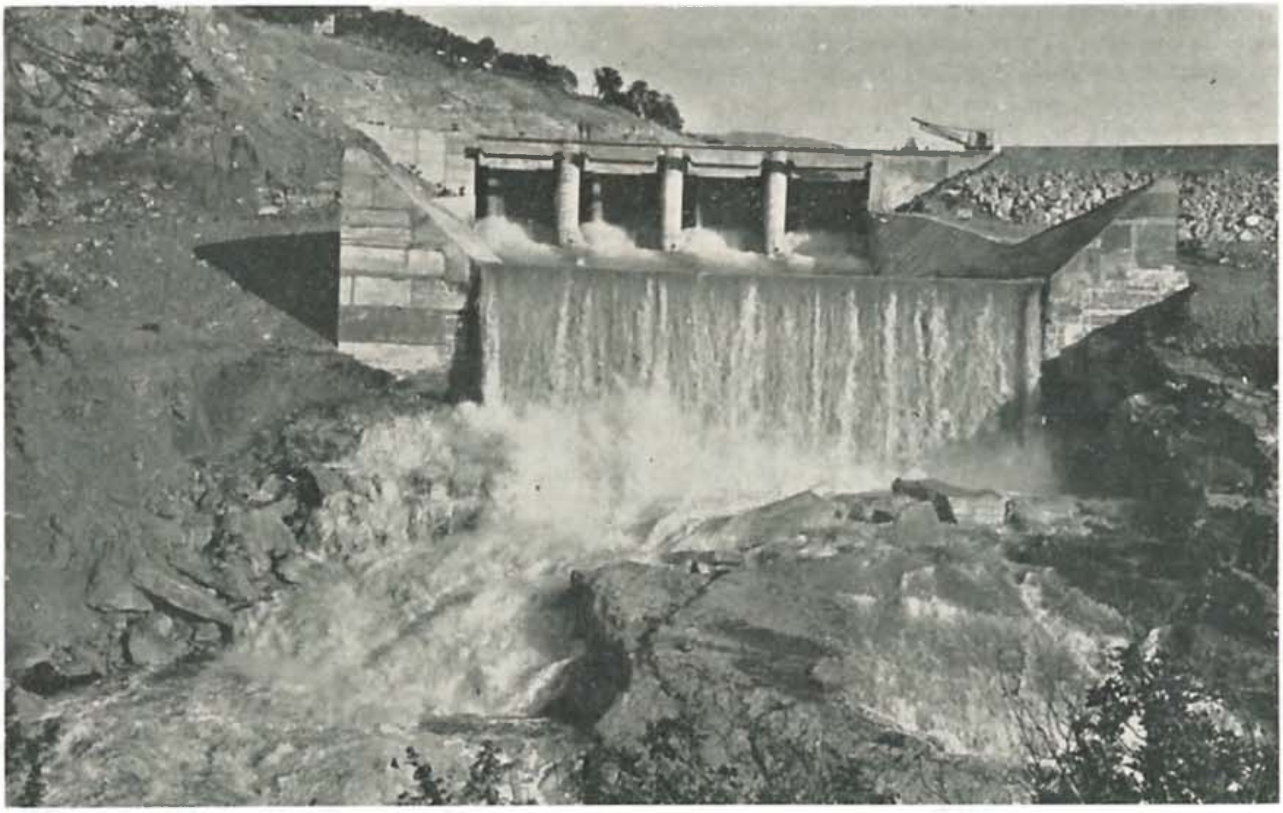
Serial No.	Grouting Operation	Rows No.			
		200	40	300	400
1.	Number of holes	28	29	28	29
2.	Number of stages	715	601	471	550
3.	Total length of stages for cement bentonite (m)	218	183	143	168
4.	Total consumption of cement (kg)	177959	135667	112015	127524
5.	Total consumption of bentonite (kg)	88974	67834	56007	63761
6.	Total consumption of dry material (kg)	266933	203501	168022	191285
7.	Average consumption of dry materials (kg/m)	1224	1112	1174.9	1138.6
8.	Total length of stages for Cemex A (m)	142	85	106	89
9.	Total consumption of Cemex A (kg/mN)	67524	47809	76708	35156
10.	Average consumption Cemex A (kg/m)	475.5	562.4	723.7	395

satisfactory grouting as compared with the permeabilities obtained prior to the grouting works which were of the order 10^{-3} to 10^{-2} cm/s.

Unfortunately, the results of each phase of grouting, that is to say the definition between cement/bentonite and chemical injections had not been tested separately and therefore the degree of effectiveness of the chemical grouting itself has not been clearly demonstrated.

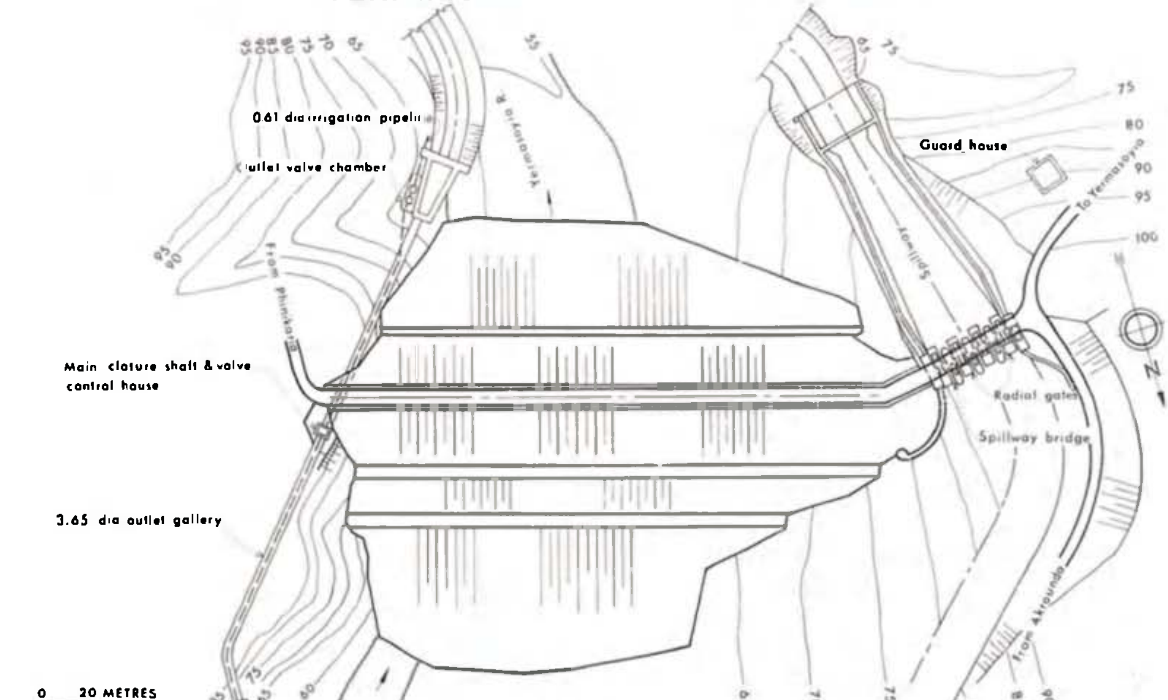
Rock grouting on the East and West abutments was the most expensive part of the grouting works because of the extensive solution channels and cracks of the Pakhna Formation. In particular the East curtain had to be extended much more than originally estimated in view of the many cavities and channels met during construction.

Another aspect of interest was the erosion of the loose sandstone rock downstream of the flip bucket which was built on sound rock foundation above river bed level. This happened during the first flood of December 1969. As a result, the spillway channel was extended in concrete lining beyond the flip bucket and fixed on sound sandstone rock.

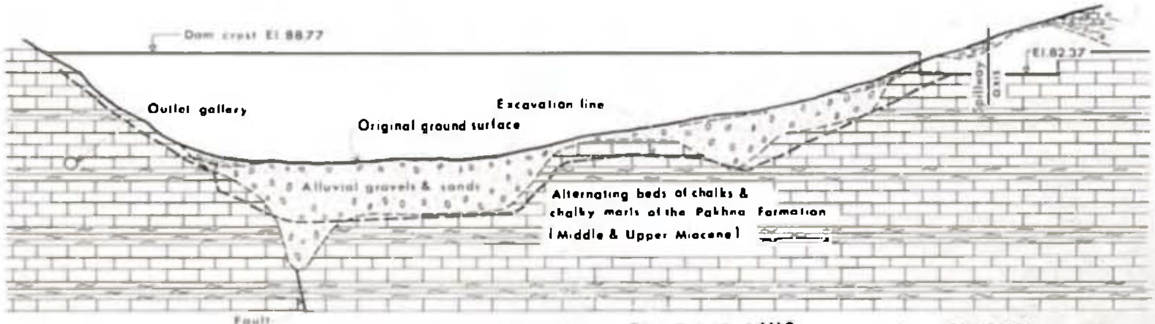


Erosion of sandstone below spillway flip bucket during first impoundment (1969 floods).

YERMASOYIA-YERMASOYIA DAM

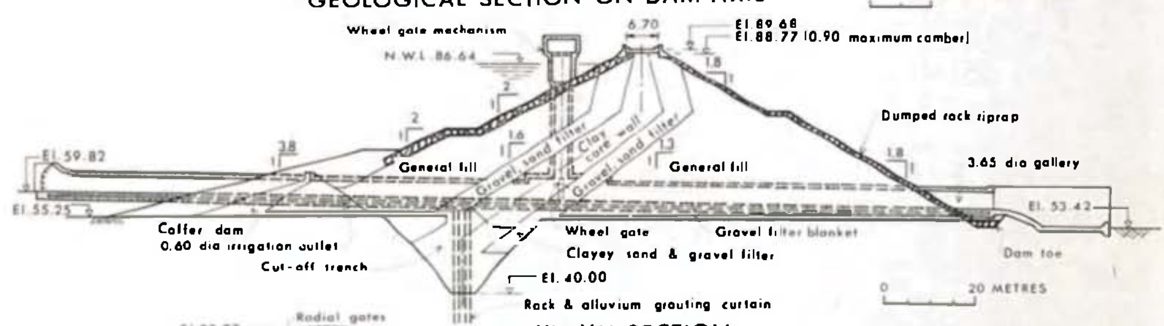


PLAN

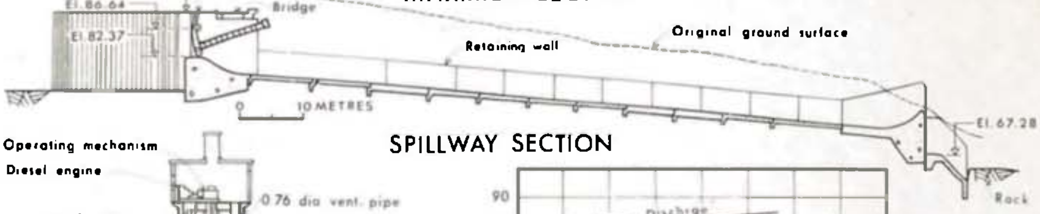


GEOLOGICAL SECTION ON DAM AXIS

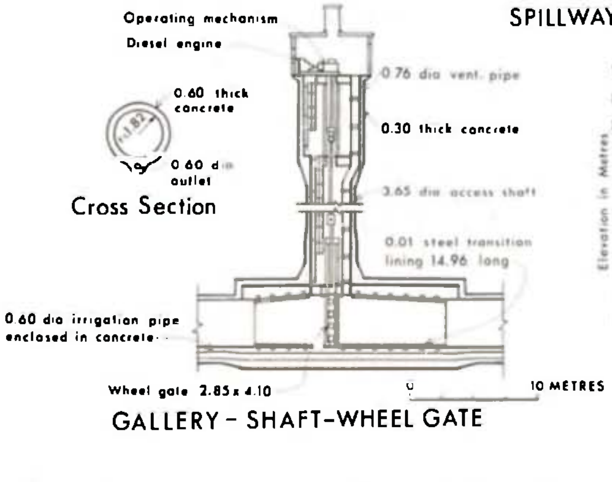
0 20 METRES



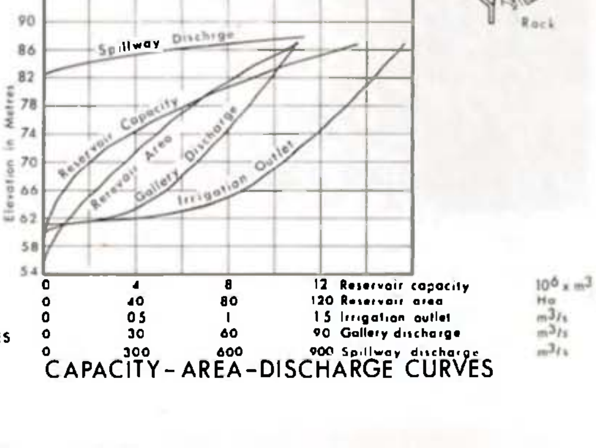
MAXIMUM SECTION



SPILLWAY SECTION



GALLERY - SHAFT-WHEEL GATE



CAPACITY-AREA-DISCHARGE CURVES

Data

Catchment

Area	156.7 km^2
Average rainfall	615.5 mm/a
Average runoff	22.5 million m^3/a
1/1000 years flood	695 m^3/s
Maximum height	1,370 m
Maximum length	32 km

Reservoir

Area	110 ha
Capacity	13 6 million m^3
Live storage	13 million m^3
Length	2.890 m

Embankment

Structural height	48.77 m
Height above ground level	33.52 m
Hydraulic height	31.39 m
Depth of foundation cut-off	15.25 m
Freeboard	2.13 m
Crest length	294.1 m
Top thickness	6.70 m
Base thickness	152 m
Upstream slope	1:2, 1:3.8
Downstream slope	1:1.8
Upstream core slope	1:1.6
Downstream core slope	1:1.3
Minimum core thickness	3.05 m
Maximum core thickness	15.23 m

Random fill	303,160 m^3
Clay core	105,250 m^3
Sand filter	89.040 m^3
Rock rip-rap	27,550 m^3
Total earth fill	525.000 m^3

iv. *Excavations*

Total	86,980 m^3

v. *Spillway*

Size	6.40x36.8 m
Capacity	850 m^3
Length	115 m
Concrete	9.710 m^3

vii. *Gallery*

Size	3.65 m dia
Capacity	81.28 m^3/s
Length	225.50 m
Concrete	3,952 m^3
Operating gate size	4.10x2 84 m

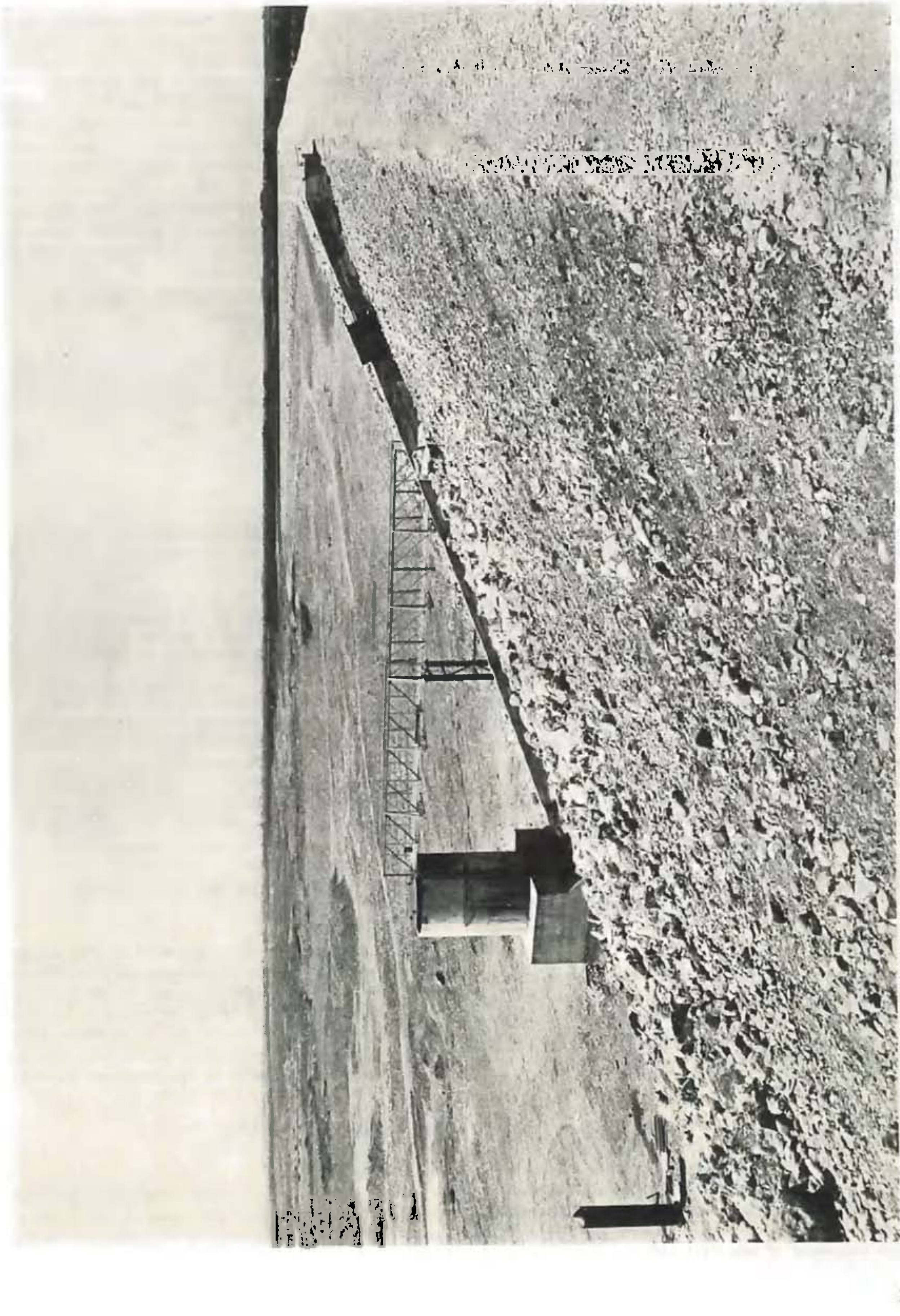
vii. *Access Shaft*

Size	3.6 m dia
Length	30 m
Concrete	600 m^3

viii. *Outlet (steel pipes)*

Size	0.60 m dia
Capacity	1.95 m^3/s
Length	240 m

MASARI



The Masari dam.

SERAKHIS — MASARI PROJECT

1. PURPOSE

Through hydrological studies made on the Serakhis catchment area, it has been determined that the frequency and size of floods occurring still yielded sufficient flow over and above the downstream water utilization potential including the Morphou dam. As the demand for water for the irrigation of the existing plantations far exceeds the supply from the ground and surface resources, the construction of this dam became a necessity in order to meet part of the overdraft from the aquifer.

Information about the recharge capacity of Serakhis river bed, upstream of Morphou dam, comes mainly from flow measurements by automatic water level recorders at Masari bridge. Taking into consideration the flow records, the inflow measurements into Morphou dam, the water levels and spillway discharges of Morphou dam, the water diverted through intakes, and the water levels in boreholes close to the river, it has been established that the Morphou dam as well as the river bed in its existing condition could only cope with small floods occurring in the Serakhis river. Additional recharge projects had therefore to be constructed in order to make full use of the water available.

After considering several alternatives, it has been decided that the best solution was the construction of Masari dam as a storage and recharge dam together with some improvement of the river bed between this dam and the Morphou dam to facilitate infiltration. The main recharge works, apart from the dam already completed, will consist of the canalization of the river and the construction of basins on either side of the central river canal within the existing river bed between the Masari and Morphou dams. The basins will have a capacity of 1.20 million m^3 which together with the Masari dam, the existing Morphou dam and the proposed recharge works will utilize all the river flow, except in the case of large floods occurring consecutively, when water would still escape to the sea.

2. LOCATION

The Masari dam is located at the junction of the three main tributaries of the Serakhis river, i.e. the Akaki, Peristerona and Meriki rivers about 18 km from the sea and 7 km upstream of Morphou dam. It is situated at an elevation of about 130 m asl.

3. PLAN

a. Water and Land

The catchment area of the dam is about 430 km^2 reaching a maximum altitude of 1,550 m on the Troodos mountains. The average rainfall on the catchment is 490 mm/a. There are four automatic flow recorders on the catchment of this river and the average annual runoff estimated at the damsite is of the order of 21 million m^3/a. Flood inflow curves gave the 100 years flood as 600 m^3/s and the 1,000 years flood as 1,400 m^3/s.

Water rights downstream of Masari dam are as follows:

The Vathys concrete intake, starting at the damsite and leading to an earth channel, belongs to Katokopia and Argaki Irrigation Division and is mainly used by these two villages for irrigation.

The Katakrous and Zavrazis intakes belong to the Morphou Irrigation Division.

All these intakes exist between Masari and Morphou and they are all used for spate irrigation of a total area of about 9,710 donums. Diversion for irrigation has been calculated to be about 0.5 million m^3/a. The existing intakes will be fed from the outlet pipe with water stored in the dam except the Vathys intake which starts from the embankment.

In addition there were two old chain-of-wells traversing longitudinally the river valley in the alluvial deposits underneath the dambody. The Palloudheri chain-of-wells on the left abutment still in use at the time, had to be filled in with clay and replaced by a steel pipe of 0.30 m dia to discharge up to 0.30 m^3/s as water rights. The Masari chain-of-wells on the right abutment has been dry for a number of years and was therefore filled in with clay along its course under the dambody.

Since the Morphou dam has been built first, it has been decided to let this dam fill first before Masari Dam. Hence, during a flood, the outlet gallery of Masari dam will remain open until Morphou dam and Protopapas recharge basins downstream are filled. The tunnel will then be closed and water will start being stored in Masari dam. When the dam is full and there still is extra water then the tunnel gate would be opened thus allowing water into the river bed to recharge the underground acquifer between the two dams.

b. Geology

The dam has been built on Recent alluvial deposits, mainly of silty gravels of variable thickness ranging from 16 m at the right abutment to 50 m at the left abutment underlaid by a thin strata of limestone of variable degree of consolidation, to be succeeded by calcareous sandstone very often mixed with clay and shell fragments, belonging to the Athalassa formation.

4. MAIN FEATURES

a. Dam

This dam is a zoned earthfill, containing 278,300 m^3 of compacted fill material and a storage capacity of 2.27 million m^3. River sandy gravels have been used for the upstream and downstream shells, with an impervious thin clay core, in the middle. An upstream blanket with a key cut-off at its upstream end, and a downstream toe drain have also been provided. The upstream slope is covered with dumped rip-rap, 0.45—0.6 m in thickness, of large boulders obtained from the river abutments. The maximum structural height of the embankment is 16 m and its maximum crest length 1,100 m.

A free flow ogee crest spillway 60 m wide and 107 m in length is incorporated at the right abutment and has been designed to discharge 560 m^3/s, at full reservoir level. This is 40% of the 1,000 years flood and almost the entire 100 years flood. Next to the main concrete spillway is the emergency spillway which has been constructed from earth material as a breach section and has been designed to pass an additional 840 m^3/s being the balance of the 1000 years flood.

Below the embankment and at its maximum section there is a reinforced concrete conduit of 3.3 m internal diameter, which can pass 42 m^3/s when the water surface in the reservoir is at the normal water level A hydraulically operated gate controls the inlet to the conduit and has rectangular dimensions of 2.55x1.95 m. A 0.68 m dia outlet pipe is embedded below the conduit conveying water for irrigation.

b. Recharge Works

The main recharge works, downstream of the dam will be constructed at a later stage and will consist of the canalization of the main river bed and the construction of a number of speading basin on either side of the central river bed as far downstream as the Morphou dam.

5. CONSTRUCTION

The dam was constructed by the WDD during the period 21st of July 1971 to 31st of June 1973.

The embankment was constructed in two stages because it was necessary to allow for the free flow of the river during the winter period. The first 750 m stretch of the embankment was started in conjuction with the reinforced concrete spillway. The remaining stretch was completed during the summer of 1972 together with the reinforced concrete 3.3 m dia gallery.

All fill materials have been obtained from borrow areas in the river bed, apart from the earthfill for which the borrow area was situated approximately 2.5 km away from the site at the outskirts of Philia village.

For the compaction of the clay fill a pneumatic tyre roller has been used with a 10 ton weight. At places where the surface area to be compacted was too narrow for this type of equipment, a sheepfoot roller has been employed. All other fill materials have been compacted by a vibrating roller.

Compaction and proctor tests have been carried out frequently and the values obtained for the dry density were always well within the specified limits of 1,650 to 1,800 kg/m^3 dry density at 15—20% moisture content. Similarly, crushing strength tests, on concrete cubes obtained at the site, have been made by the WDD laboratory.

No special problems were encountered during the construction of this dam. On the other hand as no water has been retained in the reservoir yet, it has not become possible to evaluate the recharge effects on the acquifer or to observe any post-construction problems.

6. MANAGEMENT

This Project is managed by the Government. This has been found necessary in order to safeguard the water rights downstream belonging to many Irrigation Divisions. Also in this way it is more efficient to regulate the water from the dam either for recharge or for direct irrigation. The farmers are expected to pay towards the cost of the dam by being taxed on the water extracted from their boreholes which will be benefited.

Data

i. Catchment

Area	430 km^2
Average rainfall	490 mm/a
Average runoff	21 million m^3/a
1/1000 years flood	1,400 m^3/s
Maximum height	1,550 m
Maximum length	40 km

ii. Reservoir

Area	62 ha
Capacity	2.27 million m^3
Live storage	2.26 million m^3
Length	1,110 m

iii. Embankment

Structural height	16 m
Height above ground level	14.5 m
Hydraulic height	10.85 m
Depth of foundations cut-off	1.50 m (min.)
Freeboard	3.65 m
Crest length	1,000 m
Top thickness	4.26 m
Base thickness	62 m
Upstream slope	1:2.5 and 1:2.25
Downstream slope	1:2.0 and 1:1.8
Upstream core slope	1:0.2
Downstream core slope	1:0.2
Minimum core thickness	3 m
Maximum core thickness	8.5 m
Random fill	161,000 m^3
Clay core	85,000 m^3
Gravel, sand filter	27,000 m^3
Rock rip-rap	5,300 m^3
Total earth fill	278,300 m^3

iv. Excavations

Total	50,000 m^3

v. Spillway

Size	61 m wide
Capacity	560 m^3/s
Length	110 m
Concrete	3,640 m^3

vi. Gallery

Size	3.3 m dia
Capacity	42 m^3/s
Length	49 m
Concrete	1,100 m^3
Operating gate size	2.55x1.95 m

vii. Outlets (Steel pipes)

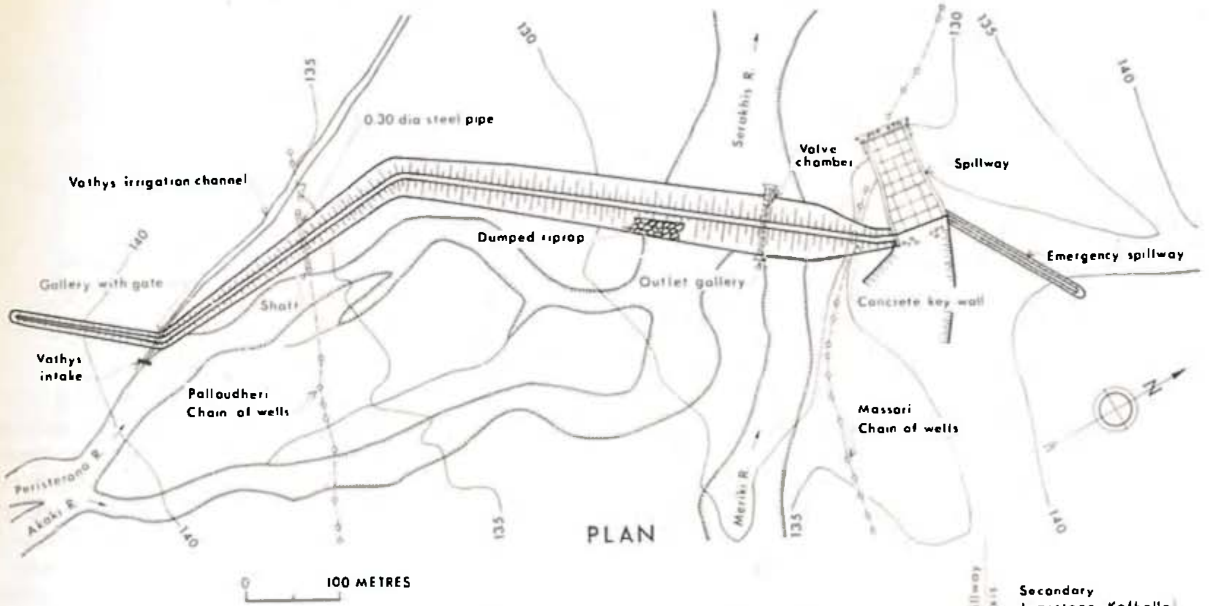
(i) Irrigation

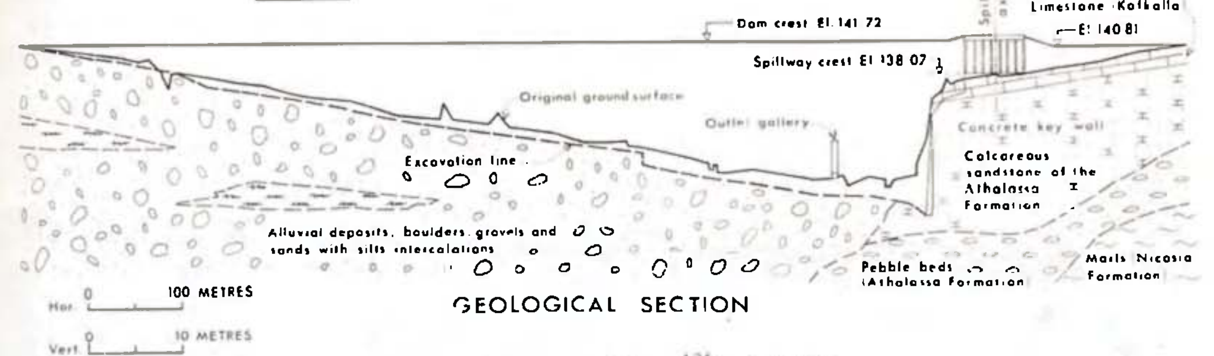
Size	0.68 m dia
Capacity	1.42 m^3/s
Length	81 m

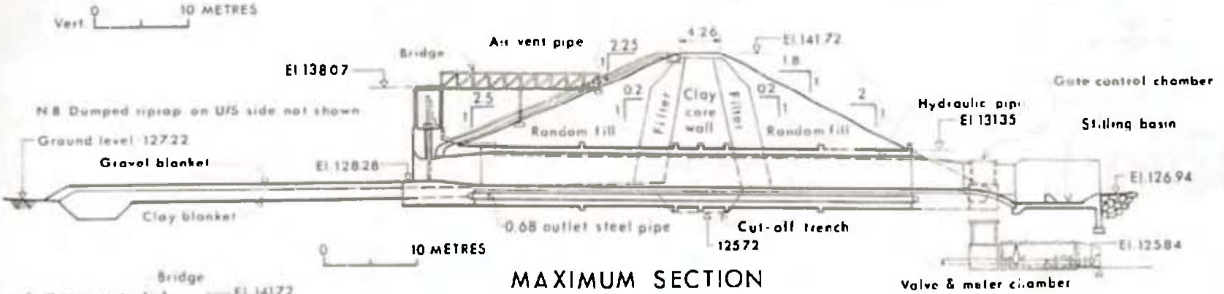
(ii) Compensation (Palloudheri)

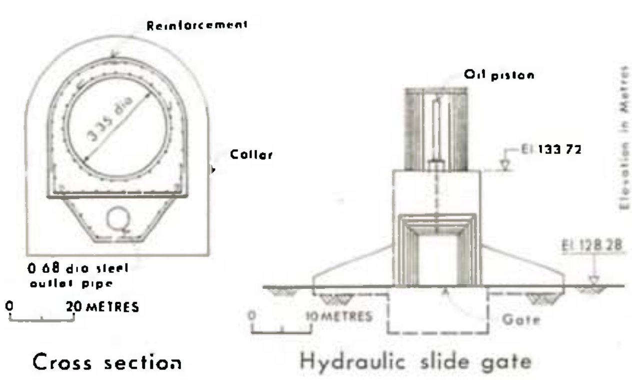
Size	0.30 m dia
Capacity	0.30 m^3/s
Length	50 m

SERAKHIS - MASARI DAM

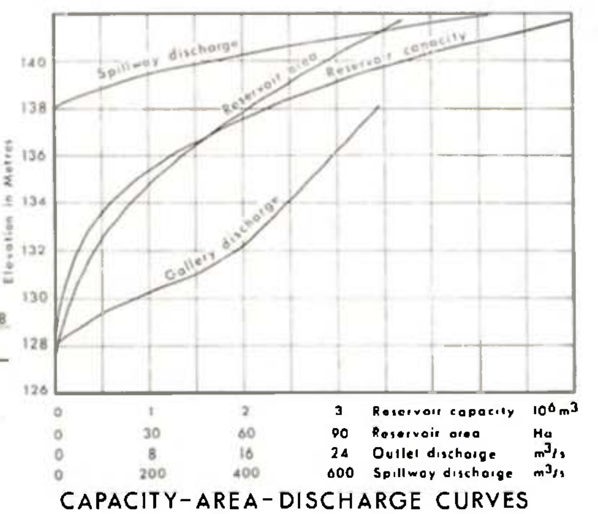








OUTLET GALLERY



CAPACITY-AREA-DISCHARGE CURVES

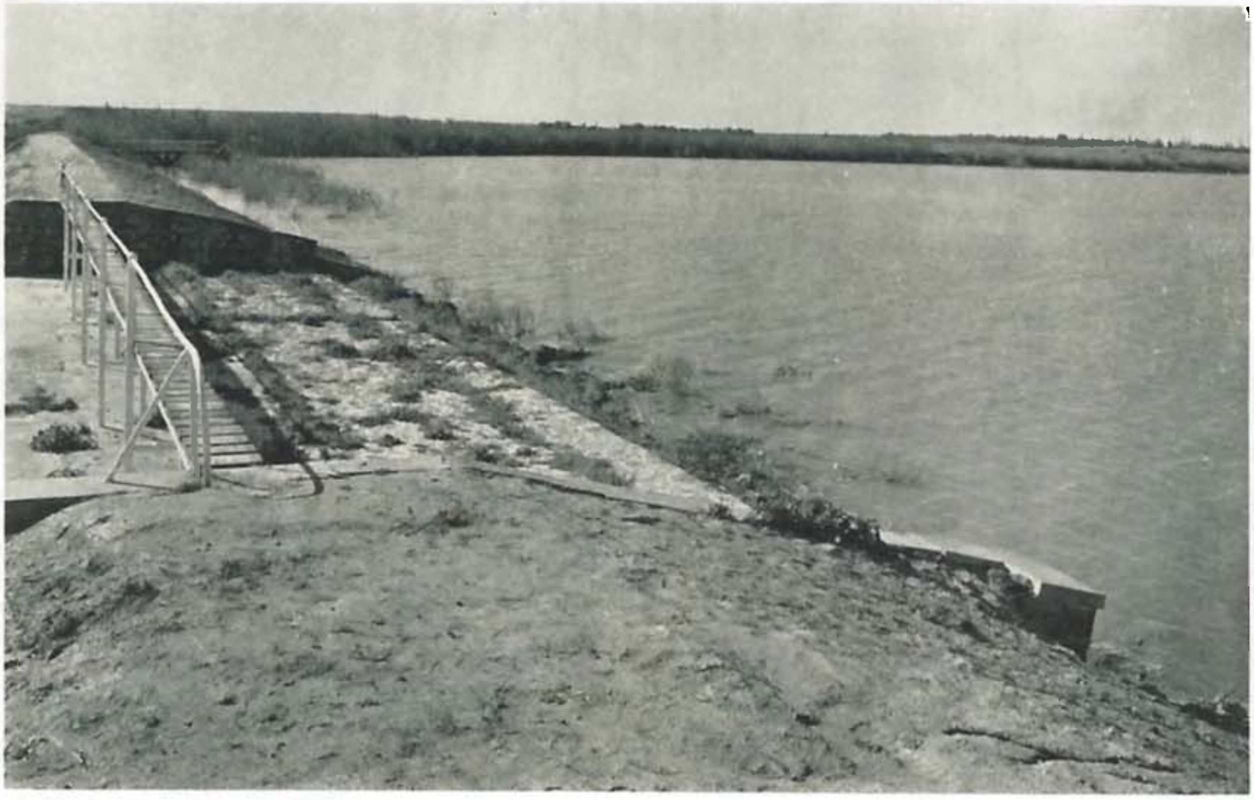
CHAPTER VI

GROUNDWATER RECHARGE

EARTHFILL DAMS

A great number of small dams have been built since 1960 on permeable rocks overlying the aquifers of Famagusta and Kyrenia, intercepting the flow of streams and thereby causing vertical and lateral infiltration through the reservoirs into the aquifer. Also the dams are used for storing the water which is later diverted by gravity or pumping to suitably located spreading grounds.

The reason for building recharge dams in these regions is the depletion of the groundwater resources.



The Kouklia reservoir.

1. FAMAGUSTA REGION

Famagusta aquifer is made up of four different aquifers bounded to the North by the road of Famagusta—Kondea, to the West by the road of Kondea—Dhekelia and to the South and East by the sea. This aquifer region as a total, is the second most important aquifer region of Cyprus. The first of the four aquifers in Famagusta which is by far the largest, is made up of sandstones of the Plio-pleistocene formation of appreciable thickness at Ayios Memnon and Famagusta where extensive pumping for the past 20 years has caused sea intrusion. The second important aquifer, is the reef limestone of the Middle Miocene which is phreatic and confined and is found around Cape Greco. The Gypsum aquifer of the lower Miocene age is the third important and stretches between Pergamos and Paralimni.

The fourth aquifer is made up of poor confined aquifers of chalks near Xylotymbou and North of Cape Greco.

The extraction from the whole aquifer in an average year is 53 million m^3 whereas the annual replenishment is only 22, thus leaving an annual deficiency of the order of 31 million m^3. Figure 8 shows the groundwater tables and the extent of sea intrusion in all coastal regions. The salinities in some parts are in excess of 1000 ppm NaCl and in some instances they reach 4,000 ppm. This region, 10 years ago, was the most important citrus growing part of Cyprus. There is still some 22,000 donums of different types of citrus

The Ayios Loucas reservoir on the right and the fresh water lake on the left.

making it the second most important citrus producing region in Cyprus after Morphou.

One of the reasons for the decline of the citrus plantations is the unreliable amount of water, and the high salinity. In sandy soils such as exist around Famagusta coast it is possible to irrigate citrus with water up to 1000 ppm NaCl provided leaching is carried out. However, the most important cropping is vegetables of various types, more particularly potatoes, Famagusta being the major potato producing region of Cyprus. A total of about 55,000 donums of vegetables, mainly potatoes, are grown in this region annually.

In view of the decline of the water resources of the aquifer, groundwater recharge works were decided to be carried out.

It is well known that in an unconfined aquifer like that of Famagusta, the best method of artificial recharge would have been by vertical recharge through reservoirs or surface spreading which would create a water table ridge. Such a ridge, of course, to be completely effective must be of sufficient height above sea level to repel sea water. In Famagusta such water level should be over 0.6 m above sea level, so that the fresh water—salt water interface would meet the impermeable Miocene which would act as a natural subsurface dam preventing the sea water intrusion. Further the water injection rate should be sufficient to hold the intrusion. However, due to the limited available water for recharge, the effort in Famagusta is not to eliminate completely the sea water intrusion but to minimize it and at least check its further inland movement.

Three regional recharge schemes were carried out in the Famagusta Region as follows:-

a. The Famagusta—Dherynia Recharge Scheme

In view of the limited land free of cultivation available in this region, there was little possibility for spreading grounds to be built and it was decided to depend mainly on an infiltration gallery which was excavated into the sandstone aquifer.

The first pilot recharge works were started in 1952 by driving a 20 m long infiltration tunnel 1x0.5 m at Ayios Memnon connecting six wells at a depth of 7 m asl and traversing the sandstone aquifer. Recharge experiments carried out through this tunnel in 1952 gave a rate of 2,000 m^3/day with a total replenishment of 300,000 m^3 in 1952.

In 1954 the main recharge works were initiated made up of the Ayios Loucas reservoir 0.70 million m^3 capacity, (receiving water from its own catchment and the surplus from Kouklia reservoir 4 million m^3 capacity, through a 16 km long channel, used mainly for storage) and an 1,400 m of recharge tunnel 1.2x0.9 m excavated from Ayios Loucas reservoir through the sandstone aquifer running parallel to the coast, the invert level of which was 1 m asl.

In 1956 additional recharge works were excavated consisting of an extension of the tunnel parallel to the coast to join Ayios Loucas and Ayios Memnon tunnels, of a total length of 7,200 m. The maximum rate of infiltration through the tunnel was found to vary between 6,000 m^3/day on the Ayios Loucas side to 10,000 m^3/day on the Paralimni side.

Later on in 1963 and in order to make available more water for infiltration through the tunnel additional storage was provided by constructing the Fresh Water Lake on the other side of the embankment of Ayios Loucas of 3.4 million m^3 capacity, a reservoir at Ayios Nicolaos of 0.4 million m^3 and the relevant diversion weirs and conveyors interconnecting these reservoirs. Also an antiflood and recharge dam was built outside Famagusta during the same period.

In 1964 a scheme was carried out, whereby the spare flows from the Plakos river coming from Kyrenia Hills were diverted into the Fresh Water Lake.

In the following years it became evident that in wet years when large quantities of water were available through the various systems of supply, the available potential for recharge was rather insufficient. The system as described above was found to be cabable for the recharge of about 15,000 m^3/day and it would take about 6 months to utilize all the water through the infiltrating tunnel taking also into account evaporation and transpiration losses.

During such a lengthy period, however, it was found that the water quality was deteriorating, becoming brackish especially in the Fresh Water Lake, because of brackish inflows, the salty soils in the reservoir and evaporation. Observations taken in 1965 showed that in 126 days the salinity in the Fresh Water Lake rose from 50 ppm NaCl to 440 ppm which is about the limit of allowable salinity for groundwater recharge.

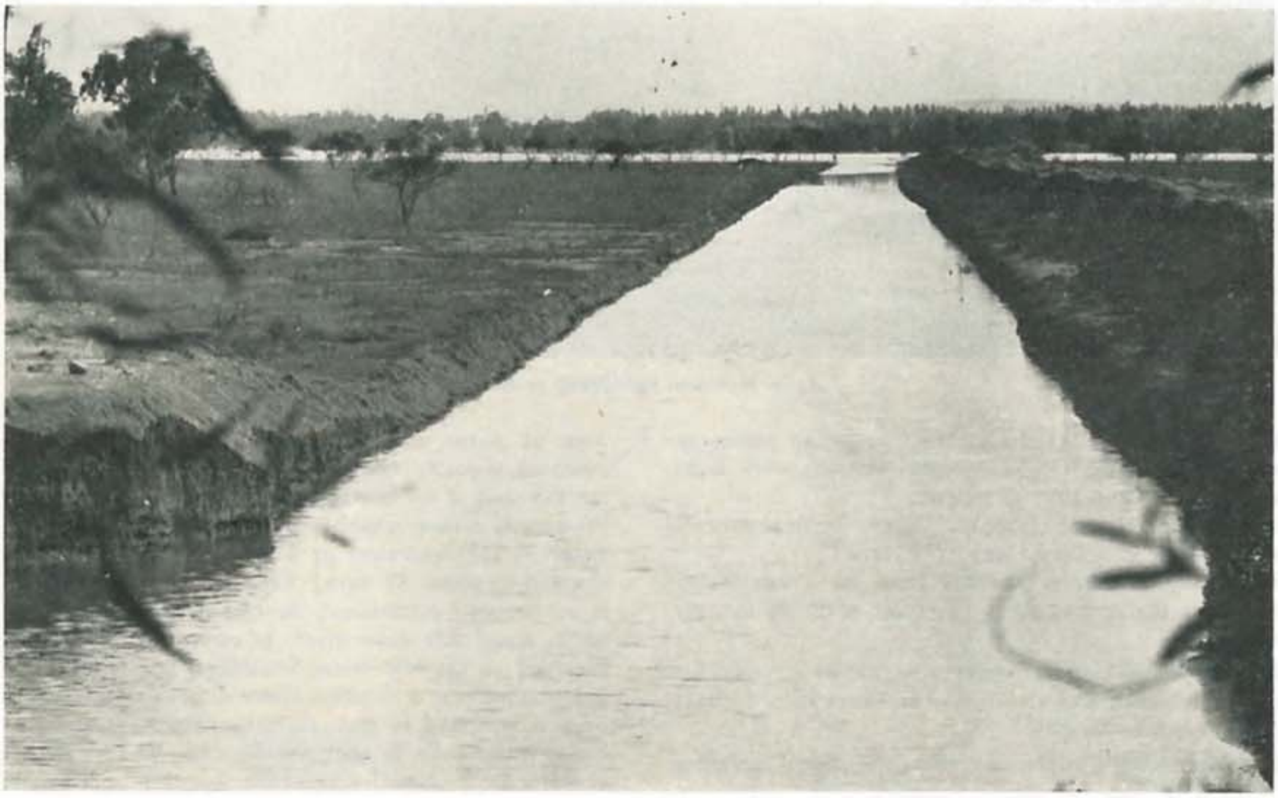
Furthermore the rate of infiltration through the tunnel was found to decline, with time, and a number of other problems arose with the operation of the tunnel for recharge as outlined below.

The main deficiency of the underground gallery method applied was the extent to which the sides of the porous sandstone clogged with particles of silt brought into the tunnel by the exfiltrating water, and although much of this silt was deposited in the Ayios Loucas Reservoir, it was found that large quantities found their way into the tunnel, thus causing clogging and a reduction in the infiltrating rate.

An inspection of the tunnel had revealed that there was silt in the tunnel base throughout, ranging from 75 mm to 100 mm, and the sides were also clogged by a thin membrane of silt. The tunnel was first desilted in 1963.

Another problem with the tunnel was the danger of polluting the aquifer. Of-course it is known that surface runoff water does not contain bacteria, but on its way such water may pass from polluted areas. When introducing this water unfiltered into the tunnel and if this water comes in direct contact with the groundwater then the groundwater will be polluted. This danger exists in Famagusta for it is known that at 1,500 m from the Ayios Loucas Reservoir and for a distance of 360 m the groundwater table in the gallery is up to 0.5 m above the invert of the gallery. In cases where the recharged water is not in direct contact with the groundwater the danger does not exist for the water is filtered along the depth of the aquifer and the bacteria are killed. A very long period of sedimentation in the reservoir would also help in killing the bacteria.

Another problem is that the horizontal permeability through the side of the gallery is much less than the vertical.



Main conveyor from Mutti-tis-Halis into the fresh water lake.

Finally, the gallery being underground, it is costly and difficult to inspect and maintain properly as an infiltration gallery.

In view of the problems described above, in connection with the infiltration tunnel and the deterioration of the quality of the water in storage with time, it was decided to build expensive spreading grounds at higher levels where land was available and pump the water there for recharge. This system, whereas expensive, yet would expedite the recharge through a more efficient system. Also the siting of the various spreading grounds was selected in such a way as to give a more uniform recharge through the aquifer. In addition the system is used for direct irrigation whenever water is required. Eight pumping stations were built between 1965 and 1970 as follows:-

i. At Ayios Nicolaos, supplying water to four spreading grounds at 6,000 m^3/day.

ii. At Ayios Loucas, pumping from the Fresh Water Lake into the Ayios Loucas Reservoir of 25,000 m^3/day capacity.

iii. At Ayios Loucas, pumping at 12,500 m^3/day for direct irrigation and also to the recharge dam built at Ayios Yeoryios locality.

iv. At one third of the way along the tunnel pumping at 2,500 m^3/day for direct irrigation and finally into a recharge dam.

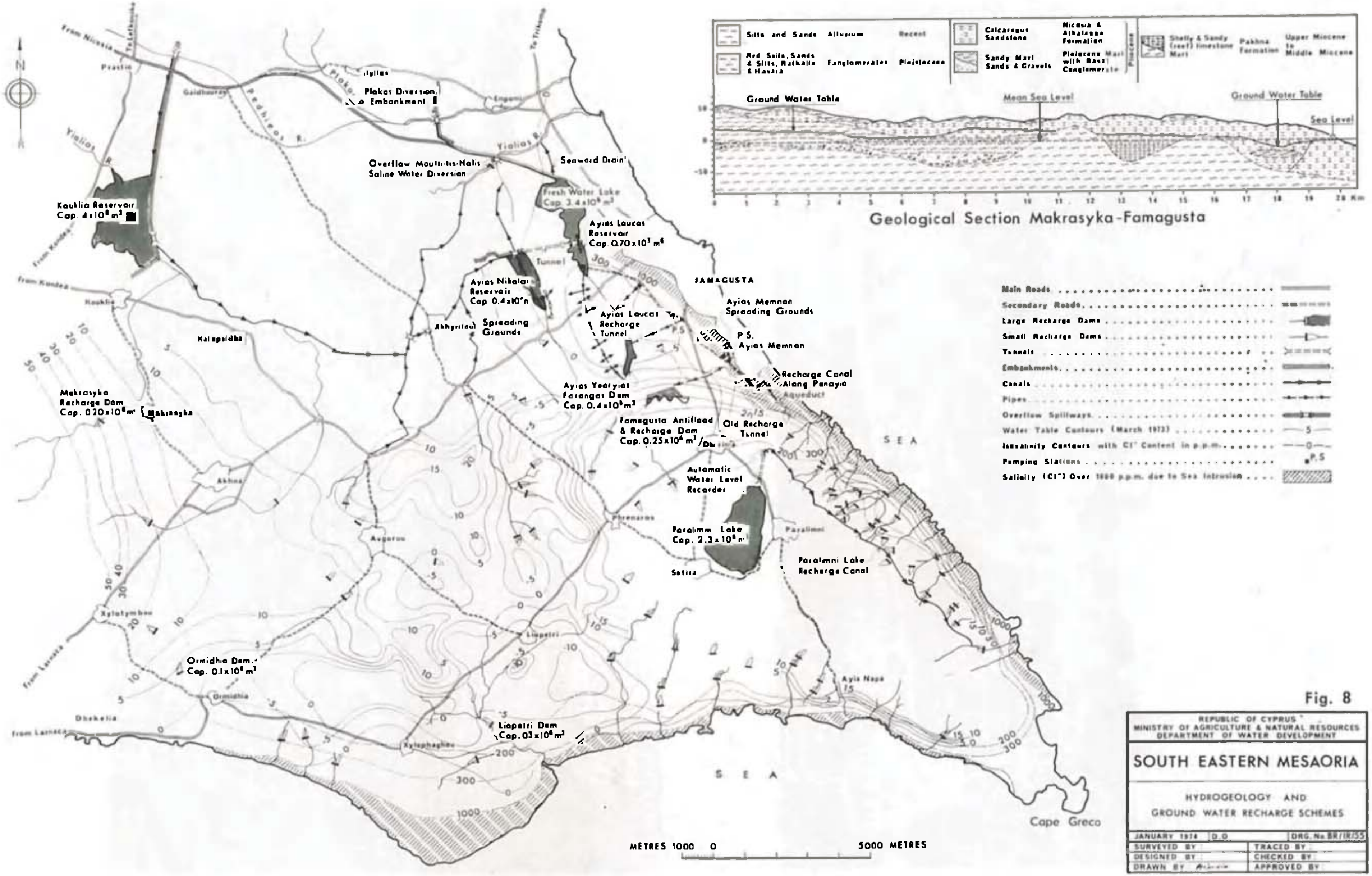
v. At two thirds of the way along the tunnel pumping at 4,500 m^3/day for direct irrigation and then into the Ayios Yeoryios recharge dam.

vi. & vii. At near the end of the tunnel, two pumping stations pumping at 7,000 m^3/day into spreading grounds at Ayios Memnon and into the antiflood-recharge dam near Famagusta.

viii. The last pumping station of capacity 4,500 m^3/day right at the end of the tunnel pumps water into the old part of the tunnel and into a recharge canal excavated in the sandstones.

Excluding the pumping from the Fresh Water Lake to the Ayios Loucas Reservoir which is only a conveyance of water within the same storage system, the pumping capacity for recharge provided is 37,000 m^3/day and if we add the 15,000 m^3/day capacity of the tunnel system when properly maintained, then the water in the reservoirs of 4.5 million m^3 can be recharged in about 87 days. Even if the tunnel system is not properly maintained, the reservoirs can be emptied in about 100 days which is considered reasonable provided no brackish water is allowed into the system. For this purpose a brackish water diversion scheme has been constructed at Mutti-tis-Halis before entering the Fresh Water Lake. In addition, there may be 4 million m^3 of water in the Kouklia Reservoir, but this is primarily used for local irrigation in the Mesaoria villages.

Ayios Memnon spreading grounds.


Geological Section Makrasyka-Famagusta
Ground Water Table
Mean Sea Level
Sea Level
Silts and Sands Alluvium Recent
Red Soils, Sands & Silts, Rafkalla & Havara Fanglomerates Pleistocene
Calcareous Sandstone
Sandy Marl Sands & Gravels
Nicosia & Athalassa Formation
Pliocene Marl with Basal Conglomerate
Pliocene
Shelly & Sandy (reef) limestone Marl
Pakhna Formation
Upper Miocene to Middle Miocene
Main Roads
Secondary Roads
Large Recharge Dams
Small Recharge Dams
Tunnels
Embankments
Canals
Pipes
Overflow Spillways
Water Table Contours (March 1973)
Isosalinity Contours with Cl⁻ Content in p.p.m.
Pumping Stations
Salinity (Cl⁻) Over 1000 p.p.m. due to Sea Intrusion
Kouklia Reservoir Cap. 4×10⁶ m³
Plakas Diversion Embankment
Overflow Mouttis-tis-Halis Saline Water Diversion
Seaward Drain
Fresh Water Lake Cap. 3.4×10⁶ m³
Ayios Loucas Reservoir Cap. 0.70×10³ m⁶
Ayios Nikolaos Reservoir Cap 0.4×10⁶m
Spreading Grounds
Ayios Loucas Recharge Tunnel
FAMAGUSTA
Ayios Memnon Spreading Grounds
P.S. Ayios Memnon
Recharge Canal Along Panayia
Aqueduct
Ayios Yeoryios Farangas Dam Cap. 0.4×10⁶m³
Famagusta Antiflood & Recharge Dam Cap. 0.25×10⁶ m³
Old Recharge Tunnel
Automatic Water Level Recorder
Paralimni Lake Cap. 2.3×10⁶ m³
Paralimni Lake Recharge Canal
Makrasyka Recharge Dam Cap. 0.20×10⁶ m³
Makrasyka
Ormidhia Dam Cap. 0.1×10⁶ m³
Liopetri Dam Cap. 0.3×10⁶ m³
SEA
Cape Greco
METRES 1000 0 5000 METRES
From Nicosia
From Kondea
From Larnaca
Yialias R.
Pedhieos R.
Kouklia
Kalopsidha
Akhna
Avgorou
Phrenaros
Sotira
Paralimni
Liopetri
Xylotymbou
Ormidhia
Dhekelia
Xylophaghou
Ayia Napa
Akhyritou
Engomi
Prastio
Fig. 8
REPUBLIC OF CYPRUS
MINISTRY OF AGRICULTURE & NATURAL RESOURCES
DEPARTMENT OF WATER DEVELOPMENT
SOUTH EASTERN MESAORIA
HYDROGEOLOGY AND
GROUND WATER RECHARGE SCHEMES
JANUARY 1974
D.O.
DRG. No BR/IR/55
SURVEYED BY
TRACED BY
DESIGNED BY
CHECKED BY
DRAWN BY
APPROVED BY

The Paralimni lake recharge conveyor canal.

The Panayia recharge dam at Paralimni.

The total cost on the Famagusta-Dhekelia Recharge Works reached up to £250,000.

The benefits derived are difficult to assess, but an indication can be given from the amount of water recharged until 1973 and the effect on the water tables and salinity.

A total of about 12 million m^3 of water was recharged during the period between 1952 and 1973 which means an average of about 1 million m^3 annualy.

Considering amortization at 7% rate of interest over 40 years life, replacement of pumps once and including operation and maintenance cost the unit cost of water is estimated to be about 25 mils/m^3.

As this is the cost of the recharged water, we have to allow for losses in the aquifer and for pumping costs to recover the water. If we allow an 80% recovery and 5 mils/m^3 pumping costs, the total price of the water supplied from the recharge works would be of the order of 35 mils/m^3. This is considered reasonable by Cyprus standards and especially in view of the importance of supplying water to Famagusta in order to save existing citrus plantations and to maintain domestic water supplies.

Numerous observations on groundwater tables are taken regularly, which show the immediate effect on the levels and on the quality where sufficient water enters the aquifer.

b. The Paralimni Recharge Scheme

In this coastal sandstone aquifer the over-extraction has also caused a decline of the water table and limited sea intrusion with a consequential salinity of the groundwater near the coast. It is traversed by 13 small streams running to the sea or to the Paralimni Lake with a total flow of about 2 million m^3/a.

The extraction from the aquifer is about 1.5 million m^3/a used for the irrigation mainly of early summer vegetables. The main problem of over-extraction is that the aquifer is of very little storage potential being mostly in a dynamic state, so that in wet years there is no problem of overpumping. However, water from wet years and the recharge works carried out can be accommodated in the aquifer if the frequency of occurrence is not more than once every two years. Actually in this area it is very rare to have consecutive wet years.

For the above reasons, recharge works have been carried out consisting of a major storage at Paralimni Lake of about 2.3 million m^3. of 35 small recharge dams situated at suitable sites along the stream valleys and of a conveyor recharge canal 11 km length, gradient 1/1000, which conveys water from the lake to the recharge dams and also infiltrates water along its length throughout its unlined section 3x4 m excavated on the calcarenite. The rate of infiltration along the canal has been estimated to be about 5,000 m^3/day. Water can also be supplied from the recharge dams into the canal as may be required. The total storage capacity of the 35 recharge dams is about 0.20 million m^3. The works were carried out in 1963 at a cost of £34,000 and until 1973 they received water three times of a total quantity of 4.5 million m^3.

Through observations taken, the water infiltrated rapidly through the reservoir beds and the canal, in a matter of days, and the result on the water tables was almost immediate. One serious problem regarding this scheme is the possible rise of the salinity of the water in storage in the Paralimni Lake, and for this purpose any water that is received in the lake has to be quickly utilized.

A saline water diversion weir has been provided at the entry of the canal for disposing any saline flows. The salinity in the lake emanates from the connate salts contained in the clay bed of the lake and increases with evaporation.

c. Kokkinokhoria Recharge Scheme

This is the third groundwater recharge scheme in the Famagusta Region and is made up of small recharge dams wherever hydrogeological conditions allowed.

Sixtythree small dams of 1.50 million m^3 capacity covering 13 villages on the Famagusta aquifer have been constructed in the last 10 years at a total cost of £130,000.

The over-extraction on this large part of the aquifer is at its greatest, and the groundwater in storage has disappeared in most parts. It is fortunate that the irrigation practiced in this region is about 90% vegetables and more particularly spring potatoes which largely rely on rainfall and groundwater pumpage from the annual replenishment. The methods of irrigation here are very efficient being sprinkler throughout.

A list of the recharge dams buit in this region is given in Table 17.

TABLE 17

Village	Number of Dams	Storage Million m^3
Phrenaros	12	0.10
Ayia Napa	12	0.10
Sotira	3	0.05
Dherynia	4	0.05
Ormidhia	2	0.15
Xylophaghou	5	0.10
Avgorou	9	0.10
Xylotymbou	4	0.05
Akhyritou	2	0.15
Makrasyka	2	0.20
Akhna	6	0.25
Kondea	1	0.10
Lysi	1	0.10
Totals	63	1.50

The observations taken both on the rate of infiltration through the reservoir beds and the effect on the groundwater tables have shown satisfactory results. Wherever necessary, water is released downstream for regulated spreading downstream of the dams.

Of great importance is the clogging of the reservoir beds by silt, which requires regular desilting. This is easily done in all cases because of the flat topography and of the good accessibility for tractor work.

Recharge dam at Makrasyka.

Ayia Napa typical recharge dam. An old aqueduct can be seen in the backround.

2. NORTH KYRENIA COASTAL REGION

This aquifer of Pleistocene deposits extends along the Northern coast and is approximately 4 km wide. It overlies the Miocene Kythrea formation which is next to the Lapithos Chalks and the Hilarion Limestone. All these formations are to be met within 4—6 km from the coast. The Pleistocene Formation is made of terraces of fossiliferous porous calcarenite and of gravel and sand beds. The depth of the aquifer is about 30 m and a limited amount of water is met all over.

A great number of wells and boreholes have been drilled mainly during the past 20 years and the pumpage has been increasing annually due to the needs for lemon plantations which constitute the main crop of the area and because of increasing domestic requirements. In many parts of the aquifer in view of the overpumpage, considerable depletion of the water table has been observed and near the coast the water is usually brackish. This aquifer, although large, is in a similarly dynamic state as that of Paralimni described previously and it can be recharged and overfilled in wet years. However, the frequent dry years occurrence is a permanent characteristic of Cyprus conditions and therefore a decision was taken to go ahead with the construction of groundwater recharge dams on the numerous small streams of the region similar to those built in Famagusta.

An intensive programme started in 1965 and until the present time 32 small dams have been constructed of a total capacity of 0.45 million m^3, as is given in Table 18.

TABLE 18

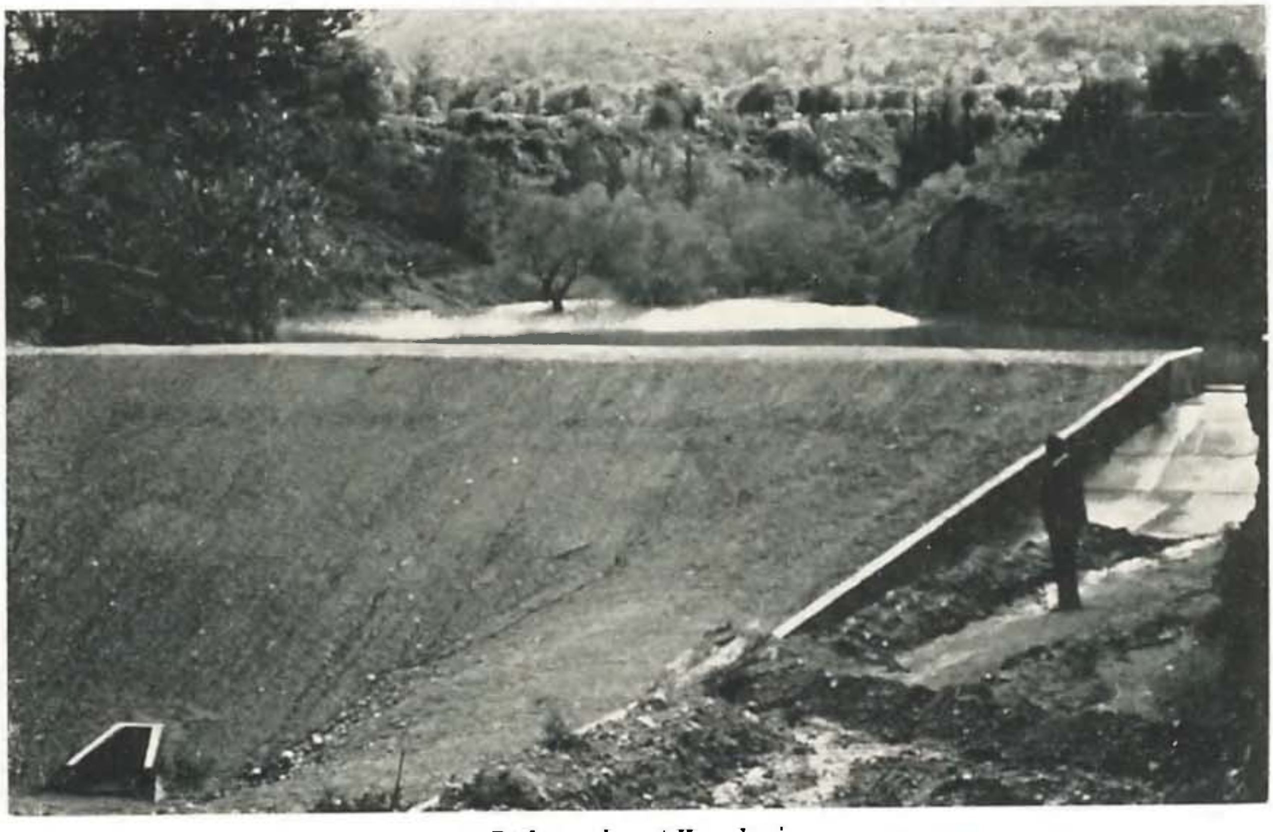
Village	Number of dams	Storage Million m^3
Elea	2	0.03
Ay. Yeoryios)		
Trimithi)	8	0.10
Karmi)		
Thermia	1	0.02
Karakoumi	1	0.02
Kazaphani	4	0.08
Ay. Epiktitos	7	0.10
Akanthou	9	0.10
Total	32	0.45

The rate of infiltration and effect on the aquifer are equally good as at Paralimni, but again here the main problem is that of desilting the dams, which can easily be done due to the favourable topographic conditions and accessibility.

The cost of the dams constructed to date has been £43,000.

Akanthou typical recharge dam

Recharge dam at Karakoumi.



Recharge dam at Kazaphani.

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