Engineering geology of dam foundations in north - Western Greece

Sotiris A. Papageorgiou

ENGINEERING GEOLOGY OF DAM FOUNDATIONS IN NORTH - WESTERN GREECE

by

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MAIN VOLUME

WALLS

Without consideration, without pity, without shame
they have built big and high walls around me.

And now I sit here despairing.
I think of nothing else: this fate gnaws at my mind;

for I had many things to do outside.
Ah why didn't I observe them when they were building the walls?

But I never heard the noise or the sound of the builders.
Imperceptibly they shut me out of the world.

AS MUCH AS YOU CAN

And if you cannot make your life as you want it,
at least try this
as much as you can: do not disgrace it
in the crowding contact with the world,
in the many movements and all the talk.

Do not disgrace it by taking it,
dragging it around often and exposing it
to the daily folly
of relationships and associations,
till it becomes like an alien burdensome life.

ITHACA

When you start on your journey to Ithaca,
then pray that the road is long,
full of adventure, full of knowledge.
Do not fear the Lestrygonians
and the Cyclopes and the angry Poseidon.
You will never meet such as these on your path,
if your thoughts remain lofty, if a fine
emotion touches your body and your spirit.
You will never meet the Lestrygonians,
the Cyclopes and the fierce Poseidon,
if you do not carry them within your soul,
if your soul does not raise them up before you.

Then pray that the road is long.
That the summer mornings are many,
that you will enter ports seen for the first time
with such pleasure, with such joy!
Stop at Phoenician markets,
and purchase fine merchandise,
mother-of-pearl and corals, amber and ebony,
and pleasurable perfumes of all kinds,
buy as many pleasurable perfumes as you can;
visit hosts of Egyptian cities,
to learn and learn from those who have knowledge.

Always keep Ithaca fixed in your mind.
To arrive there is your ultimate goal.
But do not hurry the voyage at all.
It is better to let it last for long years;
and even to anchor at the isle when you are old,
rich with all that you have gained on the way,
not expecting that Ithaca will offer you riches.

Ithaca has given you the beautiful voyage.
Without her you would never have taken the road.
But she has nothing more to give you.

And if you find her poor, Ithaca has not defrauded you.
With the great wisdom you have gained, with so much experience,
you must surely have understood by then what Ithacas mean.

Poems of C.P.CAVAFY

(1863 - 1933)

To the Memory of my Mother

Foreword

This thesis describes a programme of research that was conducted both at Durham University, England and in the field in Greece.

The author has been responsible for the geological investigations at the three dam sites which form the basis of the thesis and he has also been responsible for planning and supervising the foundation grouting programmes for the Assomata and Sfikia dams.

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ABSTRACT

The significance of the geological factors which controlled the formation of damsites, and the relationship between these factors and the soundness of foundation bedrock (mainly from the watertightness point of view) are studied in this work. The need to incorporate geotechnical design implications, in dam foundation studies, besides the geological descriptions which are involved in current classification schemes, is considered. The void volumes in the rock foundations of earth and rockfill dams, in three different geological environments in Greece, are assessed by the grout volumes spent in sealing off the dam rock foundations, and expressed as distinct grouting parameters. A prior assessement of the geological and the geotechnical records (up to the design stage) was made to isolate the geological factors governing the watertightness (permeability) characteristics of the three different foundation bedrocks. It was found that the dimensionless parameter "grout take per metre run per area of segment", as used in this study, has particular attractions for assessing the pre-impounding watertightness of dam foundations. The relations of the bedrock discontinuity density distributions (as main grout absorbing elements) with the major geological factors involved in each site are discussed and their implications to the grouting results are pointed out. The geological environments of the Greek dams and

the main geological factors pertaining to dam incidents have also been examined for comparison purposes, and for determining the degree of correlation for possible practical use in the design of future damsites.

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CHAPTER 1

Introduction

1.1 Historical background - some aspects of dam engineering and associated problems.

Water resources development and water regulation projects are some of the foremost concerns of modern times. Dams and their reservoirs are the most important structures for water reallocation and exploitation and this largely affects the growth of modern human society. The energy crisis of the last decade forced countries into harnessing their natural local resources to exploit the potential of hydroelectricity. Greece has much to do in this sector because she has developed only 14 per cent of her hydroelectric potential (this is indicated in the later Table 2.4).

Among the earliest recorded dams was the embankment dam Sadd-El-Kalara in Egypt, near Cairo, constructed between 2950 and 2750 B.C. (Smith, 1971). Apart from other ancient embankment-type dams reported elsewhere in the world (India, China, and other ancient civilisations, Khasla ,1968; Mermel, 1976), arch dams have been constructed since Roman and Byzantine times (Schnitter, 1976). In the middle of the 19th century rockfill dams were constructed in the western United States, first by gold miners and later, at the beginning of the 20th century, hydraulic fill dams were constructed.

Due to world industrialization, the demand for water increased and the development and construction of dams in the last two centuries increased rapidly. Data from the World Register of Dams, produced by the International Commission on Large Dams (ICOLD), can easily support the conservative estimate that two dams per day are being added to the planet's surface to enhance its resources (Mermel, 1976). More than 12,000 large dams are identified in more than 70 member countries.

Dams range in size and complexity of construction from low earth embankments or concrete structures, constructed to impound or divert water and wastes, to massive earth or concrete structures (see Tables 1.1 and 1.2) across major rivers to store water for irrigation, municipal use, hydro-electric power generation, or flood prevention.

As in the case of all engineering structures, dam foundations must meet certain requirements, particularly because the vast volumes of water in the reservoirs pose a constant threat to those who live downstream, and because they are highly capital-intensive schemes. Requirements for the foundation are summarised as follows:

a) It must have the proper watertightness or drainage characteristics for its function.
b) It must have sufficient strength to support the loads which will be imposed by the superstructure and those of the impounding water or other dynamic loads.
c) It must permanently retain its watertightness (or controlled drainage) characteristics and its strength.

The acceleration in dam construction during the last decade has exhausted the most favourable natural dam sites; hence foundation treatment to improve new sites becomes imperative in order to

TABLE 1.1

World's highest dams

(after Mermel, 1976)

Name	Country	Dam Type	Height (m)
Rogunsky	U.S.S.R	TE	325
Nurek	U.S.S.R	TE	317
Grand Divence	Switzerland	PG	285
Inguri	U.S.S.R	VA	272
Vaiont	Italy	VA	262
Mica	Canada	ER	242
Sayanskaya	U.S.S.R	VA	242
Chicoasen	Mexico	ER	240
Patia	Colombia	ER	240
Chivor	Colombia	ER	237
Mauvoisin	Switzerland	VA	237
Oroville	U.S.A	TE	235
Chirkey	U.S.S.R	VA	233
Bhakra	India	PG	226
Hoover	U.S.A	VA/PG	221
Contra	Switzerland	VA	220
Mratinze	Yugoslavia	VA	220
Dworshak	U.S.A	PG	219
Glen Canyon	U.S.A	VA	216
Daniel Johnson	Canada	MV	214
Toktogul	U.S.S.R	PG	213
Auburn	U.S.A	VA	209
Luzzone	Switzerland	VA	209
Keban	Turkey	TE/ER/PG	207

UC = under construction, TE= earth, ER = Rockfill, PG = gravity, VA = Arch, MV = Multi Arch

TABLE 1.2

World's largest dams

(after Mermel, 1976)

Name	Country	Dam Volume ($m^3 \times 10^6$)
New Cornelia Tailings	U.S.A	209,500
Tarbela	Pakistan	142,000
Fort Peck	U.S.A	96,034
Dahe	U.S.A	70,343
Oosterscheld	Netherlands	70,000
Rogunsky	U.S.S.R	70,000
Yacyreta-Apipe	Argentina Paraguay	70,000
Mangla	Pakistan	65,651
Cardiner	Canada	65,553
Afsluitdizk	Netherlands	63,400
Oroville	U.S.A	59,639
San Luis	U.S.A	59,378
Nurek	U.S.S.R	58,000
Garrison	U.S.A	50,846
Cochiti	U.S.A	49,417
Tabka	Syria	46,000
Kiev	U.S.S.R	44,000
W.A.C.Bennett (Portage Mt)	Canada	43,733
High Aswan	Egypt	43,733
Saratov	U.S.S.R	40,400
Mission Tailings No 2	U.S.A	40,088
Fort Randall	U.S.A	38,383
Kanev	U.S.S.R	37,860
Itymbiava	Brazil	36,800
Kakhova	U.S.S.R	35,640

meet the above requirements and to minimize the risks involved. The closest cooperation of engineers and geologists as well as other experts is needed in the planning, design, construction and maintenance, so as to ensure a maximum degree of safety and the minimum possible damage to the environment.

Among the records of dam building, referred to above, exist dam failures which average two per year, on a world wide scale (Germord, 1977). Many of the causes of dam failures are attributed to poor foundations arising from geological deficiencies (see Section 2.2.1).

In Greece, although the number of dams is small, there have been several dam incidents (and one failure) specifically attributable to foundation deficiencies (see Chapter 2).

The satisfactory performance of dam foundations depends upon the extent to which certain defects can be envisaged and possibly remedied. The engineering geological-geotechnical studies which will be conducted for a particular site cannot always be translated into useful design parameters. Despite all the advances in the theoretical,laboratory and field investigations concerned with the mechanical and hydraulic properties of the foundation rock,dam construction precedents have clearly shown that each damsite and its dam are a unique case as a direct result of the multiplicity of factors involved and their interactions.

Thus, in evaluating a number of possible alternatives for the appropriate foundation, Casagrande's notion of "calculated risk" should be a guiding consideration in the decision-making process:

a) "The use of imperfect knowledge, guided by judgment and experience, to estimate the probable ranges for all pertinent quantities that enter into the solution of a problem."

b) "The decision on an appropriate margin of safety or risk, taking into consideration economic factors and the magnitude of losses that would result from failure" (Casagrande, 1965).

Foundation improvement methods vary according to the nature of the ground (soil or rock), the nature and function of the superstructure, the duration (permanent or temporary) of the superstructure, the duration (permament or temporary)of the improvement and, of course, the economics of the project, consistent with the safety factors adopted for the design.

In the case of dam foundations on rock, the construction of a grout curtain into the riverbed and the abutments, as a continuation of the impervious element of a dam, is a world-wide practice. It consists of drilling holes into the rock and pumping cement-based grouts into the defective rock to substantially prevent leakage and to eliminate detrimental effects which water circulation can exert downstream. The accomplishment of the grouting works, by a well-designed drainage curtain downstream of the grout curtain, widens the requisite safety margins for the hydro project and guarantees the substantially permanent operational efficiency of the project in terms of allowable leakages, non-erosion and stability.

Prior to construction all means such as site investigations and field testing, laboratory testing and theoretical analyses are used to simulate the foundation conditions and to specify the extent of the improvement works for ensuring that the foundation treatment will be adequate and economic.

During the grouting stage of construction, technical features such as the number and location of boreholes drilled, together with the volume of grout used, serve two purposes: firstly to correct specific geological defects; secondly, to define more precisely the character of those defects. These defects may arise from the following factors: the lithology and the results of geotectonic stresses in forming structural weaknesses, weathering processes, and other alteration processes. These structural characteristics may be clearly visible at surface level but their geotechnical significance may only become apparent from exploratory investigations at depth. Thus, the grouting stage of construction can be used both to confirm the accuracy of existing information about the foundation conditions and also to expose hitherto undetected features. Consequently, further corrective measures and different procedures for collecting and evaluating information may result from the grouting stage of construction. The importance of grouting as a source of vital information with respect to economics and safety is illustrated by the following examples.

One outstanding example, among others, of the extent of foundation improvements is the grout curtain of the 117m high, Docan arch dam in Iraq (Binnie et al,1959). Here, a 24km curtain was constructed. Its final area was 450,000 m^2. The total length of drilling for rock sealing and consolidation was 300,000 m and the injected cement and sand was 116,000 tonnes weight.

On the negative side, catastrophic dam failures of the Teton dam, U.S.A and the Malpasset dam in France are both attributable to geological causes (Penman,1977; Thomas, 1976). It may be argued that an informed appreciation of the foundation conditions and modification of specifications might well have prevented such massive failures.

1.2. Purpose and scope of the present study

The main purpose of this study is to provide a guide to the significance of certain geological factors which influence the watertightness of a given damsite in a given geological environment, using simply the grouting data and the geometry of the grout curtain. Because grouting operations for the creation of a cut-off curtain comprise a metre-by-metre probing of defects in the rock continuity, such grouting results provide the data for assessing the void volume of foundation bedrock and assist in grading the importance of the several geological and geotechnical records and analyses for the design of the curtain. The results also clarify some of the obscurities in the appraisals that have previously existed. Correct interpretation of the results can guide the engineer towards such additional corrective measures as are required to achieve the requisite standards of foundation performance and also provide guidance on the important geotechnical factors which have to be considered in damsites of similar geological environment, saving time and money and allowing the best choice of alternatives.

Chapter 2 reviews the prospects of dam construction in Greece and also considers post-construction incidents that have occurred in Greek dams. These dams are assessed in a global dam-construction setting. The engineering geological problems relevant to dam foundations in Greece are outlined and the need for more technically informed appraisal of the foundation conditions of damsites for future development is pointed out.

In Chapter 3, the overall geological environment in which Greek dams have been built, or are going to be built, is examined. The complexities and the diversities of the geology of Greece in a regional sense are briefly outlined. More locally, the geological characteristics conditioning the foundation environments of Greek dams are pointed out and the potential risks noted.

Chapter 4 is concerned with a description of the field work which forms the basis of this thesis. It examines the foundation conditions at three new damsites, the results of various tests conducted at the sites prior to bedrock grouting, and the grouting results from the three different dam foundations. One dam is now complete and has been in service since 1981. The other two dams are still under construction and due to start generating power during the second half of 1984.

The foundation bedrock of the three dams each belongs to different formations in three different geotectonic (isopic) zones of Greece, but the drilling methods and the grouts used to construct the respective grout curtains were basically similar.

This latter feature provided a good background from which to compare the different geological factors involved in the water-tightness of the foundations of each site.It is generally the case that grout "takes" received by the foundation bedrock are controlled by the following factors:

a) the structural defects (voids, discontinuities), their form (particularly orientation) and degree of inter-connection, and their infillings;

b) the degree of rock weathering;

c) the degree of relaxation (dilation due to geotectonic stress relief);

d) the degree of stress relaxation, re-distribution and damage caused by excavation, blasting, tunnel driving and other associated activities of dam building.

Because of the multiplicity of the factors involved and the need to use comparable information from one curtain for application to the other curtains, the geometrical characteristics of each grout curtain have been "divided" vertically into basic segmental areas of 50m wide multiplied by the depth of the curtain in each segment. The quantities of the grout injected into each hole and into each segment are then considered to be clearly-defined grouting parameters which can be related to the different geological conditions which facilitate grout absorption.

The particular types of grout used for injection are noted and the injection pressures are discussed with respect to the limitations which a given rock imposes on grout takes, and vice versa (see Appendix C).

Thus, Chapter 4 in the thesis is designed to provide answers to the following questions:

1. What is the effect of different lithological units (stratigraphy) within a site upon the grout takes?
2. What is the effect of the different structural defects on the grout takes, and what are their form and nature?
3. How does rock weathering influence the groutability of the foundation bedrock and does it pose any threat to the foundation watertightness?

4. What is the effect of relaxation on grout takes due to stress relief produced by tectonic phenomena and by engineering construction?
5. What is the inter-relation of the above parameters?
6. To what extent do the results of grouting activity confirm the assumptions upon which the initial design specifications were drawn up? Where these results show a clear deviation in practice from the initial expectations projected by the specifications, do the findings assist the designers in meeting any new safety tolerances which will then possibly be required?
7. What is the relationship to the regional geology between the results of these grouting works and the previous information gathered on the site?
   Is it possible to produce data on engineering geological zoning for use in comparable prospective damsites in different areas of Greece?

Finally, in Chapter 5, the conclusions and practical implications of the previous chapters will be outlined and some recommendations for future action will be made.

CHAPTER 2

A review of dam construction prospects in Greece

2.1 Some aspects of the current energy problem in Greece and the prospects for hydroelectric development

2.1.1 General

Greece is located in the southern-most part of the Balkan peninsula in south-eastern Europe (Fig.2.1). It covers an area of 134 000 km^2 (50 944 $miles^2$) with a population of ten million people. The gross national product in 1978 had reached 1 182 billion drachmas (32 000 million dollars) and the annual *per capita* income in 1979 was 3 500 dollars (Financial Times, Sept. 1979; Simpson, 1979). After a fall of 1.5 per cent in 1981 the Greek gross national product remained stationary in 1982 (Collmer, 1983).

Productivity in the non-agricultural sector grew at a rate of more than 3 per cent in 1975-1978 while the years after that showed a decline affected by the world's economic recession of the last decade. During the decade 1964-1974, the annual average rate of growth was 4.75 per cent (Yannopoulos, 1979).

In addition to a limited presence of heavy industry, the major sectors of the economy comprise agriculture, light and medium industry, tourism, trade, and other minor contributions.

With an average *per capita* income that is 60 per cent of the average mean annual *per capita* income of the EEC countries, Greece is the 10th member of the community since 1981. Because of its close links - political, economic and social - with the

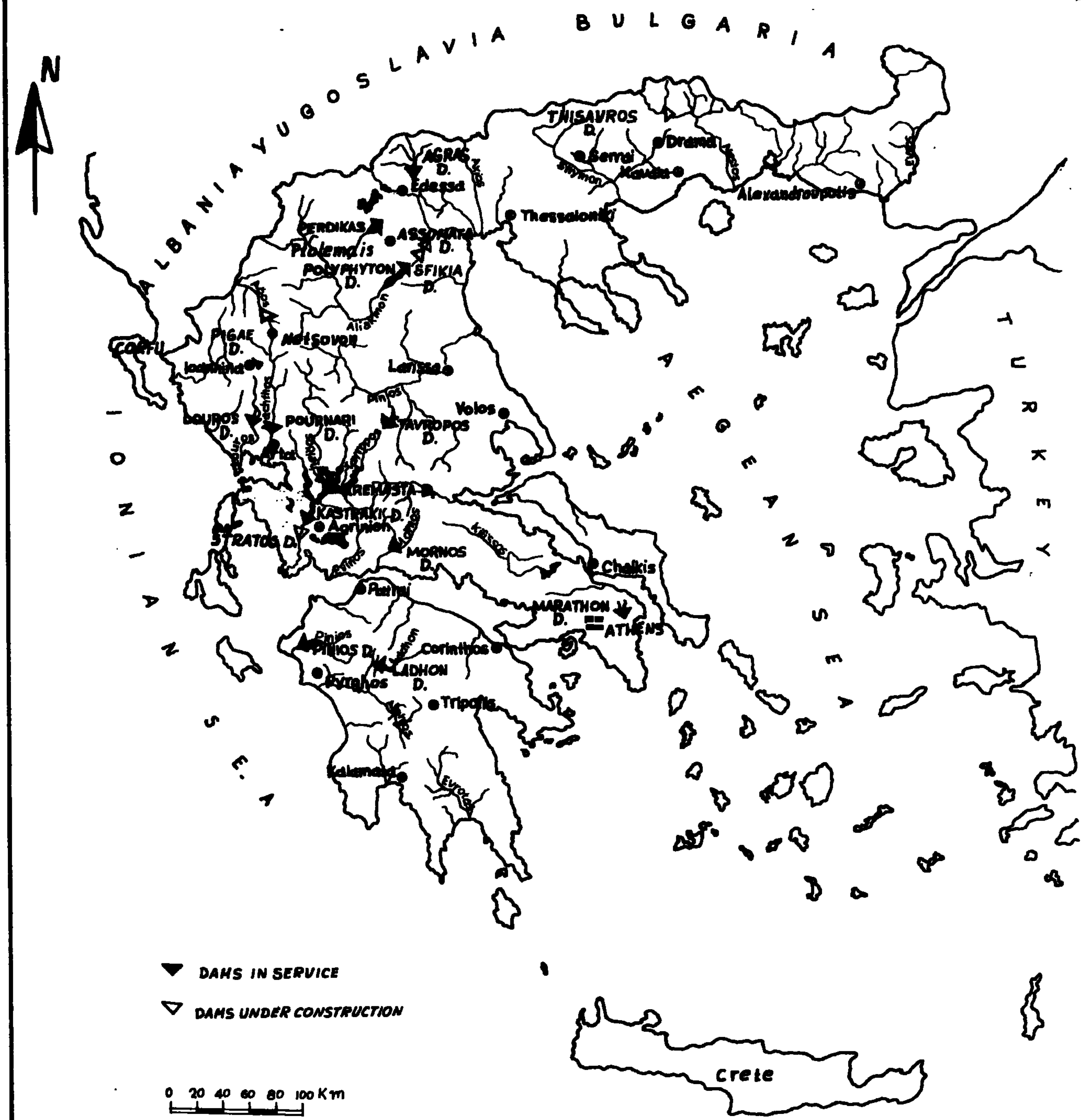


Fig. 2.1 DAMS IN GREECE
(after Greek Com. on Large Dams and PPC)

western world, Greece faces a (1980's) decade of competition, and the national emphasis has to be given to a rational and rapid development of its natural resources.

The world-wide economic crises (inflation in Greece in 1979 was 23 per cent, 25 per cent during 1981 and 22.5 per cent in 1982) of the past decade, and especially the oil crisis, highlighted the importance that energy occupies in a programme of rapid development for every country, and especially for Greece which must be regarded as a developing country.This is exemplified by the following Table 2.1, which shows the present composition

TABLE 2.1

National system installed capacity per type of power station in Greece

(after the London Times, Dec.11,1979)*

Type of station	Generating capacity 31-12-1978	1979-1988 programme (MW)	Total forecast to 31-12-1988 (MW)	Total forecast on 31-12-1988 (%)
Lignite fired	1 893	3 600	5 493	47 7
Hydroelectric	1 400	2 270	3 670	31 8
Total Domestic	3 293	5 870	9 163	79 5
Oil fired	1 230	528	1 758	15 3
Nuclear	-	600	600	5 2
Total	4 523	6 998	11 521	100 0

* Information from the Public Power Corporation (PPC) of Greece

of the electrical power generating stations and the development to be achieved by the implementation of the 1979-1988 programme.

Under this programme, Pournari dam (one of the dams forming the research substance of this Thesis) was incorporated into the national grid with 300 MW (started in 1981). Four other dams are under construction, that is: Stratos dam, with generating capacity of 150 MW; Sfikia dam (studied later) with 315 MW; Assomata dam (also studied later) with 108 MW; and Pigae of Aoos with 210 MW. A few more dams are under design*, but dates of completion have to be re-considered.

Thermal power stations which have to be added within the above programme are due to be installed near Ptolemais in a nearby newly-explored lignite field.

2.1.2 The energy problem and the Greek economy: background, statistics and prospects

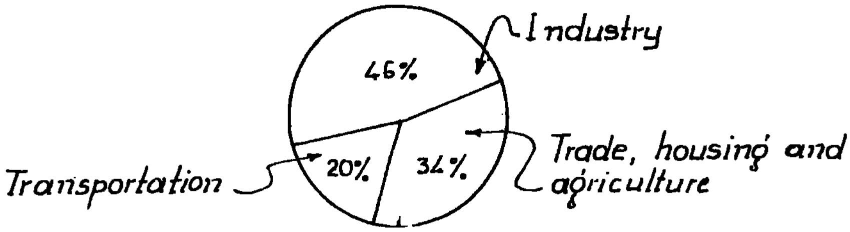
In a conference held in Athens in 1977, the problem of energy in the Greek economy was discussed in the context of its present status and future developments, with emphasis on the exploitation of the Greek natural resources. The main points of interest stemming from this study are reviewed below.

Since the second world war (1939-45) the gross energy consumption in Greece has increased at an annual average rate of 11%. Yet in spite of this high rate the average *per capita* consumption is still only one-third the average consumption per capita in the EEC (Kasapoglou *et al*, 1977).

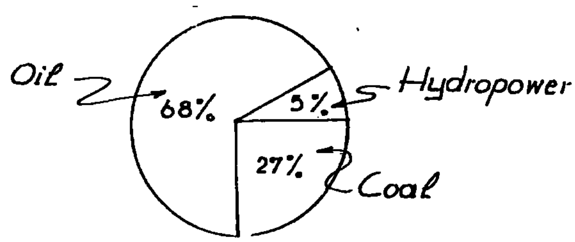
---

* Construction of the 2 kilometre diversion tunnel of Thisavros damsite on the Nestos R. is due to start in 1983.

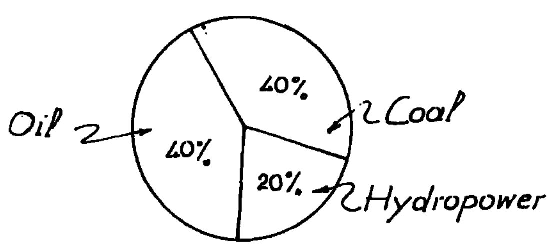
The Consumption in 1975 was 518 x 10^{15} joules and the diagram below indicates the energy demands of several sectors of the economy:



In the same year the partition of the main primary resources for this energy production were as below:



For the period 1972-1977 the electrical energy produced was as follows:



The forecast for the future electrical energy production to meet the demands of the year 2015 have been calculated to reach <u>150 TWH</u> (150 x 10^6 MWH), which implies an annual rate of increase of production of about 6%. The contribution of the several resources are shown diagrammatically in Fig. 2.2.

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* The contribution of the lignite fired power stations to the production of electrical energy in 1982 was 56% (18 000 million KWH) while the contribution of hydroelectric power stations was 16% (3 550 million KWH) (after PPC, 1983).

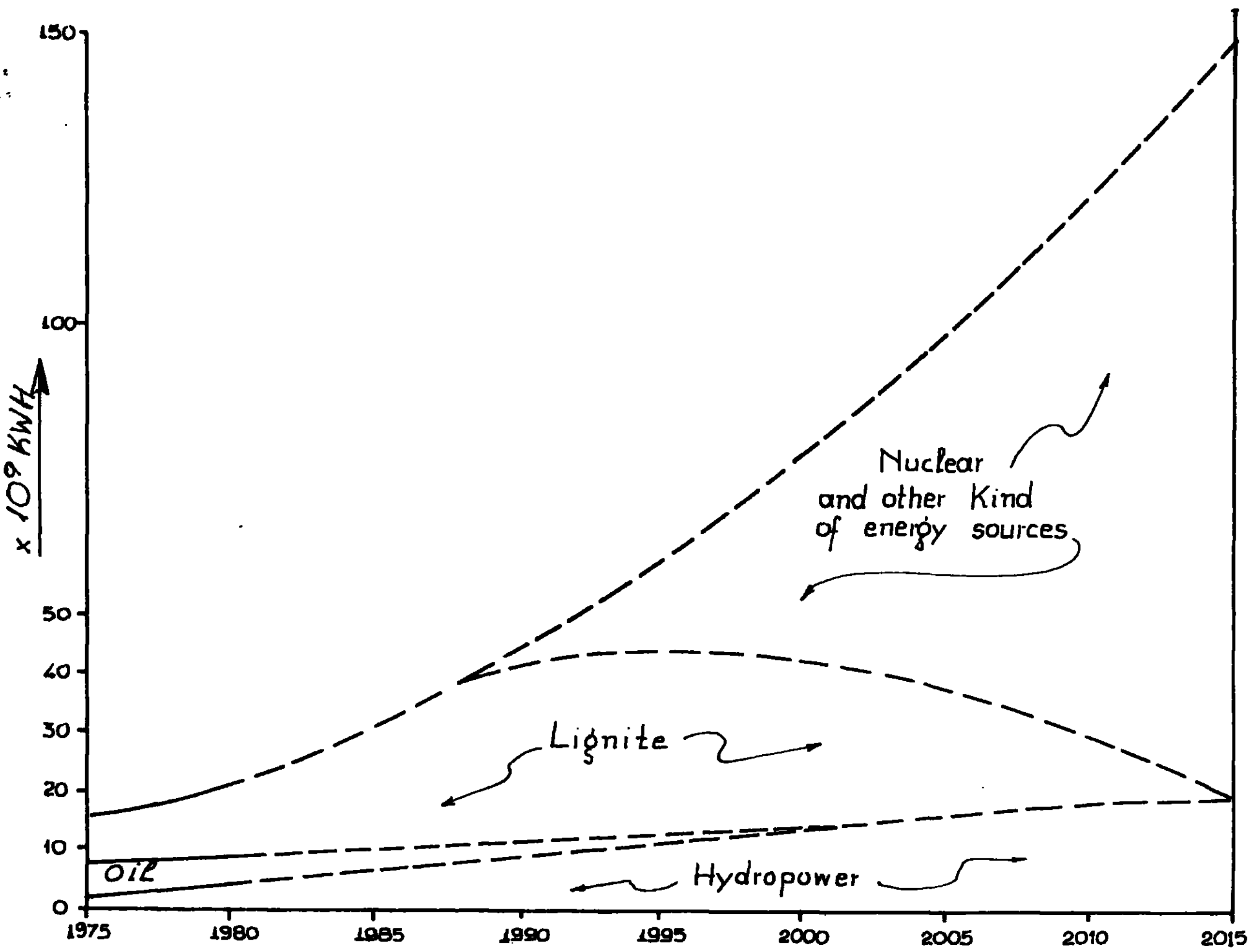


Fig 2.2 Greek electrical energy production up to the year 2015 and the expected contribution of the several natural resources

(after kasapoglou, <u>et al.</u>, 1977)

2.1.3 Greek natural resources for energy production

The major acknowledged Greek resources for energy production under current exploitation are:

i) coal (lignite and peat);

ii) hydropower.

A third resource should be considered, namely the north Aegean Sea oil and gas where production started in 1980. It is estimated that by 1981 oil will be produced at the rate of about 25 000* barrels per day and gas (its equivalent in barrels of oil) at 10 000 barrels per day (Moissis, 1979). Exploration for oil has been intensified throughout Greece, with promising indications in the western Peloponnese at present.

A. Coal (lignite)

The main lignite fields are located in western Macedonia (Ptolemais-Plate 2.1) with 2 080 million tonnes of acknowledged (proven) lignite deposits, in the Peloponnese (Megalopolis) with 540 million tonnes, and in Aliveri where the existing economically-exploitable lignite is almost exhausted (see inset map in Plate 2.1). In the three above locations,thermal power plants have been installed and are in continuous operation. The main peat deposits are in eastern Macedonia (Filippi) with 4 000 million m^3 and where the installation of a thermal power plant had been decided in the early seventies, but subsequently abandoned in the middle seventies.

The explored and likely-exploitable quantities of lignite and peat

---

* The production of 25 000 barrels a day was achieved in the second half of 1981.
1 barrel = 158.987 litres (42 U.S. gallons)

Plate 2.1 Panoramic view of lignite open cast mines of Kardia (Ptolemais)

deposits in the whole of Greece are shown in Table 2.2.

TABLE 2.2

The Greek lignite and peat potential for exploitation

Natural resource	Certain amount (millions)	Probable amount (millions)	Total (millions)
Lignite	3 685 (tonnes)	1 026.6 (tonnes)	4 771.6 (tonnes)
Peat	4 000 (m^3)	115 (m^3)	4 115.0 (m^3)

The lignite production in 1976 was 29 994 million tonnes, and it is estimated that it will increase progressively up to 54.7 million tonnes by 1986.

B. Hydropower

Up to the year 1950 the hydroelectric production in Greece was very limited, with an annual production of 15.5 million KWH, served from power stations with an installed capacity of about 7 MW.

After 1950, when the Public Power Corporation (P.P.C) was established, a systematic study of the potential for hydroelectric development of the Greek rivers began, and certain dams were constructed (see Table 2.3 and Fig. 2.1). The hydroelectric stations now have an installed capacity of 1 700 MW. The 300 MW output from Pournari dam was incorporated in the national grid in 1981.

TABLE 2.3

Main hydroelectric dams in Greece

See also Fig. 1.1

Name	Type of Dam/height above foundation river	Installed capacity/ date of completion	Region of Greece	River
1. Agras	earthfill/ 5m	50 MW/1954	W.Macedonia	Edesseos
2. Louros	concrete gravity curvatured/18m	10 MW/1954	Epirus	Louros
3. Ladhon	concrete gravity/58m	70 MW/1955	Peloponnese	Ladhon
4. Tavropos	concrete-arch double curvature/83m	130 MW/1959	Thessaly	Tavropos
5. Kremasta	earthfill/165m (gravel)	437 MW/1965	Aetolo-Akarnania	Acheloos
6, Kastraki	earthfill/96m (gravel)	320 MW/1969	"	"
7. Polyphyton	rockfill/ 96m	360 MW/1974	Western Macedonia	Aliakmon
8. Pournari	earthfill/ 102m	300 MW/1981	Epirus	Arachthos

The developed fraction of the potentially economic and technically exploitable Greek hydropotential is only 14 per cent (Shaw, 1978; see Table 2.4 below).

The Greek mean annual hydroelectric potential is estimated to be 88 billion KWH. The estimated possible exploitation of this hydropotential for hydroelectric purposes by present-day technology is 21 billion KWH. The master plans for the hydroelectric development of the Greek rivers encompass 41 possible dam sites, and for most of these sites preliminary or engineering studies have already been carried out (Nicolaou, 1977).

TABLE 2.4

Economically feasible hydropotential of various countries

(after Shaw, 1978)

Country	Economically feasible hydro-electric potential (TWH/year)*	Ratio to its gross potential (per cent)**	Part developed by 1973 (per cent)
USSR	1 100.0	24.4	11
NORWAY	104.5	18.8	69
TURKEY	71.8	16.7	4
FRANCE	64.5	20.5	73
ITALY	64.1	18.8	61
SWEDEN	60.0	30.6	99
YUGOSLAVIA	47.5	21.0	35
SPAIN	47.1	32.7	62
AUSTRIA	32.9	21.5	58
SWITZERLAND	32.0	22.2	87
ICELAND	30.0	21.4	7
FINLAND	18.0	38.3	57
ROUMANIA	17.5	20.6	43
GREECE	15.6	18.3	14
W.GERMANY	15.5	16.3	99
BULGARIA	10.2	29.1	25
PORTUGAL	9.9	23.6	74
CHECHOSLOVAKIA	9.0	22.0	27
U.K	8.6	13.7	53
POLAND	6.0	18.8	31

* (1 TWH = 10^6 MWH)
* Gross potential is determined from the interaction of precipitation, tography, and geology (which determines run-off and infiltration characteristics)

At present, four dams - Stratos, Assomata, Sfikia and Pigae on the Aoos River are under construction and three others are under final design and preparation of the documents for tender.

TABLE 2.5

Water supply and irrigation dams

Name	Type of dam/height	Purpose/ date	Location	River
1. Marathon	Gravity 63m	Water supply	Athens	Haradros
2. Perdicas	Earthfill 30m	Water supply	Western Macedonia	Perdikas
3. Pinios	Earthfill 53m	Irrigation	Peloponnese	Pinios
4. Mornos*	Rockfill 126m	Water supply	Fokis	Mornos

* Two hydroelectric units of 6 MW are due to be installed under a new programme for small hydroprojects.

A substantial share of the remaining hydropotential could be used for irrigation or water supply purposes, an option which has been considered in previous studies and adopted in practice (see Table 2.5) for river development. This is also the case for small hydroelectric projects and pump-storage schemes in combination with existing reservoirs, since the topography and geology of the country favours such schemes. Irrigation and water supply dams are in existence (Table 2.5), but more should be constructed. Occurrence of prolonged dry periods and the effects on agriculture,industry, and population concentration in urban areas necessitate the construction of multipurpose dams and reservoirs as a viable policy of water resource allocation.

C. Other potential resources for future development

i) Among the other potential sources of energy in Greece it seems at present that the oil and gas resources are limited and their exploitation does not justify the construction of power stations to run on such fuels. On the other hand, a nuclear power station is being considered for the late 1980's. This is so, despite the fact that nuclear minerals, while present in northern Greece, have yet to be proved economically exploitable.

ii) Solar energy is used in many countries, and in Greece, for domestic heating purposes. But a wider use of solar energy is being considered since Greece has a favourable geographic position.

iii) Aeolian (wind) energy can be used if the costs of installation are less than 450 dollars per kW (Kasapoglou et al., 1977). The most promising sites in Greece are the Aegean islands where research is currently being undertaken.A present drawback for the generation of both solar and aeolian energy is that the sources are not constantly active to produce steady-state energy, and the problem of storing the energy that has been produced has not been solved.

iv) There are sites in Greece with a potential for geothermal energy (for example, Methana, Thermopylae, the islands of Icaria, Lesbos, and others). Investigations have been carried out in Milos island where exploratory boreholes about 1 100 m deep have been drilled with encouraging findings. Vapour at 300-310^{o} (with a pressure of 83 to 125 atm.) has been extracted at a rate of 40 tons/hour with a water proportion content of 30-40%. It is estimated that 10 boreholes in Milos could support a 40 MW thermal station (Kasapoglou *et al.*, 1977).

2.1.4 Dams and reservoirs: background and prospects for hydro-electric and other dam development in Greece

The "World Register of Dams" as mentioned earlier, produced by the International Commission on Large Dams (ICOLD), records more than 12 000 large dams identified in more than 70 member countries of ICOLD (Mermel, 1976). In the U.S.A more than 5 000 dams are recorded and about 125 are being added annually.

Japan comes second with 1 900 completed, and more than 30 are being added annually. Among the other members of ICOLD, Spain has over 800, Brazil over 600, Britain over 500, Canada over 500, Italy over 500. In Greece only 12 large dams have been registered (*see* Table 2.3 and 2.5) but more can be added (*see* Table 2.8).

A. Types of dams and trends of construction

Construction of dams and reservoirs requires the closest cooperation of engineers, soil mechanics experts, and geologists in the planning, design and construction so as to assure a maximum degree of safety and operational efficiency, the minor possible damage to the environment and to be economically beneficial. Dams range in size and complexity of construction from low earth embankments or concrete structures, made to impound or divert water and wastes (*see* Tables 1.1, 1.2) to massive earth or concrete structures across major rivers to store water for several purposes.

Although there is no existing dam that exactly duplicates another existing dam or any other that will ever be built, it is possible to identify several basic kinds of dams. Differences depend upon

their geometric configuration and the materials from which they have been constructed (Table 2.6).
Under special circumstances, features of the basic types are combined within a particular dam to meet unusual design requirements. An example of this is the Keban dam in Turkey which combines earthfill, rockfill and concrete gravity elements (see Table 1.1).

TABLE 2.6

Types of dams

(after Wahlstrom,1974)

---

1 Embankment dams

   a) Homogenous dams constructed entirely from a more or less uniform natural material.

   b) Zoned dams containing materials of distinctly different properties in various portions of the dam.

2 Concrete arch and dome dams

   a) Single arch and dome (double arch) dams (cupola dams)

   b) Multiple arch and multiple dome dams

3 Concrete gravity and gravity arch dams (masonry)

4 Concrete slab and buttress dams

5 Dams combining two or more basic characteristics of the above basic types.

Structures which are considered today as dams also comprise those which store mine (industrial) waste materials, the "waste tailing dams". A recent survey of dams in the U.S.A identified several of these structures (Mermel, 1976), some of which stored volumes exceeding many of the world's largest dams, such as the New Cornelia

Tailings dam in Arizona which has a volume of 209 x $10^6 m^3$. In comparison, the world's largest earth embankment dam, Tarbela dam in Pakistan, has an embankment volume of 142 x $10^6 m^3$. It is noted in this brief review that the trends in dam construction over the last decades favour earth and rockfill dams.

The world register of dams indicates that since 1960 the rate of building of embankment dams has increased compared to that of other types of dams. Mermel (1976) states that, in United States alone, the dams constructed during the last few years are 98% embankment dams. Nicolaou (1977) reports that from the 1 043 dams which were constructed throughout the world during 1969-1971, 852 (81%) were embankment dams and only 191 were concrete dams. Schnitter, in 1976, quoted the information in Table 2.7, pointing out the decline of high arch dams after 1960.

TABLE 2.7

Percentage and number of high arch dams

(after Schnitter, 1976)

Period of completion	Maximum height (m) 100-149	150-199	over 200	Number of arch dams
	per cent*	per cent*	per cent*	
Before 1940	28	-	-	5
1940-1949	50	33	-	5
1950-1959	43	75	100	28
1960-1969	32	66	56	51
1970-1979 (estimated)	24	39	44	29

* of all dams of the same height built in the decade.

Among the highest embankment dams which have to be mentioned here are the Rogunsky dam (earthfill) in USSR with a height of 325m, the Nurec dam (earthfill) in USSR with a height of 317m, and the recently - announced construction of an earthfill dam in the Elan Valley in Wales with a height of 369.4m, which is due to be constructed in the 1980's.

In Greece, 8 of the 12 dams which have been registered are embankment dams built after 1960. The highest of these dams is the Kremasta dam at 165 (see Tables 2.3 and 2.5).

The reasons which explain the trend towards the construction of embankment dams in a higher proportion to other types of dams must be the following:

a) Exhaustion of favourable sites for building concrete gravity or arch dams, and

b) Embankment (earth and rockfill) dams are flexible in their design and more amenable to highly mechanized procedures, so they become highly cost-effective.

B. Prospects for dam building in Greece

In the foregoing paragraphs the problem of Greek hydropower has been discussed in the context of Greece's needs and prospects for energy in the future. It was noted that only 14% or so of the potential has been exploited (see Table 2.4).The dam construction programme has to be intensified in the present and the next decade in order to meet peak energy demands which will arise in future.

The master plans for the hydroelectric development of the main Greek rivers cover 41 possible dam sites (see Table 2.8).

TABLE 2.8

Distribution of annual runoff and hydropotential of Greece

(after Nicolou, 1977)

Main Greek Areas	Surface runoff km^2	Mean height m	Mean annual rain-fall mm	Mean runoff mm	Theoretical surface hydropotential 10^6 KWh	Technically exploitable hydropotential 10^6 KWh	%	Technically exploitable rivers	Main Projects under study	Projects operating or under construction: Name	Energy production 10^6 KWh
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1.Western Thrace	6.325	540	970	480	6.212	1.500	24	Nestos	Thissavros* Platanovrissi Temenos		
2.Western Macedonia	11.700	785	814	432	10.444	2.300	21	Edesseos Agras		Edesseos Agras	108
								Aliakmon	Palialona	Polyfyton Sfikia* Assomata*	1.428
3.Epirus	9.338	531	1.220	1.050	15.642	6.300	40	Arachtos	Steno-Kalaritiko St.Nikolas Pistiana	Pournari	
								Kalamas	Soulopoulo Vrosina Minina Kioteki		
								Aoos	Pigae* Elefteron Vovoussa Melissopetra		
								Louros		Louros	59
4.Western Sterea Hellas (Western Central (Greece)	9.667	705	1.265	800	14.880	5.500	37	Acheloos	St.George Messohora Avlaki Sykia* Stratos	Kremasta Kastraki	2.450
								Evinos	Famila Perista Dendrohorion		
								Agrafiotis	Agrafa		
								Tavropos	Viniani Markopoulon	Tavropos	258
								Trikeriotis	Trikeriotis		
Total (1-4)	37.030				47.178	15.600	33				4.807
5.Other Greece	78.321				37.423	5.200	14	Ladhon		Ladhon	293
Total (1-5)	115.351				84.601	20.800	24				5.100

Remarks: $\text{Column (8)} = \frac{\text{Column (7)}}{\text{Column (6)}} \times 100$

* Under construction

Preliminary or engineering studies have been carried out for most of the sites. In parallel, other Government bureaux are planning several dams and reservoirs, for uses other than hydropower, at other sites. About 85% of these sites belong to regions within the main body of Greece, that is, Central, North and Western Greece (see Table 2.8 and Fig. 2.1).

This distribution allows a more systematic study of the main characteristics such as geological, geomorphological, geotechnical, seismological factors, as well as the influence which such characteristics exert on the local conditions of each site concerning the main formations - lithological and stratigraphical- their structural details, hydrogeological details, reservoir watertightness, foundation strength and permeabilities, weathering, and availability of construction materials.

The above characteristics, if examined together with the existing topography and hydrology, can facilitate design decisions based upon the selection of the type of the dam which has to be constructed at a given site. Of course the designer has to consider factors other than technical ones, such as those mentioned previously. These factors comprise the project function or functions, environmental considerations, social and overall economic benefits. In the next paragraphs some of the above factors, which have to be considered for the hydroelectric development of the Greek rivers, will be discussed briefly. They constitute the environment in which the Greek dams are placed, or in which many of these have to be constructed and comprise the following : hydrology, topography and geomorphology, geology and geotectonics of the foundations, and the quality and availability of construction materials.

C. Hydrology

The main surface hydrological characteristics of Greece, the hydropotential of some Greek rivers, and the projects under study, their construction or operation, are shown in Table 2.6 (see also Fig. 2.1 for their location).

The character of the Greek rivers may be classified as torrential, due to climatic and topographic conditions. The rains occur mainly during autumn, winter and the early spring, with a virtual absence of rains during the summer. Nevertheless, the rivers sustain a considerable summer flow because many of them are fed from karstic springs, and snow remains until late summer in the high ridges surrounding the upper parts of the river basins. Karstic springs are very important for some dams, like Louros and Ladhon (see Fig. 2.1), which for their operation depend on the water which is provided by such springs. In other cases big karstic springs provide a considerable proportion of the mean annual flow of the large Greek rivers. Reference is made to the Smardacha springs of the Acheloos river at a location upstream of the Kremasta dam, which provide 18 m^3/sec. (Liakouris, 1971).

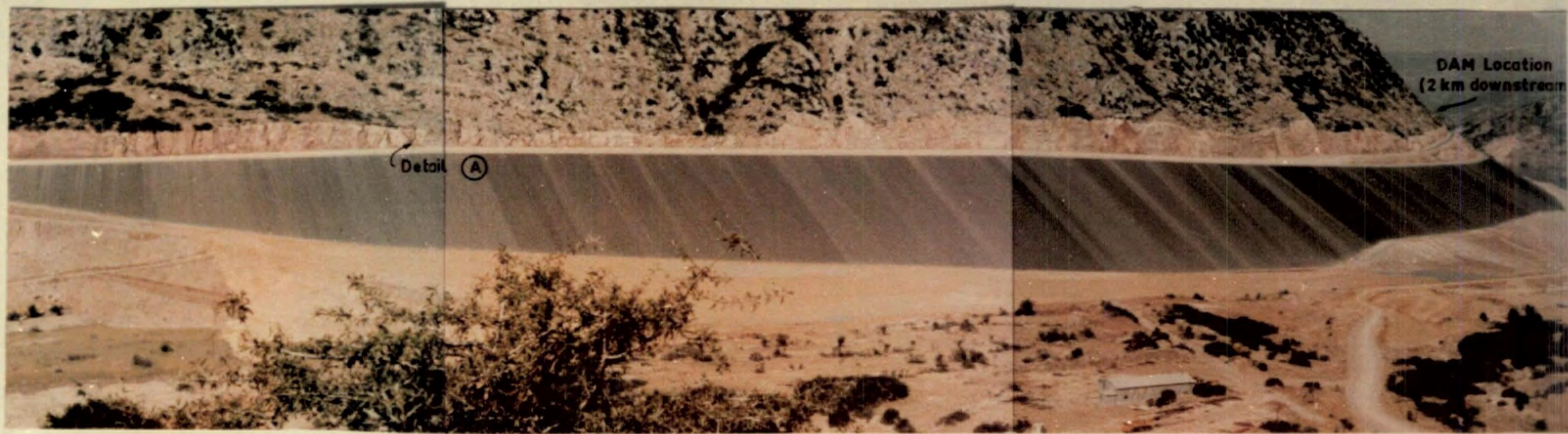
The above implies that for an optimum use of the annual run-off, reservoirs with a high storage capacity are necessary. For example, Kremasta dam, which has a maximum capacity of $4.7 \times 10^9 m^3$, and a useful one of $3.3 \times 10^9 m^3$, constitutes the biggest artificial lake in Greece.

D. Topography and Geomorphology

Topography and geomorphology play an important role in dam construction, affecting the reservoir capacity as well as the type and size of the dam. They also influence the location,size and the type of its appurtenances, such as the powerplant, and the spillway. Topography is usually examined together with the general geology during the feasibility studies of a project development.

Unstable slopes and river terraces, caused by rapid erosion and undercutting of the valley flanks, narrow gorges through hard strata, karstification, active faults, an abundance of constructional materials (such as clay in the terraces with gravelly valley floors and rocky abutments) for most sites, compose at one and the same time a favourable but often problematical environment in which the Greek dams have to be built (see Plates 2.2, 2.3, 2.4).

Greece has a geologically-recent topography with high ridges trending NNW to SSE (for example, the Pindos mountains following an arc which finally trends eastwards, while older ridges like Orthris and Rhodope trend west to east) as aresult of structural and erosional processes which have created the nature (grain) of the country. In many cases these ridges are over 2 000 m high and the main rivers and their tributaries flow several hundred metres below, through narrow valleys or deeply-incised canyons. Terraces (as in Assomata and Sfikia) or high plateaus (like Pigae on Aoos River) are often present as the result of different erosion rates caused by differences in strata hardness and aided by structural anomalies. An example of this latter is the lower Aliakmon canyon which was created by erosion along a regional fault. This canyon has a length of about 25 km and three dams have been built on it: Polyphyton (see Plate 2.3),Assomata and Sfikia (see Fig. 3.13). Generally, the Greek rivers trend north to south or east to west (or west to east), outflowing in



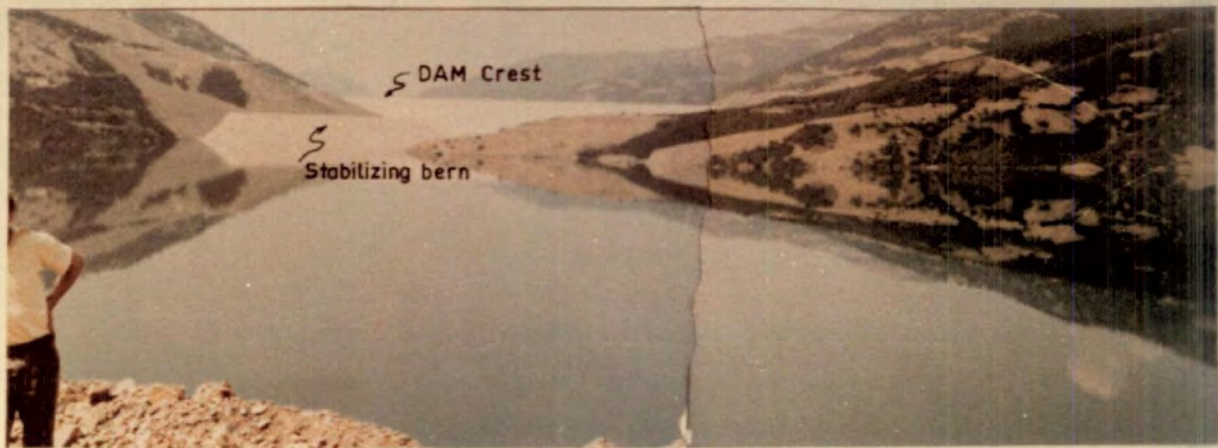


Plate 2.2 Mornos Dam: Protection from reservoir leakages.
An asphaltic concrete slab in Karstified Pyrnos Ridge.
The Mornos Dam is founded on flysch formations.

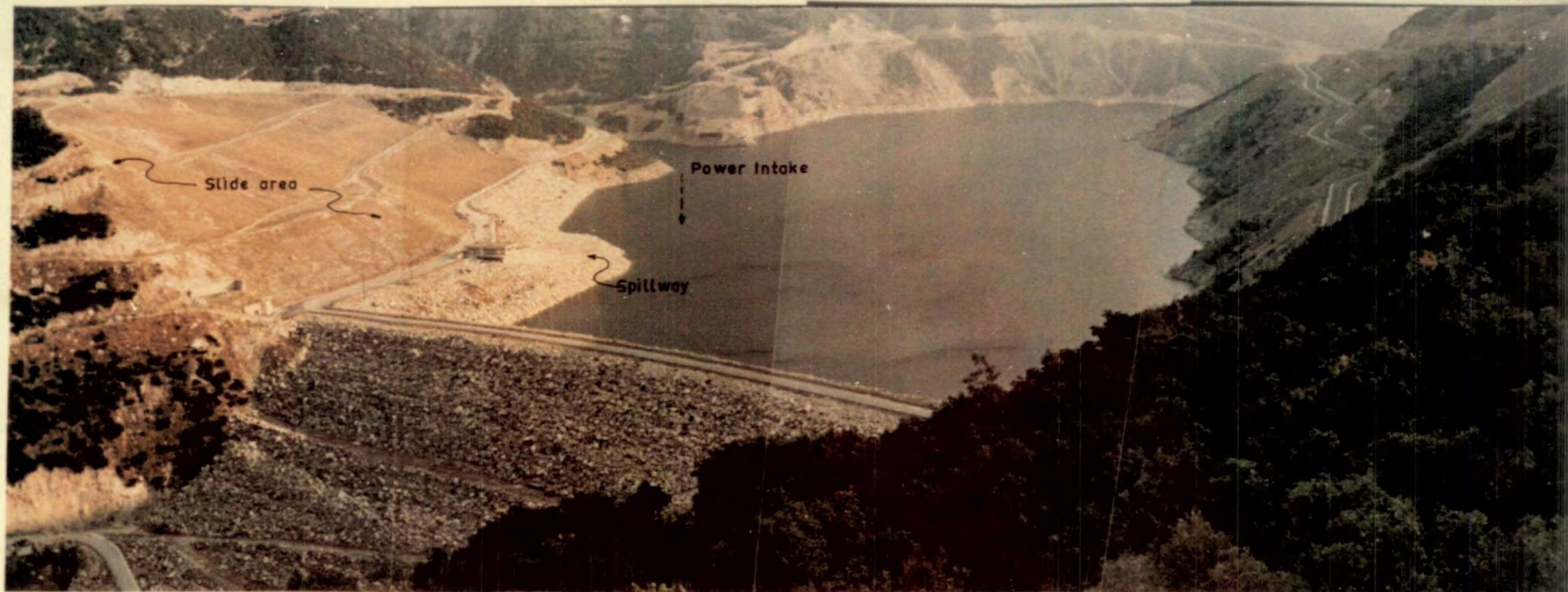
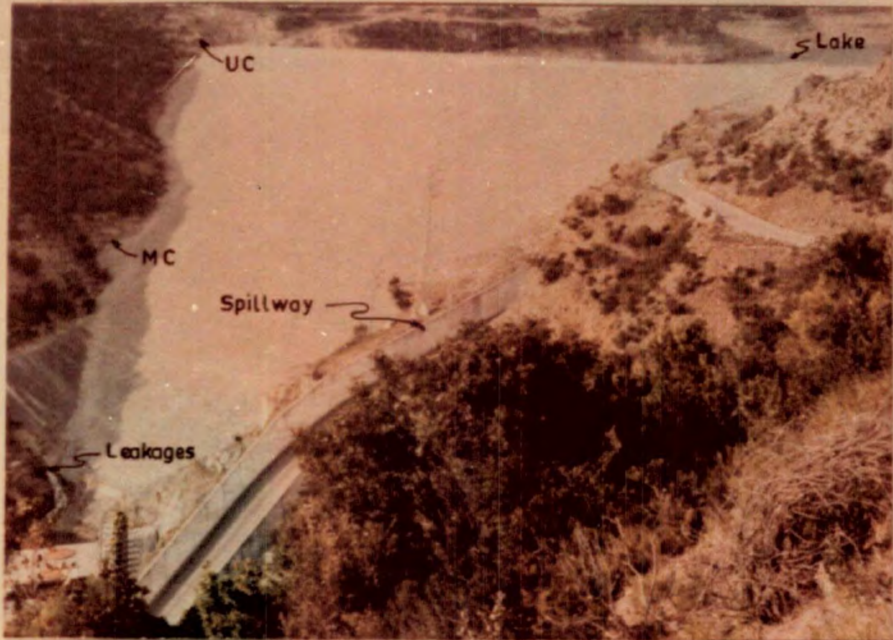


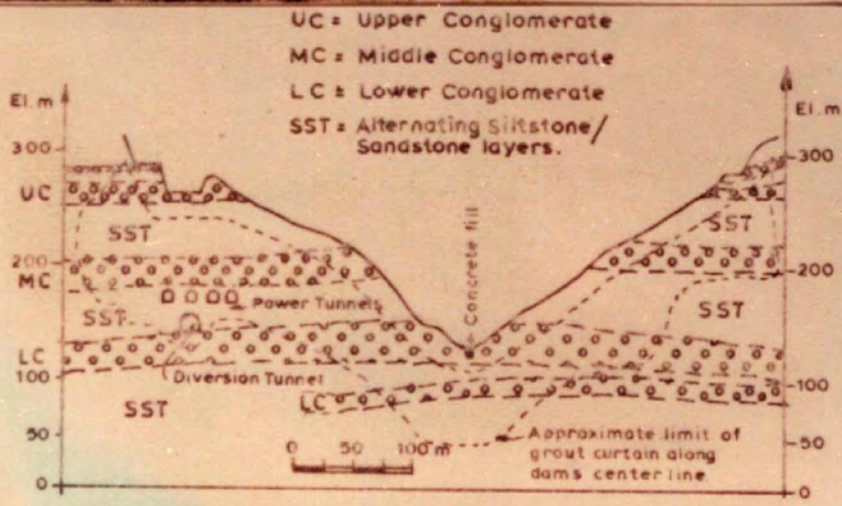
Plate 2.3 Polyphyton Dam (Entrance of Lower Aliakmon Canyon)
Unstable left abutment area in granodioritic – gneissic rocks of
Pelagonian zone.



Kremasta dam
(lookig from dowstream)

Upper
Conglomerate

Plate 2.4. Kremasta Dam. High leakages (and potentially unstable abutments) in flysch formations.



Combination of the Conglomeratic horizons with traversing fault gave rise to extensive leakages.

KREMASTA DAM. Geological section along dam axis.

flood planes near the sea. Figure 2.3 shows diagrammatically the hydraulic gradient of the Acheloos river and its tributary the Tavropos river, between the Tavropos Dam and the Stratos damsite, which is the lowest dam to be constructed on the Acheloos river through a horizontal distance of about 120 km. Probably the highest dam in Greece will be that of Pigai on the Aoos river (see Fig. 2.1), with a foundation level of about 1 277 metres and maximum water level of 1 346 metres. The water will be diverted to the Metsoviticos river, a tributary of the Arachthos river. The head which will be used is 683 metres, with the turbine level at an elevation of 663 metres.

2.2 Geology and foundations

Greece's geology is relatively new, formed through the Alpine and post-Alpine orogenic events and with successive intense fracturing which formed Greece as it is today. The Country is traced by active faults related to the plate tectonic environment which exists in the area and the high incidence of seismicity that has been recorded. The subject of geology, seismicity and plate tectonics will be discussed in Chapter 3 of the thesis, while appropriate mention of these themes will be made when it is necessary in the next Chapters. Table 2.9 (below) lists the incidents that have occurred in Greek dams, mainly attributable to geological factors, and indicates the importance of the foundation conditions and the influence which the local, regional and general geology has played in creating them.

2.2.1 Dam incidents

In the earlier paragraphs some statistics have been presented which concern international activity and the trends in dam construction.

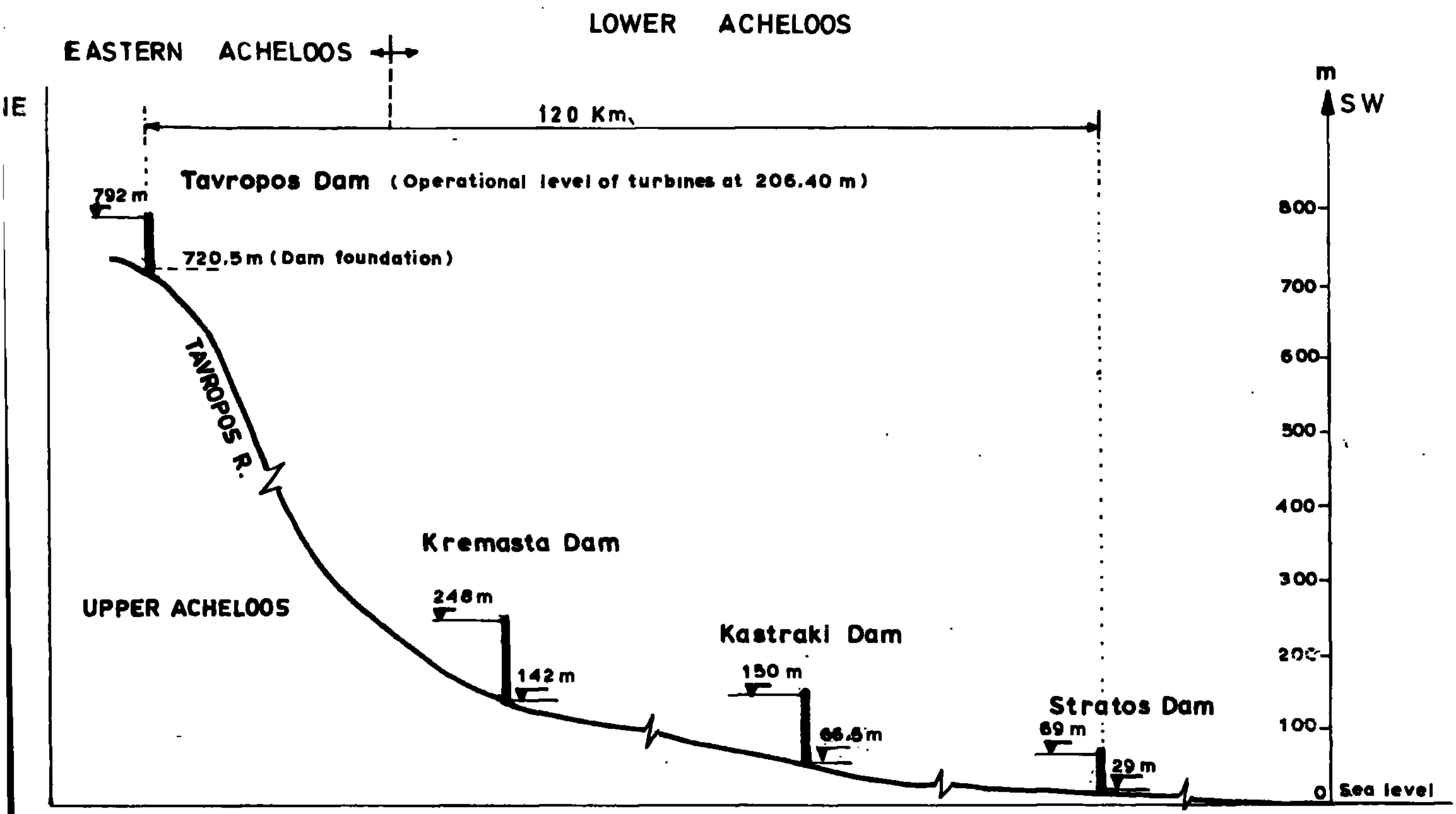


Fig 2.3 Diagrammatic section showing the steep gradient between lower Acheloos River and its tributary the Tavropos River.

TABLE 2.9

Incidents on Greek dams

	Dam/River Area	Foundations/Reservoir Geological Environment	Incident
A	Perdicas/Western Macedonia Earthfill	Alluvial deposits resting on highly karstified limestones at varying depths.	Empty. After filling the reservoir to 20m, sink holes opened. No further remedial measures have been taken.
B	Kremasta/Acheloos Aetoloakarnania Earthfill (Gravel)	Flysch formations with interlayered calcareous conglomerate with solution channels and transverse faults. Thermal sulphate springs exist in the foundation.	Widespread leakages in both abutments occurring at a rate of $1.5m^3/s$, with fears of underground erosion. Unacceptable level of hydrostatic pressures in the abutments. After extensive remedial measures the problems are under control. Induced seismicity has occurred (see Plate 1.4).
C	Kastraki/Acheloos Aetoloakarnania Earthfill (Gravel)	Flysch with an absence of conglomerates.	The flysch in certain reservoir areas presents slope stability problems.
D	Polyphyton/Aliakmon Western Macedonia Rockfill	Gneissic formations with the reservoir resting on deep marly deposits. The existence of large karstic springs from small limestone outcrops into the reservoir creates fears of water losses. Faults and deep weathering into the abutments.	Some instability problems with the right bank above the power intakes. Investigation and remedial measures are continuing. There is no indication of water losses through installed piezometers in appropriate locations (see Plate 1.3).
E	Mornos/Fokis Rockfill	Flysch formations with appearance of karstified limestones into the reservoir (Pyrnos Ridge).	Extensive sealing-off works using asphalt-concrete facing. There are stabilising terms on both abutments, immediately upstream from the dam (see Plate 1.2).

On an international level certain failures have happened (like the Malpasset, the Vaiont and the Teton dam failures) and plenty of incidents can be attributed to geological foundation or reservoir causes. Several sources report on such dam accidents. Systematic studies of dam incidents attribute the causes of such incidents as follows:

The ICOLD, in 1973, prepared a report entitled "Lessons from dam incidents" in which 466 incidents were studied. About 140 of them were considered as failures (see below Table 2.10). Thomas, in 1976 reports that: A 1933 review (ASCE transaction) listed the main causes of failures of large dams as

Embankment dams :	30% - inadequate spillways,
	10% - inadequate cutoffs,
concrete gravity dams:	12% - faulty design,
	31% - inadequate cut offs,
	12% - faulty construction.

In 1959, a list prepared in the USA attributed two-thirds of all failures to geological causes.

In 1961 a Spanish review considered 1 620 dams; about 12 per cent had suffered serious incidents:

40% of these were due to foundations,

23% to inadequate spillways,

12% to poor construction.

Sherard et al. (1963), analysing 214 cases of unsatisfactory performance mainly in USA, summarise

41% attributable to seepage,

28% to overtopping,

11% to slides of various kinds.

TABLE 2.10

Incidents on dams

(after ICOLD, 1973)

Type of Dam foundation	Number of incidents on dams of different types and different foundation						
	A*	B*	G*	E*	R*	M*	Total
Earth foundation							
Failures		3	8	39	6	3	59
Accidents		1		82	1		84
Rock foundation							
Failures	7	4	21	35	6	8	81
Accidents	11	5	14	65	8		103
No information about foundation	3	3	23	101	6	3	139
Total	21	16	66	322	27	14	466

Stage of decision	Number of incidents attributed to different stages of decision-making.						
Exploration	9	5	6	49	2	1	72
Material	1		2	8			11
Layout		1	4	17	3		25
Design	4	6	13	48	3	2	76
Construction	1	1	2	32	5		41
Operation				5	1		6
Supervision	1	1		3			5
Total	16	14	27	162	14	3	236

*A = Arch; B = Buttress; G = Gravity; E = Earthfill; R = Rockfill;
M = Miscellaneous

Gruner (1967) presented his "Classification of risks", the statistics of which might be summarized as

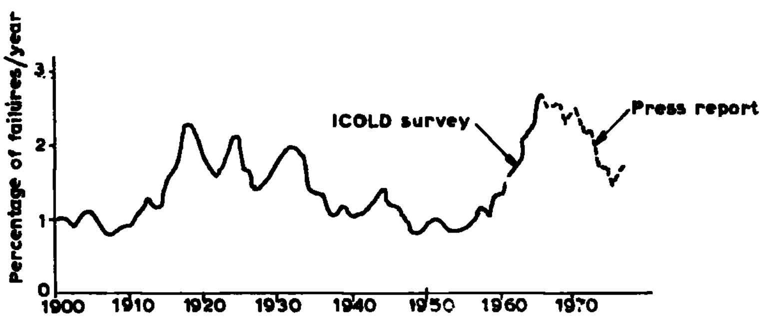
- 45% hydraulic conditions (floods, seepage, piping, uplift),
- 30% type or structure and construction (hydraulic fill, seepage through poor concrete, inadequate designs),
- 7% geology,
- 6% environment (frost, ice, earthquake, decay, hostile action),
- 6% consequences (decay, abandonment, induced earthquake).

Germond (1977), in a survey of dam risks, and working on the ICOLD report on dam incidents as well as other reports, concluded that two large dams per year fail on average. Most of them failed during the first filling (Fig. 2.4).

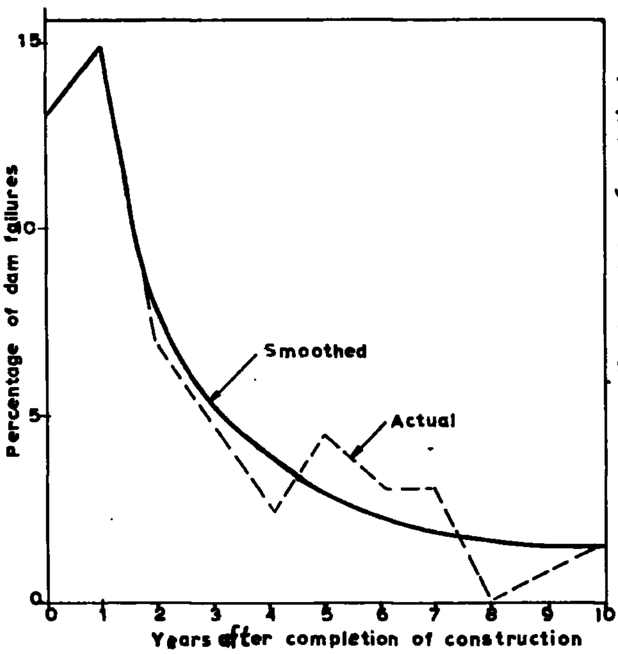
Thus, the geological factors which affect dam foundation conditions are of primary concern from the investigation stage to the post-constructional stage and subsequently the safe continuous operation of the dam. Such factors are listed in Table 2.11 as quoted by Brooker et al. (1968).

2.3 Summary, discussion and conclusions

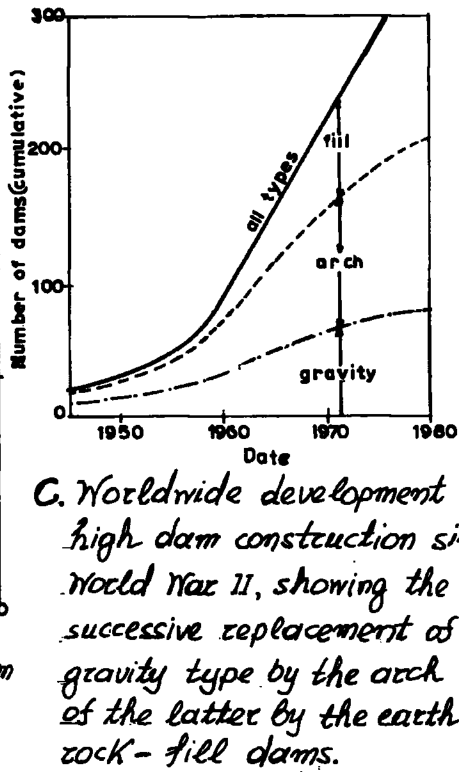
Greece is a developing country and, in order to maintain a satisfactory rate of growth, she must develop, in addition to other sectors of her economy, the primary energy resources which she possess so as to minimize the negative effects created by the present international oil and energy crisis. Basically there are two main sources of energy: coal and hydropower. Coal must be used with prudence, since it can also be used as a basic raw material in other industries (such as the chemical industry).



A. Failures of large dams (1904-1976) expressed as a running average over ten preceding years for each point.



B. Approximate timing of past dam failures.



C. Worldwide development of high dam construction since World War II, showing the successive replacement of the gravity type by the arch and of the latter by the earth-and rock-fill dams.

Fig. 2.4 Dam failures and building trends (after Germond, 1977)

TABLE 2.11

Geological factors pertinent to engineering behaviour

(after Brooker et al.1968)

Geologic factors	Influence	Engineering Consequence
Depositional Environment		
a) Marine deposition	Salt concentration in pore fluid	High swelling potential.
b) Shale, sandstones, coal,volcanic ash	Sediments, generally of variable texture and structure	Incompatible stress-strain properties of adjacent rock type.
c) Variable depth of burial	Interparticle bonds-weak to permanent	
Lithology and stratigraphy		
a) Bentonite layers interstratified with shales, sandstone and coal	Control movement of groundwater; variable stress-strain behaviour	Determines pore water pressure. Shear zones develop as a result of differential rebound during unloading.
Stress History		
a) Loading by younger sediments	Consolidation	Increase in shear strength.
b) Diastrophism and preglacial erosion	Rebound	Alteration of internal stress system.
c) Glacial erosion,loading and unloading	Consolidation and rebound	Indeterminant stresses at rest. Development of shear zones.
d) Valley erosion	Relief of horizontal restraint	Horizontal rebound and vertical jointing.
Structure		
a) Faults b) Joints c) Bedding	Planes of weakness; seepage paths	Controls strength and deformation of rock mass and, as a result, controls design.
Weathering		
a) Precipitation	Disintegration of rock mass	Develops structurally-unsound rock mass.
b) Temperature	Planes of weakness	
Groundwater		
a) Quantity	Variations in flow affect leaching and pore pressure.	Changes in shearing resistance
b) Quality	Changes in free water and absorbed water chemistry	Can result in swelling and an effect on strength.

Research efforts for the future development of other sources of energy (such as oil and gas, geothermal, aeolian and solar) must continue, since conditions in Greece are favourable for these.

Water exploitation for hydropower and other uses, by means of dam building, does provide a certain basis for present and future developments. Hydropower, partly developed (14% of the total exploitable hydropotential), although unable to solve the energy problem of Greece, does offer the most attractive potential for development. This is due to the fact that many favourable damsites may still exist, especially in north-western Greece where almost 80% of the potential hydropower exists.

A proper knowledge of dam foundation conditions pre-supposes a good knowledge of the geology and those geological and geotechnical factors which affect dam foundation characteristics such as permeability and strength.

The incidents recorded in Greek dams focuses attention on the watertightness problems of their foundations.

Lack of proper assessment of the site investigation information, particularly with respect to geological detail, or inadequate treatment,leads to unsatisfactory foundation performance.

Thus, a study and evaluation of the grouting treatment, applied to certain dam foundations in north-western Greece, can shed light on the geotechnical problems created by certain geological characteristics at the sites. This can lead to an informed appraisal of similar geological conditions governing future damsites. More precise design of foundation improvement can be achieved by such a study.

As a first stage, the geological environment of these damsites- the regional geology, geotectonics and seismicity - will be examined in the next Chapter.

CHAPTER 3

Geology and seismology of Greece and their relevance to Greek dams

3.1 General Geology

3.1.1 Introduction

Greece, the southern-most part of the Balkans Peninsula, was created by the events of the Alpine and post-Alpine orogeny.

The Hellenides (Greek ridges) belong to the eastern Mediterranean orogenic systems. Their complexity and evolution has attracted the interest of many researchers, and the geotectonics and geodynamics of the broader region suggest that a new orogenic belt is just developing outside the Hellenic (Greek) arc (Jacobshagen et al. 1978).

The orogenetic evolution of the Hellenides, wholly or partially, may be explained on the basis of three main hypotheses:

a) The geosyncline hypothesis, in which the model comprises several basins and ridges based on the definition of isopic zones.* According to this model (Phillipson, 1898; Renz, 1940; Brunn, 1956; Aubouin, 1959), in the course of several orogenic phases, a field of nappes originated in the internal zones (zones occurring in the eastern part of Greece) of predominantly eugeosynclinal character, and was thrust on to the autochthonous

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* Isopic zones are belts of strata which exhibit similar facies along their length and differ from the zones on either side-a distribution that reflects a comparable linear plane in the conditions under which the sediments were laid down (after B. P. Co, 1971).

and miogeosynclinal external zones (zones in the western part of Greece).

b) Plate tectonic concepts offered a new explanation of the structural development of Greece. After the assignment of Benioff zones dipping north-eastwards (Galanopoulos, 1973; Papazachos, 1976), plate tectonic boundaries were drawn (Ritsema, 1974; McKenzie, 1970; Galanopoulos, 1973), and plate tectonic models were proposed (Jacobshagen, 1978), explaining that the orogenic events of Greece were caused by subduction and collision processes in Mesozoic and Cenozoic times.

c) A third approach suggests that thermodiapiric processes (Makris, 1973, 1978; Wachendorf et al., 1975; Baumann et al., 1976) could explain the origin of orogenesis, causing gravity transport of nappes, deep erosion, crustal thinning and finally break-down in the interior of the uplift. Some authors (Brunn, 1976; Schwan, 1976, 1977) have tried to reconcile the concepts of plate tectonics and thermodiapirism.

3.1.2 The geosyncline interpretation.

According to this theory the Greek territories have been successively and temporarily synclines with distinct periods of uplifting and submersion, in which sedimentation and erosion occurred, and which was accompanied in some cases by limited volcanism.

Among other researchers, Aubouin (1959, 1965, 1973) considered three phases for Greece's palaeographic evolution and tectonic history, based on geological and geomorphological evidence:

a) The geosyncline period, during which the basic isopic zones were created with sedimentation taking place in

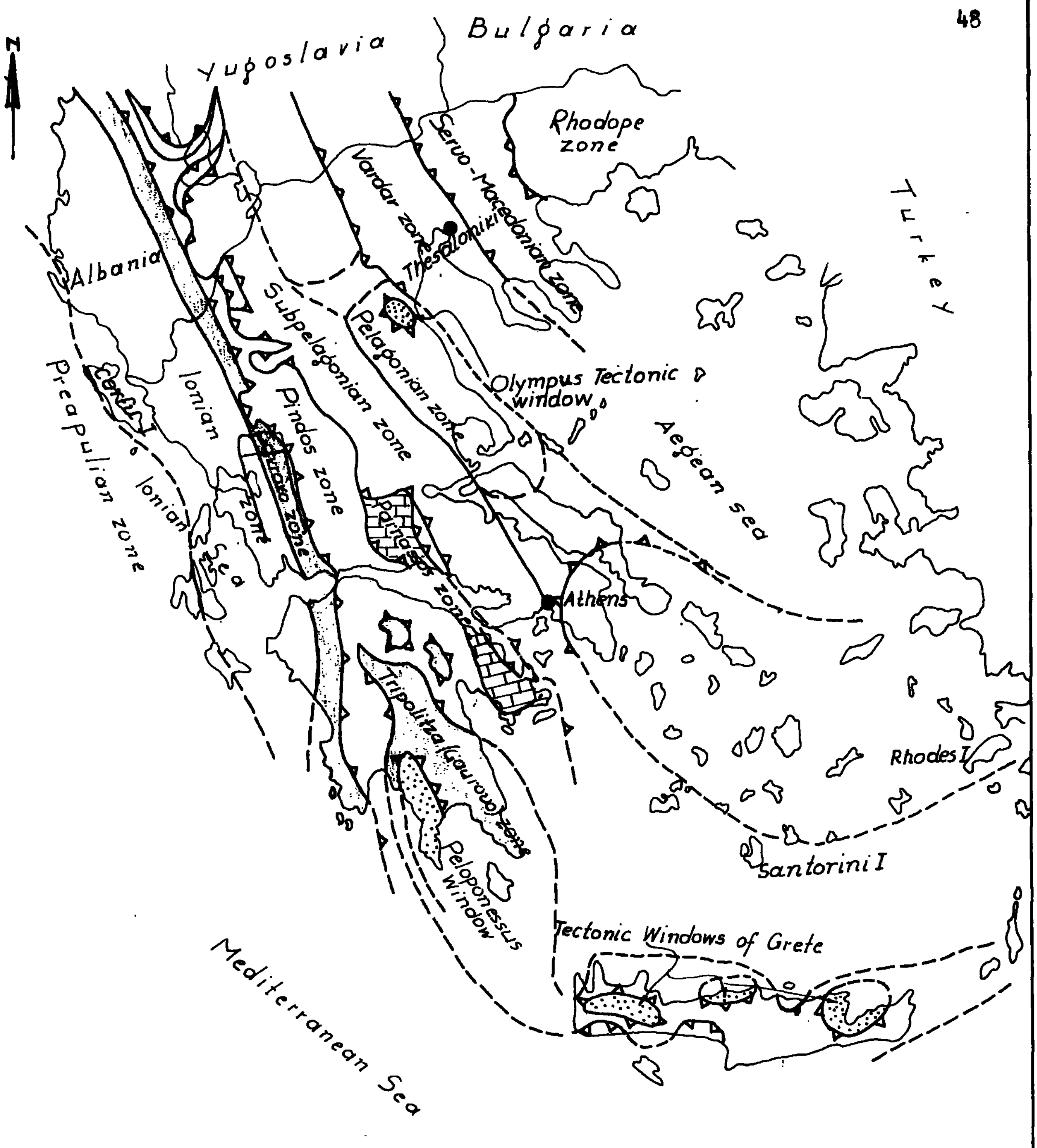


Fig. 3.1. -Isopic zones of Greece (according to Aubouin from the guide bulletin entitled "Reunion Extraordinaire en Grece" of the Geological societies of Greece and France, Greece 1976.)

LEGEND

Geographical boundaries

Certain or assumed boundaries between the main tectonic zones

Thrust faults along the lines separating the main tectonic zones several submerged basins and ridges trending NNW-SSE.

b) The late geosyncline period, in which tectonism started with folding and uplifting, while sedimentation processes continued with <u>mollasic</u>-type sediments (clayey-marly-conglomerates) deposited into remaining basins such as the Meso-Hellenic trough (represented by the Subpelagonian zone).

c) The post-geosyncline period in which intense tectonism took place and created depressions and ridges independently from the configuration of the previously-existing isopic zones.

Thus, the Hellinides (Greek ridges) create a basic geosynclinal chain and comprise two groups of isopic zones (Fig. 3.1) as have been proposed by several research workers (Table 3.1):

a) Group I (Eastern Greece), which contains from east to west the internal zones of predominantly eugeosynclinal character.

1) The zone of Rodhope, which is divided into the massif of Rodhope and the Servomacedonian massif.
2) The Vardar zone, which comprises the subzone of Paeonia and Propaeonia, the anticline (subzone) of Paikon and the subzone of Almopia (forming submerged basins and ridges recognized from the differentiation of their lithological facies).
3) The zone of Pelagonia, of which the western part, the Subpelagonian zone, comprises the Meso-Hellenic trough, and ophiolitic emissions on account of which some have considered this zone to be a separate one called the zone of "Eastern Greece".

TABLE 3.1.

The Geotectonic zones of Greece according to several research workers

	A. PHILLIPSON (1898) ①	F. KOSSMAT (1924) ②	C. RENZ (1940) ③	J.H. BRUNN (1956) ④	J. AUBOUIN (1956) ⑤	J. MERCIER (1968) ⑥	
Group I Western Greece			Paxos Zone	Paxos Zone	Pro-Apulian Zone	Pro-Apulian Zone	
	Jonian Zone	Adriatic-Jonian System of folds: Western-Epirus folds and Jonian Islands	Adriatic-Jonian Zone	Adriatic-Jonian Zone	Jonian Zone	Jonian Zone	
	Zone of Pindos: Western Sub-zone of Flysch	Adriatic-Jonian System of folds: Interval folds of Epirus	Tripolis subzone	Tymphis zone (Tripolis zone)	Gavrovon zone	Gaurouon zone	Gaurouon-Tripolis ? zone (Dercurt 1964)
	Zone of Pindos	Zone of Olonos-Pindos	Zone of Olonos-Pindos		Pindos Zone	Pindos Zone	
	Subzone Parnassos / Folded zone of Central-Eastern Greece	Zone of Vardoussia - Parnassos	Zone of Parnassos-Giona	Intermediate zone	Hyper-Pindic zone	Hyper-Pindic zone	
					Parnassos zone	Parnassos zone	s/z of Vardousia ②
	Subzone Itis-Orthrios / Folded zone of Central-Eastern Greece	Limestones of Eastern Greece and Ophiolitic zone			Sub-Pelagonian zone	Sub-Pelagonian zone	Olympos of Thessaly ③
Group II Zones of Eastern Greece	Crystalline Massifs of North-Aegean and Cyclades Islands	Pelagonian Crystalline zone	Pelagonian Massif / Attico-Cycladic Massif		Pelagonian zone	Pelagonian zone	
	Folded zone of Eastern Aegean Sea	Axios zone (Vardar zone)	Axios zone (Vardar zone)	Axios zone (Vardar zone)	Axios zone (Vardar zone)	Almopias zone Paikon zone s/z of Pro-paeonia Paeonia zone	
	Crystalline Massif of Northern Aegean	Basically the Rodhope Mountains	Rodhope Massif	Rodhope Massif	Zone of Rodhope	Branch of Rodhope / Servo-Macedonian zone	Branch of Rodhope ④

(after Mariolakos 1976)

4) The zone of Parnassos is a mountainous complex north-west of Athens. It is considered by some to belong to the internal zones and by others to belong to the external zones. However, it seems more appropriate to regard it as belonging to the internal zones.

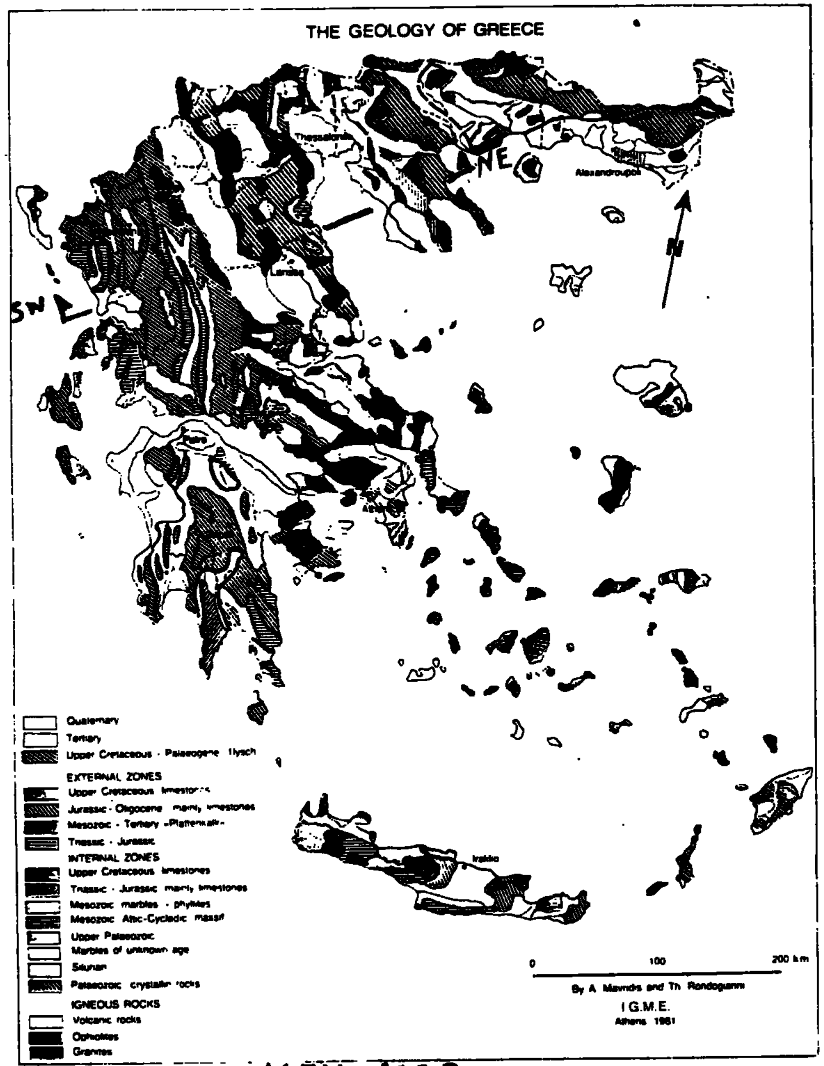
b) Group II, the external zones autochthonous and miogeosynclinal in character from east to west.

1) The Pindos zone, or Olonos-Pindos zone, part of which constitutes the Hyperpindic subzone.
2) The Gavrovo zone.
3) The Ionian zone.
4) The Pre-Apulian zone, or the zone of Paxos.

Aubouin considers that the Pre-Apulian zone, the Ionian zone, and the Gavrovo zone, are autochthonous zones, while the others are thrust nappes, like the Pindos zone thrust over the Gavrovo zone, the Pelagonian zone having been thrust over the Pindos zone. The zone of Vardar and the zone of Rodhope, although they have moved westwards, still retain their relative palaeographic positions (see Fig. 3.2).

Dercourt (1970) suggested that during the geosyncline-formation period, tectonism started from the internal zones, and extended in a westerly direction with more episodes in the eastern zones. The ophiolitic sheets of the internal zones and the nappes of the central Greek zones (ophiolites considered allochthonus by some and autochthonous * by others) are attributed to a lower

* Extrusions at the junctions between ridges and troughs. These flowed down into the troughs to form vast, differentiated igneous bodies (Brunn, 1956;Aubouin, 1959).



KEY MAP

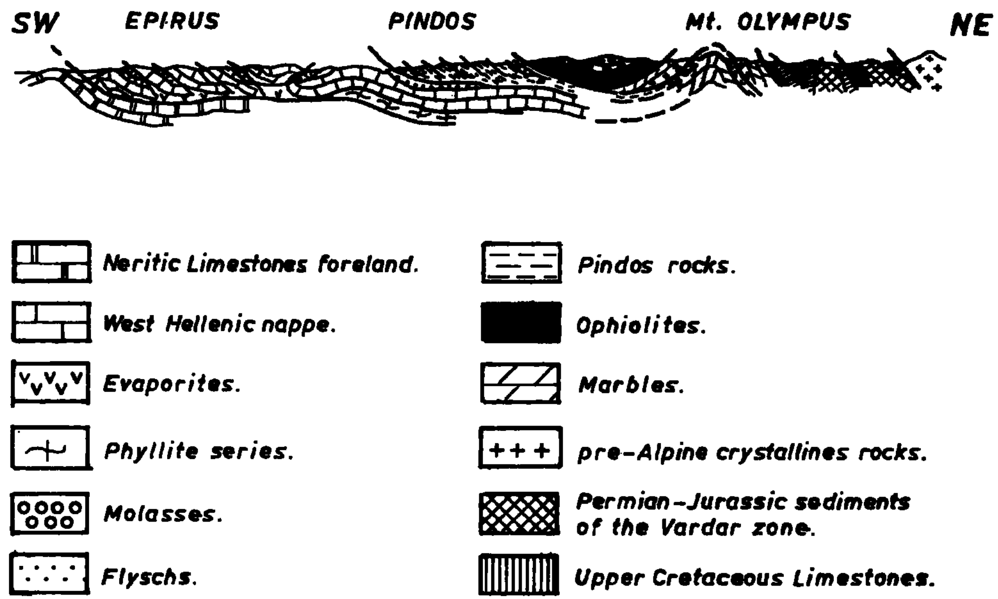


Fig: 3.2 *Cross-section through the Hellenides of northern continental Greece.* (Re-interpretation after Aubouin,1973; from Jacobshagen et al., 1978)

Cretaceous obduction of an oceanic plate (Dercourt, 1970; Bernoulli and Laubscher, 1972; Zimmermann, 1972).

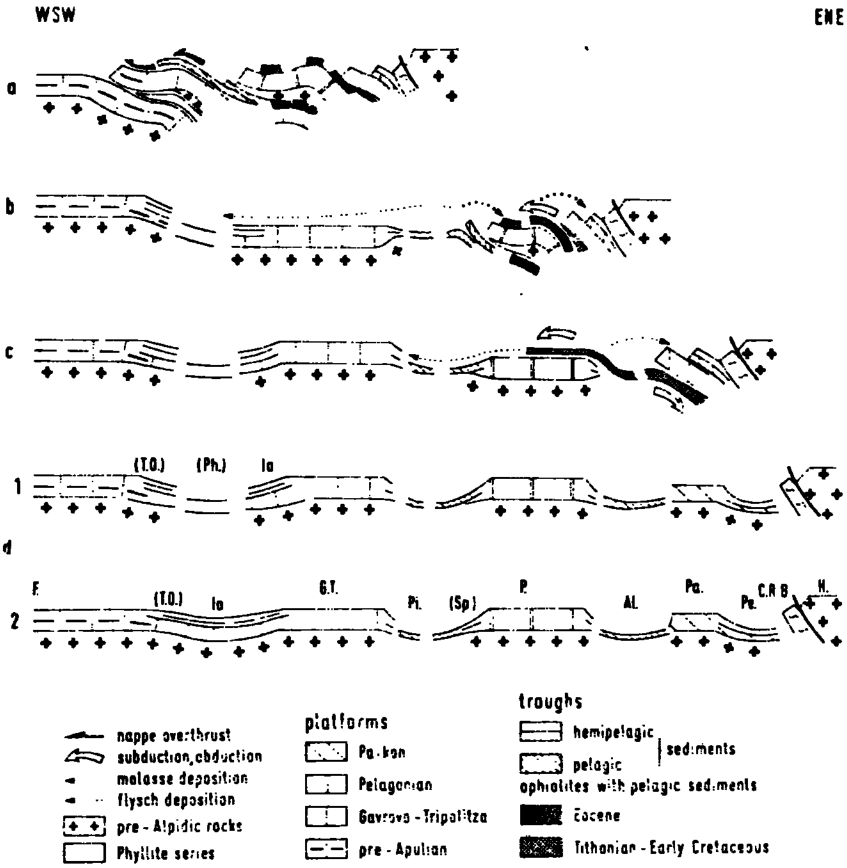
During the late-geosyncline period the main tectonic zones folded (with their main axes parallel to the zones) and faulted. Contemporaneously, the large masses thrust successively from east to west, and created scales and basic to ultrabasic nappes (the Pindos cap) with the appearance of volcanism and metamorphism in the internal zones.

Finally, at the post-geosyncline period, a vertical and horizontal intense tectonism, accompanied (and followed) by strong contemporary erosion, created the present-day Greece (see Fig. 3.3 which traces the orogenic evolution of Hellenides).

Of the two groups of isopic zones, Group I comprises igneous, sedimentary and volcano-sedimentary rocks, a proportion of which are metamorphosed, and with the appearance of either deep-seated or surface volcanism.Group II comprises sedimentary rocks, mainly limestone with superimposed flysch formations, together with siliceous limestones and cherts (Radiolarite series). Evaporites also exist, and their surface appearance (gypsum) suggests diapyric growth phenomena (Messologhi-Preveza line). Phyllite series such as the Tripolitza phyllites are considered to be the basement rocks of the external zones (Jacobshagen et al, 1978).

There are many points of argument surrounding the tectonic history of Greece, which give rise to different explanations.

One of them is the problem of "tectonic windows", which are sedimentary formations emerging among surrounding metamorphic rocks. An example of this is the Olympus "tectonic window"



(after Jacobshagen <u>et al.</u>,1978)

Fig. 3.3. <u>Stages of the orogenic evolution of the Hellenides</u>.The section crosses northern continental Greece (<u>see</u> also Fig. 3.1 and Table 3.1).

a) <u>Middle Miocene</u>:Overthrust of the Central Hellenic nappes onto the Gavrovo-Tripolitza platform, transportation of the West Hellenic nappes over the foreland.

b) <u>Middle to Upper Eocene</u>:Ophiolite obduction onto the Pelagonian platform from the Vardar ocean which was closed at that time. Subdiction of an oceanic slab (from the "Pindos" or the Vardar ocean) beneath the Pelagonian platform("Blue-schist units").Folding of the internal Pindos trough. Sedimentation of the external flysch starts; molasse deposition in the Mesohellenic trough, in the Vardar trough, and in Thrace.

c) <u>Tithonian-lower Cretaceous</u>:Subduction of the Vardar ocean beneath the structure and geodynamic evolution of the Aegean region circum-Rhodope belt and the hinterland. Obduction of the Eo-hellenic ophiolites on to the Pelagonian platform. Sedimentation of ophiolite-bearing flyschs.

d) <u>Lower Malm</u>:Troughs and ridges after the circum-Rhodope orogenesis. (1) Phyllite series as substractum of the West Hellenic carbonata series [(<u>see</u> above (a)] . 2) Phyllite series interpreted as the filling of a trough [(<u>see</u> above (b)] . Al = Almopias trough; C.R.B = circum-Rhodope belt; F = foreland;G.T = Gavrovo-Tripolitza platform; H = hinterland; Io = Ionian trough resp.zone; P = Pelagonian platform; Pa = Paikon platform; Pe = Peonias trough; Ph= Phyllite trough; Pi = Pindos trough; Sp = sub-Pelagonian zone; T.O. = Talea Ori series.

(Godfriaux, 1962, 1970). The Olympus tectonic window exposes a vaulted, "non metamorphic", entirely-neritic sequence of carbonate rocks, ranging from Triassic up to Eocene. This neritic sequence most probably belongs to the external Hellenic Gavrovo zone (Fleury and Godfriaux, 1975) or, as Godfriaux previously argued, to the Parnassos zone. Hence, the entire surrounding metamorphic terrain represents a huge basement nappe.

3.1.3 Plate tectonics in Greece

The Alpine-Mediterranean orogenies and their complex loops have always been explained as effects of crustal convergence between Africa and Eurasia. The Hellenides (Fig. 2.4) represent a south-facing (towards Africa) stack of thrust sheets in the upper plate of an active (Africa)north-east- dipping subduction zone, resulting from 150 million years of multiphase plate tectonic history (Roeder, 1978). According to the same author, depending on the assumed model, five or six sites of subduction-like processes leading to north-east-dipping plate overlaps have been identified (Fig. 3.4).

In detail, the Hellenic orogenies represent the continuous consumption of Tethis and its marginal southern seas. Subduction of the Tethyan Palaeozoic and Mesozoic oceanic crust took place at the internal edge of the Vardar zone, perhaps within the Vardar zone and below the Eohellenic obductives (phase of tectogenesis of the Jurassic-Cretaceous boundary in which ophiolitic nappes obducted). Progradation of the Eohellenic suture into the Median Crystalline belt created a second (and third) subduction zone below the Pelagonian basement nappes and below the Pindos nappe.

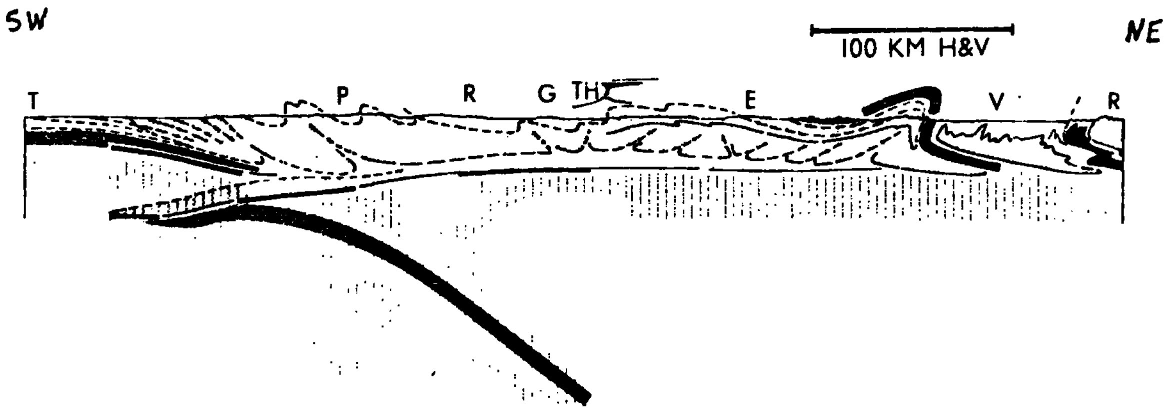


Fig.3.4. An interpretive crustal section through Hellenides (after Roeder,1978). More detail is given in Fig. 2.4 A-D

T=Hellenic trench- P=Peloponnese - R=Argolis - G=Aegina - TH=Hellenic are volcanism, projected - E=Euboea - V=Vardar zone - R=Rhodope

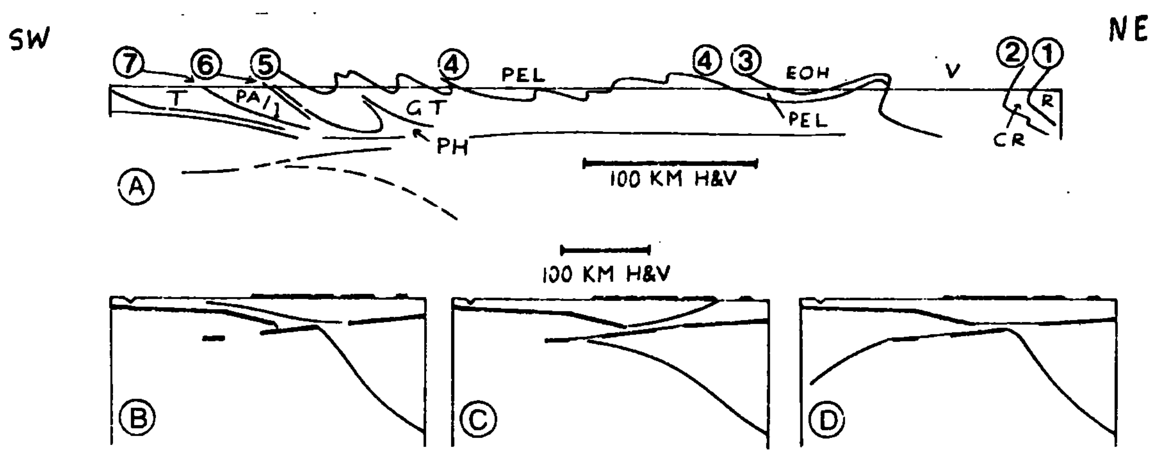


Fig.3.4 A-D. Schematic structure section across the Hellenides (A) and crustal section interpreted in three different plate tectonic ways (B,C,D).

(A) contains major sutures and crustal thrusts in the Hellenides (encircled numbers) and some thrust units (letters).(1) is the early Mesozoic eastward obduction onto Rhodope (R).(2) is the circum-Rhodope suture (CR), possibly identical with the main Tethyan suture.(V) is the Vardar zone. (3) is the Eo-hellenic (EOH) obduction,possibly the main Tethyan suture.(4)is the thrust below Pelagonian unit (PEL),interpreted as major Eocene subduction. (5) is the Pindos thrust,either a scar of a closed local basin or a prograded extension of sub-duction (4). (6) is a intracrustal shear at the Phyllite zone (PH) and the Gavrovo-Tripolitza (GT) zone.(7) is a presumed thrust at the base of the Pre-Apulian-Ionian zone (PA/I).B,C,D are crustal sections from the Hellenic trench across Peloponnese and across the island of Aegina.Thick lines are the positions of the M-discontinuity.Thin lines are the interpretive connections. (B) is the interpretation of a northeast-dipping subduction. (C) is the interpretation of the Plio-Pleistocene "flip-like" dissection of the Benioff zone by a gently west-dipping intracrustal shear. (D) is the interpretation of an arc-arc collision.

This double event may represent either one or two separate ocean basins and their subduction. Further progradation closed an intracratonic trough containing the phyllite zone and little or no oceanic crust. Present subduction contains mixed crust in the lower plate, perhaps similar to pre-orogenic crustal conditions in the phyllite zone.

It is supposed that initially the African plate was moved eastwards in relation to the European plate, and later was vectored north-westwards, and this motion is continued even today (Fig.3.5_A).

Movements of a compressional nature, at nearly northerly azimuths ((Le Pichon et al.,1973), and at rates of 1.9 to 2.6 cm/year, have been measured between the African and Eurasian plates. According to McKenzie (op cit), several zones of extension and compression exist in the active tectonics of the Mediterranean region. Those zones relative to Greece are shown in Figure 3.5_B.

Seismological evidence has shown that between the African and the Eurasian plates, several smaller plates exist, namely, the Adriatic plate, the Ionian Plate, the Aegean plate, the Levantine plate and others (see Fig.3.6_B), which are in the area of Greece.

Movements of the Levantine plate relative to Africa cause a south-westerly convergence of an estimated 3.5 cm/year across the Hellenic trench (Fig.3.6_A), which has been shown to fit well with the dimensions of the Hellenic Benioff zone (Roeder, 1978; Leydeker et al., 1978).

The tectonic zones of the southern margins of Europe (which include Greece) are delineated through "transform faults", or "peripheral troughs" such as the Hellenic trough in which the deepest point of the Mediterranean sea exists to the west of the Peloponese.

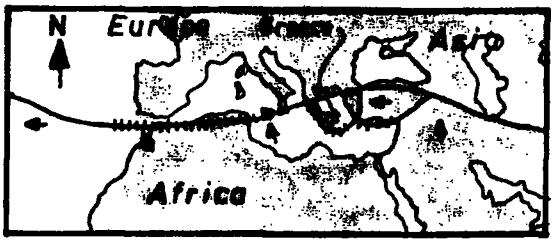


Fig. 3.5A *Plate tectonics of the Mediterranean region (after McKenzie, 1970).*

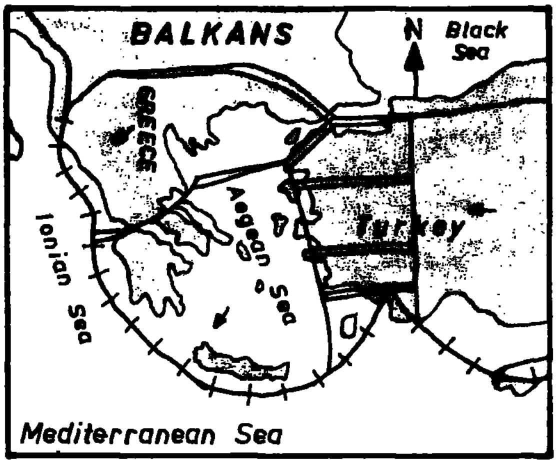


Fig. 3.5B *Active tectonics of the Mediterranean region.*

═══════ Zones of extension.

┼┼┼┼┼┼┼┼ Zones of compression.

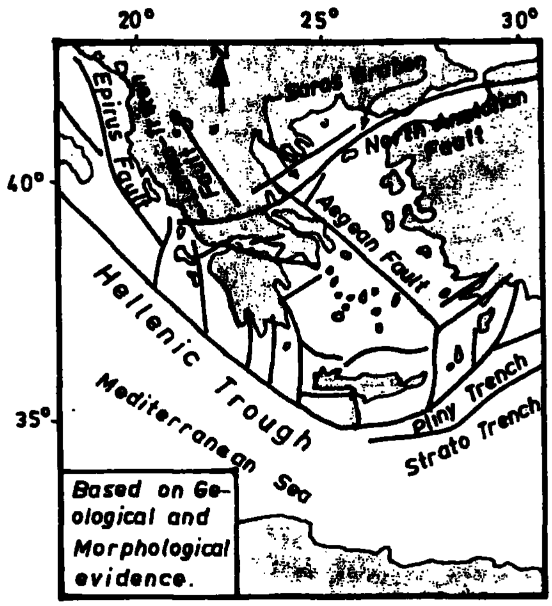


Fig. 3.6A *Tectonic fabric of Greece (after Galanopoulos, 1973).*

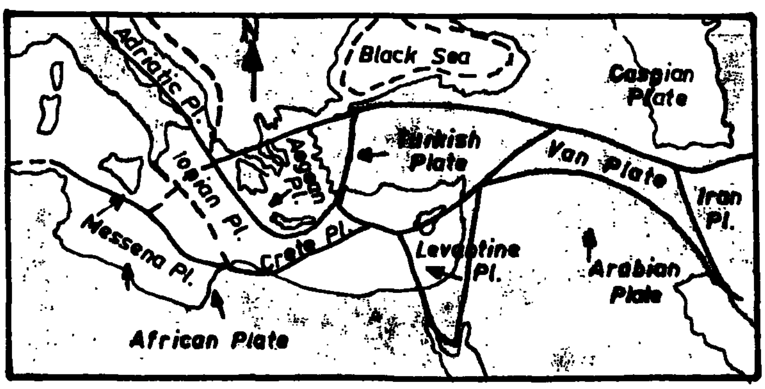


Fig. 3.6B
Plates in Eastern Mediterranean region (after Galanopoulos, 1973).

It is accepted today that the relative movements of the smaller plates of the eastern Mediterranean are caused by the northward movement of the Arabian plate thrusting the Turkish plate westwards, through the Van plate, along the transform North-Anatolian fault. Thus, the Aegean plate (and internal Greek zones) have moved westwards, according to Galanopoulos (1973).

Geomorphological and palaeontological evidence suggest that updoming of the Cyclades region, in the central Aegean sea, is still continuing (Sabot and Papanicolaou, 1976; Schuilihg, 1969, 1973; Makris, 1973, 1977, 1978). This probably explains why north of the Cyclades, deposits of the Tyrrhenian are below sea level in general, while on the south Aegean islands, on the Peloponnese and on the Ionian islands they are elevated. These movements are considered as a continuation of the late Neogene tectonics. As Aubouin (1973) has stated, they must be understood as one stage of contemporary neotectonic activities within an already existing orogenetic development.

3.1.4 General seismology and tectonics

Because of the potentially disastrous effects of any post-construction dynamic stressing on dams, it is necessary to review the seismic activity of Greece.

Greece is considered to be the most seismically-active region of Europe. It has been estimated that the released seismic energy in Greece is about 2 per cent of the total seismic energy released throughout the world (Galanopoulos, 1965).

The Hellenic arc system resembles Pacific island arc structures, by showing high earthquake activity, a trench on the convex side, an inner volcanic belt active in historic time, large free air anomalies and a marginal sea, the Aegean sea (interpretation based on morphological, geological and geophysical data after Jacoby et al., 1978; Leydecker et al.,1978). But the diffusion of shallow and intermediate earthquakes (Papazachos and Comninakis, 1971) is not consistent with what would be expected of an island arc Benioff zone.

From an analysis of observations of the seismic activity throughout the Greek territories, Galanopoulos (1963) concluded that, for the period 1843-1962, there was an average occurrence of eleven earthquakes annually of $M \geqslant 5$, eight earthquakes of $M \geqslant 6$ each 5 years, twelve earthquakes of $M \geqslant 7$ every 50 years, and three to four earthquakes of $M \geqslant 8$ every one hundred years.

It has been confirmed that in Greece there are 800 active epicentres. Throughout the period 1951-1963, there were recorded earthquakes from 620 epicentres (Drakopoulos, 1974, 1976). Despite the high earthquake activity in the Greek territories, most of the earthquakes have not been accompanied by observable surface faulting, as, for example, in neighbouring Turkey. Drakopoulos (1974) reports two cases in which surface faulting was noticed; one after the shock of Atalanti (central Greece) in April 1894 of M = 6.9 and another in Chalkidike (near Thessaloniki) in 1932. This latter had a shock of M = 6.9. A third shock in 1978 (which damaged Thessaloniki) of M just greater than six was accompanied by surface faulting, near to the north-easterly located lake of Volvi (Chalkidike area). Finally the 1981 earthquake of $M \approx 6.8$ of Richter scale which damaged Athens produced surface faulting, near Loutraki north-west of Athens, of 40 cms of vertical offset (AEG-newsletter,1981).

The relative movements of the Eurasiatic and the African plates, and the resulting rearrangements of the smaller plates surrounding the Aegean plate, are considered to be responsible for the intense earthquake activity in the area of Greece.

According to Pavoni (1961), along a major fractured zone trending from the west to the east (between Meridians 17E to 40E- a main expression of which is the North-Anatolian fault in Turkey),there is a horizontal displacement of about 400 km, which began in Tertiary times and is continuing today at a rate of 1 to 2 mm per year (2.6 cm/yr for the southern Aegean region, after Richter and Strobach, 1978). The main expression of this zone in Greece is the Saros Graben in the northern Aegean sea, which extends through the Volos area to the Lamia-Trikkeri fault zone and reaches the Ambrakikos Gulf in the Ionian sea (see Figs.3.6$_{A,B}$).

Accepting the fact that an earthquake is the result of a sudden release of strain energy due to rupture along a fault surface, the cumulative strain energy released annually in the general territorial area of Greece has been estimated (Galanopoulos, 1972 for earthquakes of the period 1950-1972) to correspond to a single earthquake of magnitude $7\frac{1}{2}$ annually.

The recurrence period of the accumulation of strain is $5\frac{1}{2}$ years, and the maximum amount of strain energy accumulation corresponds to an earthquake of magnitude $8\frac{1}{2}$ annually, released as a single event.

Galanopoulos (1965) suggests, on the basic of the distribution of the earthquake epicentres occurring in the period 1951-1962, that the conjugate fault-systems, trending NNW-SSE and WSW-ENE and created during the Miocene and after, are responsible for the major part of the earthquake activity in the area of Greece (Fig. 3.7). The WSW-ENE system of faults is considered as a prolongation of the north-Anatolian strike-slip-fault into Greece. In all the epicentre maps of Greece two major fault zones are distinguished as the sources of higher earthquake activity in the arc of Greece. One is the Cephalonian fault zone (Ionian islands) and the other is the Crete-Dodecanese islands area of the southern Aegean sea.

Substantial differences have been revealed in the stress field (see Fig. 3.8) of these two centres of higher earthquake activity (Galanopoulos, 1973; Richter and Strobach, 1978; Papazachos 1979). The Ionian centre is a site of predominantly shallow continuous earthquake activity, with a pronounced low of negative Bouguer anomalies.

The Crete-Dodecanese centre is a region in which a high and rather continuous intermediate epicentral depth of earthquake activity occurs, and high positive gravity anomalies are recorded (Drakopoulos, 1976).

In general, all the epicentre maps of Greece indicate some relatively aseismic areas which according to Drakopoulos (1976) are:

a) The Aegean block, which McKenzie (1970) calls the Aegean subplate.

b) The block formed by north-eastern Greece, southern Yugoslavia and Bulgaria.

c) The Ptolemais basin in northern Greece (see Fig.3.13).

It is also evident that almost all the large shocks of inter-

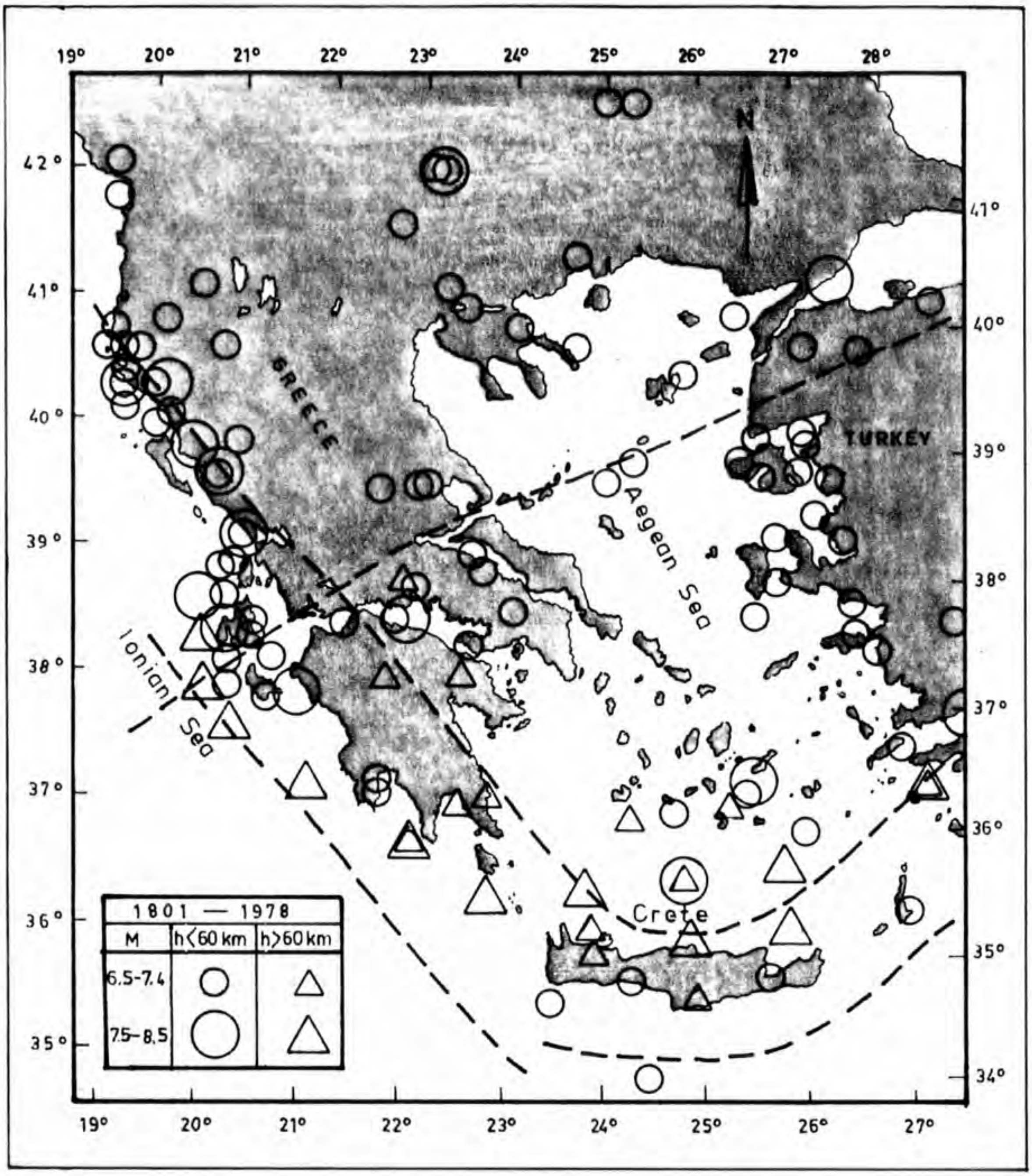


Fig. 3.7 *Seismotectonic map showing the large conjugate-fault system of Greece (after Galanopoulos, 1965). Earthquake epicenters during the 19th and 20th centuries (after Papazachos, 1979).*

MD

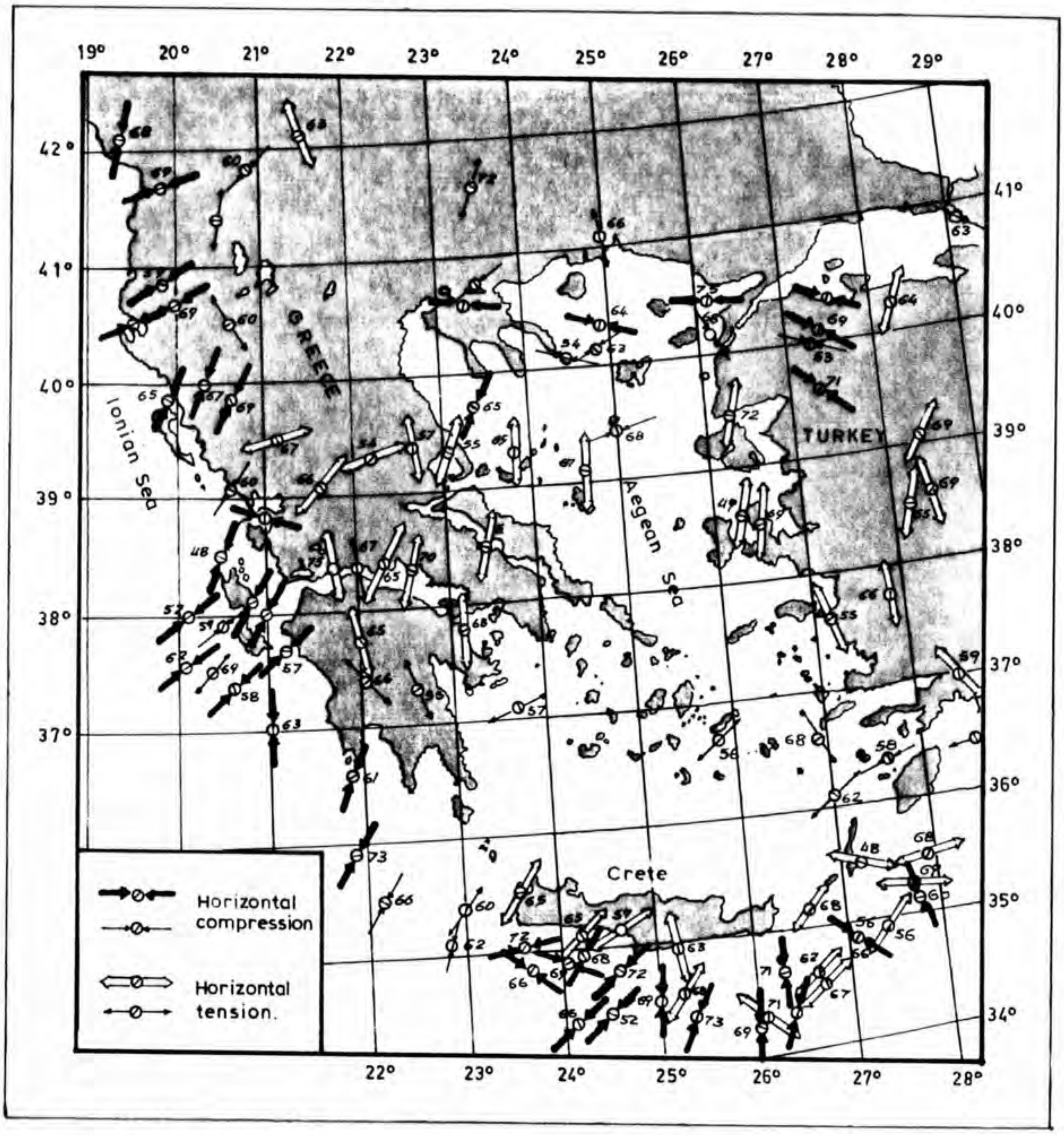


Fig. 3.8 *Horizontal stresses in Aegean area from the earthquake genesis mechanisms (after Papazachos, 1979).*

mediate depth occur south of 38.5^{o}N latitude (Fig. 3.7).

3.1.5 Geomorphology

Greece exhibits a youthful topography. Ridge lines commonly have a "sawtooth" aspect; valley slopes are steep and in many cases are only in quasi-equilibrium. Rapid stratigraphic changes from soft to hard rocks, with the assistance of climatic aggression, have traced Greece with certain erosion lines, which in most cases comprise river courses.

The main rivers trend approximately N to S, E to W and W to E, with steep gradients (see Figures 2.1 and 2.3). Their flow is usually torrential, and they run through deeply-incised canyons to outflow in flood planes near the sea. Creation of such flood planes took place during historic time; the Capital of Alexander the Great, which is now located quite a few kilometres away from the sea, was a port in the 3rd century B.C.

In some cases the rivers fail to establish normal drainage patterns (in the face of intense tectonic activity). The result is enclosed basins,as in the case of the lakes of Amvrakia and Ioannina, or successive terraces with lacustrine deposits (Kozani area basins-Aliakmon river).In those cases in which the rivers broke through the surrounding mountains, they exploited the presence of fault lines to create steep gorges such as the lower Aliakmon gorge, in which three successive dams are located (Polyphyton dam, Sfikia dam and Assomata dam; the last two being still under construction at the time of writing and being the particular subjects of later study in this Thesis).

Extensive karstification of calcareous strata is another natural phenomenon reflecting the palaeoclimatic conditions of the country and suggesting in many cases that strong tectonic movements have submerged wide calcareous areas. Karstic springs give out a considerable amount of water and in some cases they support small hydroelectric projects, like Louros (10 MW installed capacity), or possible substantial volumes of water to the main rivers.

The Smardacha springs in the Acheloos river upstream of the Kremasta dam are an example of this, having a mean spring flow, for the dry period of July-November, of $18m^3/sec$ (Liakouris, 1971). On the other hand, the Perdika dam is dry as a result of such karstification.

Strong erosion is still going on in Greece. In many cases the thickness of alluvial gravel exceeds 50m (for example the Stratos Dam site on the Acheloos river)or is minimal (a few metres or none at all in narrow gorges with steep gradients).

This suggests that aggrading processes are taking place. In other cases, landslides and talus are in the process of degrading steep valley flanks and canyons. As an example, and according to Liakouris (1971), three terraces studied in the lower Acheloos area were developed by upward movements of the major area of central-western Greece.

3.2 Geology and seismicity of North Western Greece

3.2.1 Introduction

The area which forms the specific subject-matter of this thesis is located within the main body of Greece, that is, west of Thessaloniki as far as the Ionian Sea, and north of the Corinthian

Gulf to the Greek borders with Albania and Yugoslavia. It comprises the Arachthos, Acheloos, Aoos and Aliakmon rivers among others (Fig. 2.1), in which rivers the Pournari dam, the Assomata and Sfikia dams will specifically be reported in this work(see also Table 2.8) while other dams, like Pigae of Aoos, Kremasta, Kastraki, Stratos, will be mentioned appropriately.

3.2.2 Geological setting

The nature of the deep basement rocks of the Ionian, Gavrovon and Pindos zones and generally of the central-western area are unknown. The oldest rocks identified are no older than Triassic (B.P. Co Ltd, 1971).

Older rocks have been identified elsewhere in the internal Greek zones, examples being the fossiliferous Permian rocks near Athens and on the island of Euboea (north-east of Athens), and the Devonian rocks in Khios island.

The B.P deep boreholes (3 323m) put down in this region (Ionian and Gavrovon zones) as well as surface evidence, lead to the hypothesis that, chiefly during the Miocene-Pindic orogeny, the thick (1800 m) evaporites underlying the formations of the above zones, cut by a complex of low angle thrusts together with the overlying carbonates and flysch, moved westwards, were folded and broken with steep faults in big masses and piled, one on top of the other, in similar (stratigraphically) superposed series of slices (Gill, 1964; Jones, 1968; B.P, 1971). Thus the rocks in the region range from Triassic to recent, comprising evaporites (age-dated from the overlying carbonates), a carbonitic (limestones, dolomites) sequence which includes siliceous rocks (cherts mainly), flysch sediments ranging from shales to conglomerates, and the recent sediments (alluvial, marls and so on).

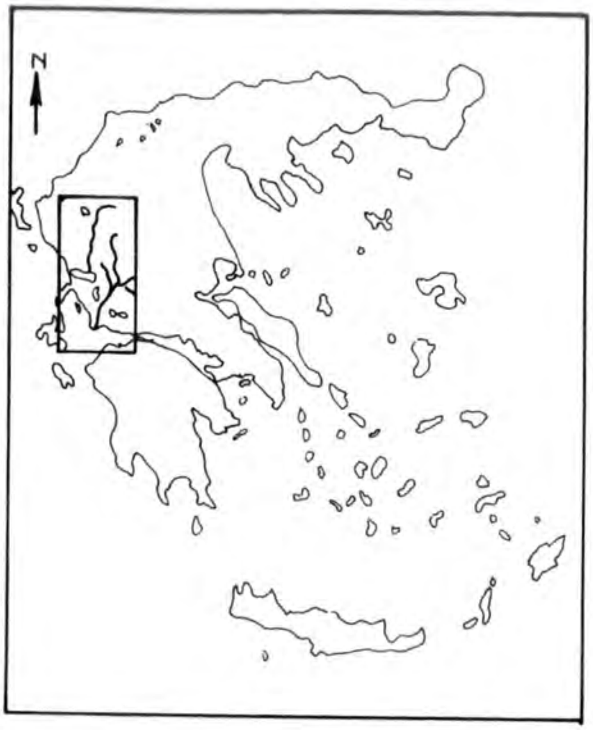
The main stratigraphic sequences strike NNW and dip mainly eastwards (see Fig. 3.9). Horizontal displacements have been recognised, as a result of thrusting, in the area of the Megdhovas river, while vertical down-thrown movements of 2 km have been noticed in the Astakos area as a combination of the Astakos anticline and the nearby fault (B.P. 1971).

The Pindos zone is characterized by a pelagic sequence of cherty limestone and radiolarites, interrupted by the first Pindic flysch of Barremian to Turonian age and ending with the internal Palaeogene flysch (Richter et al.,1978; see Fig. 3.10).

The Pindos zone sedimentary formations of Upper Eocene age have been overridden by the Pindos basic-ultrabasic rocks of Jurassic age which, by tectogenesis events spreading from the east to the west, were overthrust in a westerly direction on the younger Pindos flysch.

The inner parts (eastward margins) of the Pindos zone were affected by the Eocene tectogenesis. Thus Pindos clasts have occurred in the Lower Oligocene Molasse of the Mesohellenic trough (Brumm,1956).In the Middle Miocene the Pindos rocks are considered overthrust on to the external zones. According to Doert (1976), this is concluded from the internal deformation observed in the Tripolitza flysch.

These westward movements are considered to be affected partly by gravitational gliding (Temple, 1968) induced by an uplift of the central Aegean area. The displacement of the Eocene Pindos nappe, in a westerly direction, has been estimated to exceed a travel of 100 km. This is due to similarities identified



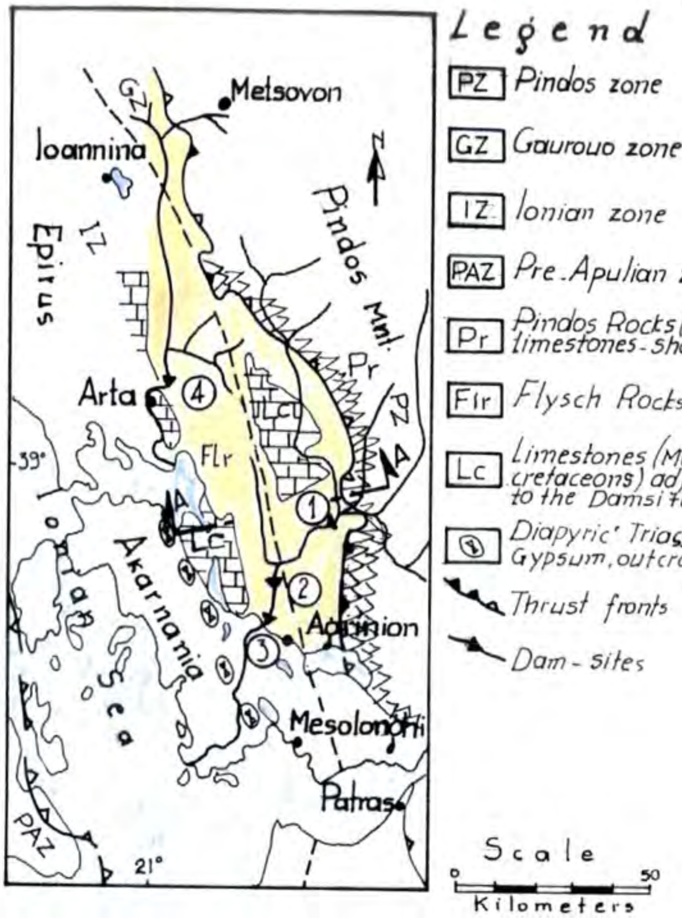
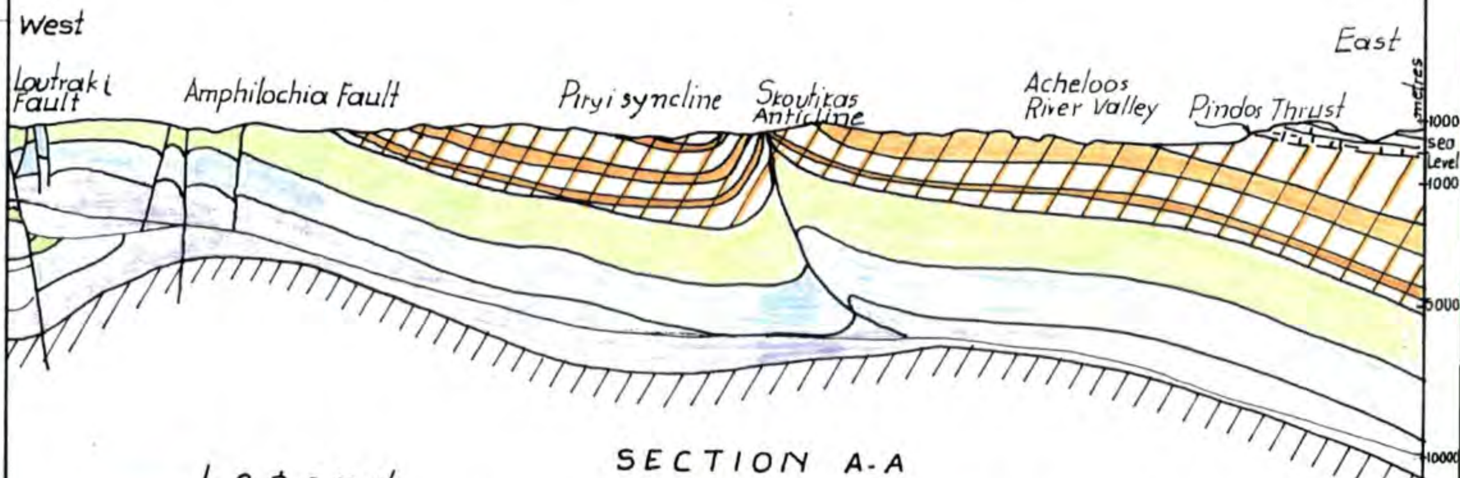


Fig. 3.9 Flysch formations and locations of the main dams in western Greece. (1) kremasta Dam (2) kastraki Dam, (3) Stratos Damsite, (4) Pournari Dam. (Information compiled from various sources).



SECTION A-A

Legend

- Miocene Marls and Flysch Sandstones etc
- Eocene to Ammonitico Rosso (limest.)
- Trias-Lias Carbonates
- Trias Evaporitic Series which icludes black limestones gypsum and limestone breccia
- Pre-Evaporites? Upper Palaeozoic

(after B.P Co, 1971)

Scale 1:250,000

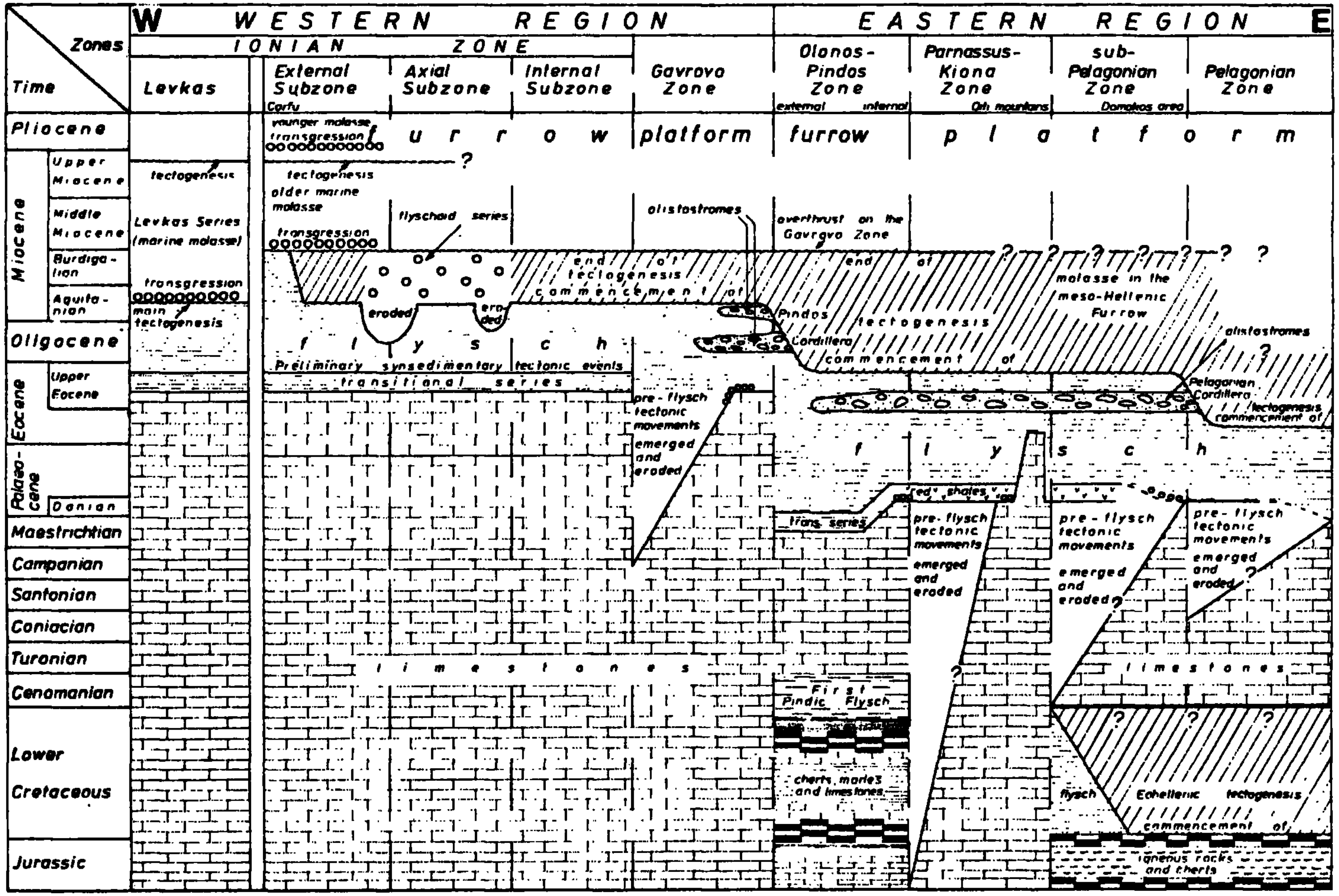


Fig. 3·10 Diagram illustrating the pre-flysch development and the flysch periods of isopic zones in continental Greece (after Richter et al., 1978) between the rocks of the Mt.Olympus tectonic window (see Figs. 3.1 and Fig. 3.2) and the rocks of the Tripolitza zone (Fleury and Godfriaux,1975).

The successive orogenic events, up to the Miocene, which affected the Pindos, the Gavrovon and Ionian zones, as mentioned previously, originated within the inner Hellenic zones; that is, the Pelagonian, the Vardar and the Servomacedonian zones (see Fig. 3.3).

Several generations of ophiolitic rocks are present within those zones. Successive immersions, uplifts and upthrusts within those zones (see Figs. 3.3 and 3.13) combined with several generations of volcanic and intrusive magmatic action (Vergely, 1976) resulted in a complex stratigraphical and lithological appearance in the lower canyon of the Aliakmon river in which the Sfikia and Assomata projects lie. Successive orogenic events (obduction) placed ophiolitic sheets, mixed with pelagic sediments thrust from the Vardar ocean over the Pelagonian massif, throughout the Mesozoic and early Cenozoic up to Miocene (Tertiary).

Subsequent vertical and horizontal tectonism during the Plio-Pleistocene period with development of successive terraces (along the Aliakmon Canyon) and the Veria-Thessaloniki plane (depression) with recent sediments suggests continuous movements up to the present day.

3.2.3 The flysch formations.

The main Tertiary flysch formations in Greece are divided into two regions, the eastern and the western. The eastern region comprises the Olonosa Pindos zone, the Parnassos-Ghiona zone, the sub-Pelagonian and Pelagonian zones, the latest one being identical with the minor appearance of Mestrichtian flysch within the Almopia subzone of the Vardar zone. The western region comprises the flysch formations of the Ionian and Gavrovo zones (see Fig. 3.10, after Richter et al., 1978).

The eastern region flysch began in the latest Cretaceous period and is considered to have ended in the Upper Eocene.

In the western region the flysch deposition started later in the Upper Eocene and continued in to the lower Miocene. This implies that, during the flysch development phase, the uplifting tectogenesis movements began in eastern regions first and continued in steps westwards thereafter. The exceptions to this are the Ionian basin and the Olonos - Pindos trough where uplift block faulting and erosion preceded the flysch stage in most areas.

3.2.3.1 The flysch of the Ionian and Gavrovo zones.

The term flysch has been used as a stratigraphical term reflecting several lithological units such as, sandstones, marls, shales, clays, conglomerates and other facies. As first used by B.Studer (1972), it means geological facies in which the sedimentation is followed by an intense tectonism.

The Ionian and Gavrovo flysch comprises several clastic sediments, that is, clayey marls, claystones, siltstones, sandstones and conglomerates, which were deposited during the last period of the life-time of the Gavrovo and Ionian sedimentation areas (Liakouris, 1971). It is worth noting that transitional zones exist in which the above facies are alternated with other rocks (including limestone or siliceous beds such as chert). Aubouin (1959), explaining a possible palaeogeographic evolution of the Hellinides, has defined the Greek flysch as a facies which has filled up a geocyncline and which was brought to an end by an orogenic paroxysm. He terms the Gavrovo flysch as a "second degree flysch", which has originated from the erosion of the Pindos flysch, itself termed a "first degree flysch". The Ionian flysch, the upper part of which originated from the erosion of Gavrovo flysch, is termed a "third degree flysch".

The Ionian zone flysch commences in the Upper Eocene with a pelitic-carbonate transitional series which overlies a pelagic limestone succession of Palaeogen age (Richter et al., 1978).

The thickness of the transitional series varies from about 60m in the western part of the Ionian zone (the external part), to about 10m at the eastern margin of this zone (the internal part). The widespread distribution of the transitional series indicates that deep-water conditions (pre-flysch) existed in the Ionian furrow before the true flysch facies. The flysch formations in the Ionian zone consist of mudstones, siltstones and graded sandstones (turbidites) and the thickness varies from 1000m in the west to 4000m in the east (Richter et al., 1978). Tectonic movements of the Oligocene age split (from N to S) the Ionian zone into three subzones in the Epirus region, each with its own facies. Flysch deposition ended in the Miocene, when the main tectogenesis started. Mollassic type sediments, later deposited unconformably on folded flysch, were marked with weak folding, indicating the continuation of tectonic events.

The flysch of the Gavrovo zone is similar to the internal Ionian subzone flysch, since the two flyschs have not been identified with any facies boundary markers. In the central and western regions of the Gavrovo massif the flysch lies transgressively on the rugged surface of the Tertiary or even Upper Cretaceous limestones. At the beginning of the flysch deposition in the Upper Eocene the western part of the Gavrovo massif rose strongly and resulted in extensive erosion and karstification of the pre-flysch limestones.

Huge blocks of Gavrovo rocks, recognized at the bottom of the Ionian flysch deposition furrow, suggest that the main tectogenesis of Gavrovo zone probably occurred during the early Miocene period and would seem to have coincided with the main folding of the Ionian zone.

3.2.3.2 Damsites on flysch formations of the Ionian and Gavrovo zones.

Damsites on flysch formations in western Greece are:

i) The Kremasta dam on the Acheloos river. The dam is located in the Gavrovo zone with part of the reservoir area extending into the Pindos zone (see Fig. 3.11 and Plate 1.4).

ii) The Kastraki dam on the Acheloos River. The dam is located in the Ionian zone with part of the reservoir area extending into the Gavrovo zone. It lies immediately downstream of the Kremasta dam (see Fig. 3.11).

iii) The Stratos damsite on the Acheloos River. The dam is located entirely in the Ionian zone (see Fig. 3.11), downstream of the Kastraki dam.

iv) The Pournari dam on the Arachthos River. The dam is located entirely in the Ionian zone (see Fig.3.12).

All these damsites and their reservoirs lie on flysch formations with the exception of Kremasta reservoir, the base of which includes limestones of the Gavrovo and the Pindos zones. Potential locations for damsites or reservoirs for future development are numerous in central and north-western Greece.

Flysch formations are sediments which have resulted from "quick" sedimentation processes. Within short distances one might expect to find abrupt changes of a cyclic alternation nature in the lithological units. It should be noted that "flysch" is a stratigraphical and not a petrological term.

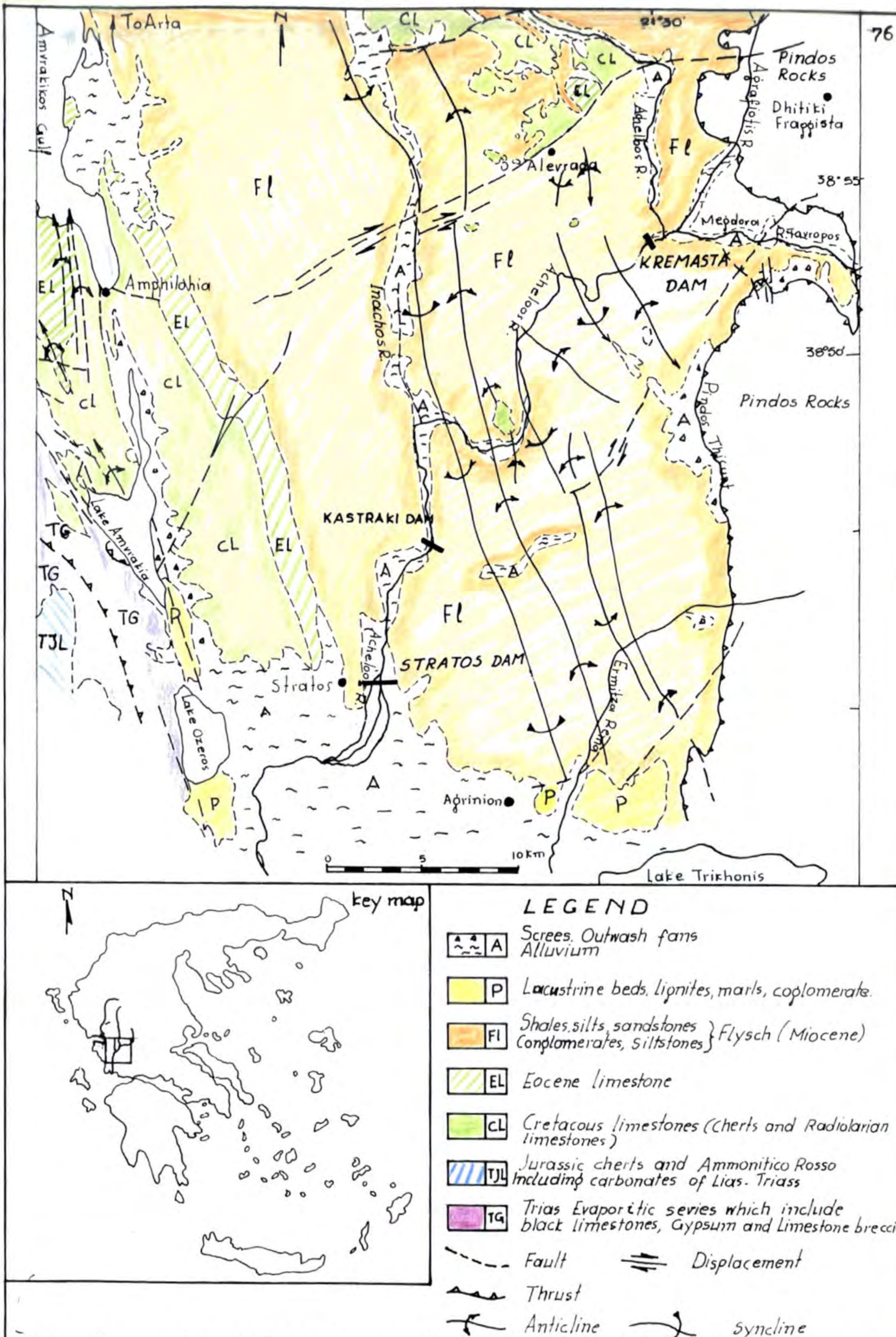
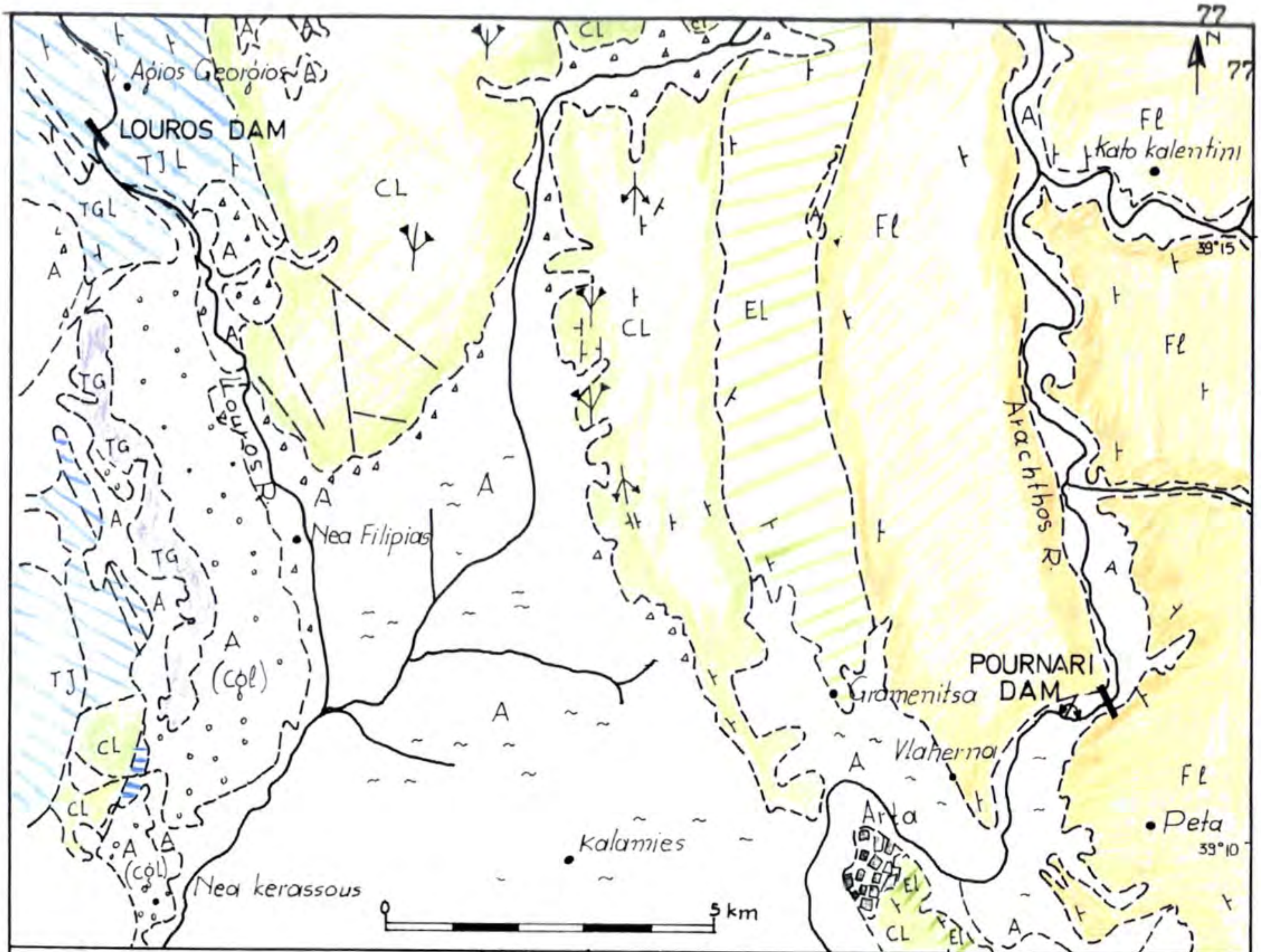
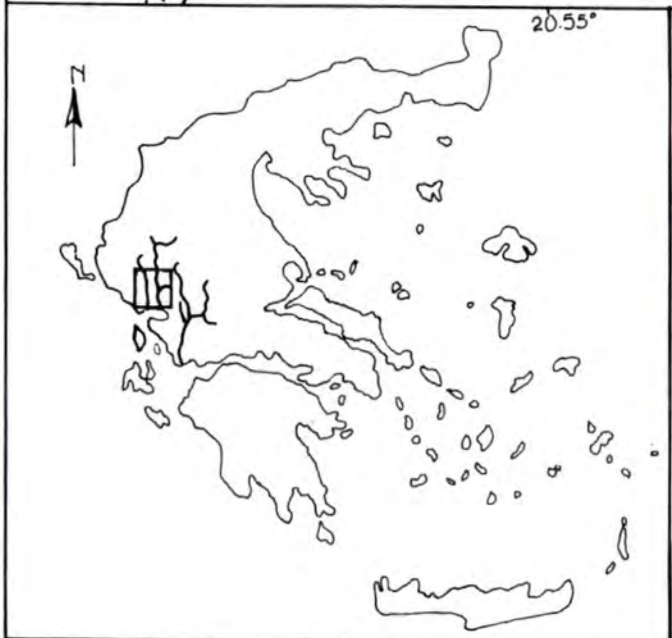


Fig.3.11 Regional geological setting of Kremasta, Kastraki and Stratos damsites (after B.P. Co, 1971)





Legend

A	Screes, Outwash fans, Talus fans, Terraces, Alluvium
A.Cöl.	Conglomerates and marly. sandy. clay (mixed Marine and continental deposits)
Fl.	Shales, silts sandstones, Conglomerates, siltstones } Flysch (Miocene)
El	Eocene Limestone
CL	Gretaceous Limestone (Cherts, Radiolarian Limestone)
TJL	Jurassic Limestones. Cherts including Carbonates of Lias - Trias
TG	Trias Evaporatic Series which include black limestones, Gypsum, Limestone breccia
Strike / dip	Bedding, Syncline, Anticline
---	Fault.

Fig 3-12 Regional Geological setting of Pournari and Louros damsites.
(after I.F.P., 1966)

Another dam, the Louros dam (Fig. 3.12), located about 10 km north of Arta town, lies on the basement limestone formations, which underlay the Ionian flysch of the Epirus-Akarnania syncline on which the Pournari dam, the Stratos damsite and the Kastraki dam are founded. The basement limestone formations are karstic along the entire line of the Ioannina-Arta-Amphilochia- Messolo-ghi sequence. The Louros reservoir accepts a steady flow from karstic springs originating in these karstic limestones.

A. <u>The Pournari and Stratos damsites</u>

The Pournari damsite on the Arachthos River and the Stratos damsite on the Acheloos River lie on flysch formations composing the western limb of the Epirus-Akarnania syncline.The flysch overlies the fine-grained Eocene limestones of the Ionian zone, the limestones being considerably karstified.Because of its great thickness and extent the flysch comprises the immediate bedrock area of all these reservoirs. The rivers trend about N-S upstream from the dam axes. Their stream courses have created an erosion valley along a thinly-bedded sequence of siltstone-sandstone, but in the case of Pournari some silty conglomerates exist.

The Pournari dam-axis trends NNW, parallel to the strike of the beds which are dipping eastwards (upstream), in a location where the river bends westwards through a down-cut erosion channel. From the upper to lower horizons the following rock series have been incised:

Thick sandstone beds

Siltstone and silty-conglomerates (melange)

Thinly-bedded siltstone-sandstone sequence.

Both abutments of the Pournari dam exhibit the same stratigraphic columns. The river width at the dam axis is about 300m, while the thickness of river bed alluvium (silty-sandy gravel and cobbles) varies from zero to a maximum depth of about 22m.

The Stratos dam-axis trends E-W along the line of bedding full-dip, the beds dipping in an easterly direction. The flysch at this site comprises an alternation of siltstone and sandstones, from thin beds (several cms) to thick beds (up to several metres). Thick sandstone beds on the left (eastern) bank contain bands of coarsely-grained sandstone up to conglomeratic (small gravel) size-range. The river width at the dam axis is about 2000 m and the alluvial river bed varies in depth from a maximum of about 50m.

Structure

In regional terms, no major structural features have been identified at the damsites.The bedding dips, moderately to steeply, eastwards, although there are local variations in steepness and occasional reversals of the dip directions, suggesting the existence of structural folds. After excavations at Pournari, some folds (one in the river bedrock at the dam axis; see Fig. 4.1) have been revealed, plunging north-eastwards. Their existence in the soft lithological members suggests that they must be considered as secondary superimposed structures on the main N-S trending regional folds (for example, the Epirus-Akarnania syncline).

No faults have been observed in the immediate damsites, but there are some nearby. Main joint sets also exist, occasionally cutting through the stratigraphic units of the sites, but are not considered to be continuous over long distances.

One such joint set, striking about E-W, and dipping steeply southwards or northwards, is considered responsible for the erosional westward bend of the river Arachthos at the Pournari site. The relation of the joint sets with the fold revealed in the dam axis will be discussed subsequently in Chapter 4.

Details of the dam foundations and the treatment applied to them will also be discussed.

B. Kastraki and Kremasta damsites

Kastraki dam on the Acheloos River lies on the upper Ionian flysch, and part of its reservoir is extended into the Gavrovo flysch. It is located a few kilometres upstream from the Stratos damsite, and downstream from the Kremasta dam. Its axis is aligned NW-SE in a location where the river bends westwards and its bedrock foundation consists of siltstone and sandstone alternations. Upstream and along the trend of the Inachos river, a tributary of the Acheloos River, the Gavrovo zone appears to be thrust on to the Ionian zone. No major structural features are present at the site.

The Kremasta dam lies upstream of the Kastraki dam, in a deeply-incised canyon of the Gavrovo flysch, with part of its reservoir extending into the Pindos zone. The Pindos zone main thrust is present east of the damsite where the Megdovas tributary river meets the main Acheloos river. In the Kremasta reservoir limestone members of the Gavrovo and Pindos zone are present.

This dam is widely discussed in the literature for two main reasons:

a) Seismicity occurred in the region after the reservoir was filled.

b) Leakages appeared during the filling of the reservoir and subsequently, even after remedial works were carried out at the site. The present level of leakages are estimated at about 1.5 m^3/sec.(Coumoulos and Therianos, 1977).

The dam axis is aligned obliquely to the strike of the beds, with the beds dipping gently upstream.

A combination of minor faults crossing the damsite, together with the calcareous conglomeritic horizons which are present in the river bed and the abutments, are considered to be the cause of the leakages. During the remedial grouting and drainage works deep weathering was observed, and a 16 m cavern was discovered, suggesting karstification of the conglomerates (Snow, 1972). The situation is made worse because of the presence of sulphurous hot springs at the dam foundation and the abutments. Hot water out-flowed from the drainage galleries, and signs of corrosion of the concrete structures have been observed.

3.2.4 Damsites in formations other than the flysch:Assomata and Sfikia dams on the Aliakmon River

The Assomata and Sfikia damsites lie together with the Polyphyton dam in the lower Aliakmon River gorge near the town of Veria west of Thessaloniki (see Fig. 2.1 and Fig. 3.13).

3.2.4.1 The regional geology and geomorphology with particular reference to the geology of the damsites

The lower Aliakmon River valley forms the south-eastern boundary connection between the Kozani Neogene and the recent depression of Thessaloniki flood plain and has been created by retrogressive erosion after a regional uplift of the Vardar and Pelagonian zones.

The Vardar zone, and in particular the Almopia subzone towards the west, includes the Assomata damsite and its reservoir area (see Fig. 3.13 and Table 3.1).

The Sfikia damsite and its reservoir is located in the eastern portion of the Pelagonian massif. The border line between the Vardar zone and Pelagonian zone lies 1 km downstream of the Sfikia damsite and is the offset of a thrust fault. It is expressed as a wide zone (in some cases over a kilometre) of fault breccia or mylonite. It includes several altered and brecciated or disturbed rocks of various origins such as volcanic, metamorphosed or sedimentary ones. Near the Sfikia site a front of thrusted and highly disturbed limestones is recognized as the thust line which strikes NW-SE and dips up to 25^{o} NE.

The Almopia subzone, horizontally from the downstream end of the canyon (a kilometre downstream from the Assomata site) up to

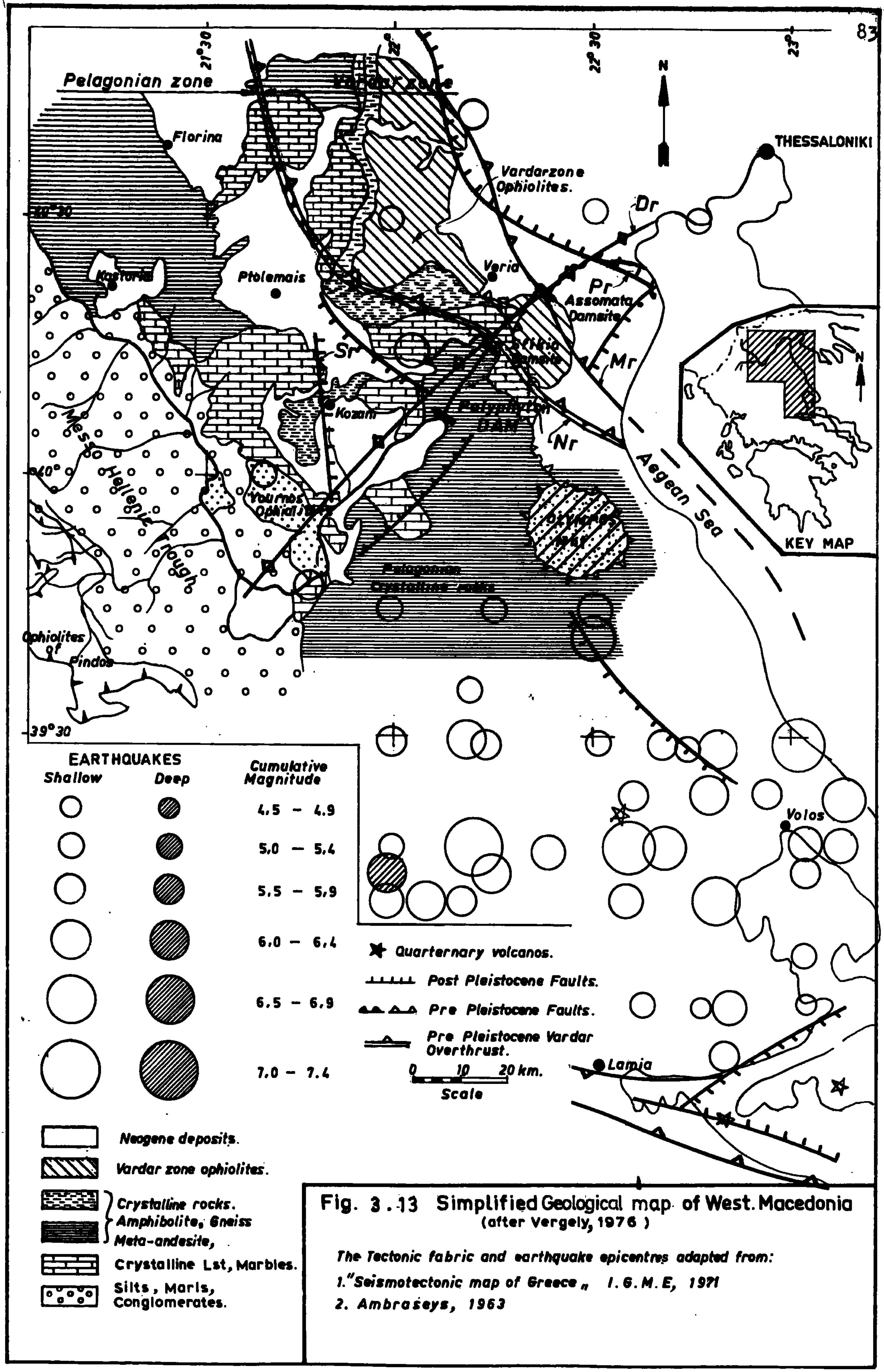


Fig. 3.13 Simplified Geological map of West. Macedonia (after Vergely, 1976)

The Tectonic fabric and earthquake epicentres adapted from:

1. "Seismotectonic map of Greece" I.G.M.E, 1971

2. Ambraseys, 1963

the thrust line which separates the Vardar and Pelagonian zones, as well as vertically from the river bed up to the top of the valley flanks (several hundred metres in height), includes several rock formations in a rather mixed facies as follows:

Vardar basal formation rocks are represented downstream from the Assomata dam site by mica schists, gneissic schists and crystalline limestones, the latter outcropping near the village of Assomata, of Triassic and Jurassic age. It is believed that these strata are underlain by Palaeozoic crystalline schists (Aliakmon study Group, 1963).

From the Jurassic up to the Cretaceous age, several strata are included as basic and intermediate volcanic rocks intruded into shales (phyllites), calcareous schists, platy or thickly-bedded limestones in successive - or mixed-up - facies. The intrusive rocks include pyroxenites, peridotites and dioritic rocks among others, while volcanic rocks include andesites, andesitic and tuff agglomerates. Clastic sediments within the magma suggest an immersed volcanism. Immediately upstream of the Assomata damsite, thick limestone strata appear embedded (floating) within the magma, while volcanic agglomerates exist there, including older sea-bed debris of intrusive and sedimentary rocks.

The volcanic rocks at the damsite appear to be extensively sheared, hydrothermally altered and transformed into serpentinites with local variations of schistose rocks, talc, asbestos, chlorites among others, while distinct horizons of tuff agglomerates exist as younger intrusions.

Uppermost Cretaceous, Tertiary and Pleio-Pleistocene deposits are represented by grey-greenish conglomerates (transition) and slightly metamorphosed shales and flyschs (Maistrichtian). Conglomerates and red clays and local tufa have formed several terraces along the canyon flanks.

An Assomata, Veria, Naoussa, Edessa line (having a N-S direction) of low terraces underlain by travertine deposites suggests a fault line escarpment in the same direction separating the highlands of the west from the Veria-Thessaloniki depression plain in the east.

Within the Sfikia project area rock formations appear less complex. From the upstream end of the Sfikia reservoir and towards the damsite and the Pelagonian-Vardar border line a sequence of the following rock series can be presented:

a. A series of metamorphosed meta-andesites, gneiss-schist rocks of volcanic origin with granitic intrusions probably of pre-Mesozoic age. These rocks occur also in the proximity of the Polyphyton dam.A reconnaisance at high elevations of the left abutment above the Sfikia damsite showed the granodioritic formations at an elevation of 700 m above sea level in Mikri Santa. At the right abutment, Sfikia village has been built on a terraced plateau at an elevation of 600 m approximately. At lower elevations, terraces are also observed downstream of the Sfikia damsite.

b. A series of undifferentiated meta-andesites with amphibolite gneissic rocks, and several instrusions of pegmatite aplitic veins along foliations and minor faulting, with a major dyke of gabbro and amphibole rocks marking the major (so called powerhouse) fault, are formations present near the riverbed and at the damsite area.

c. Some marble and limestone strata embedded within gneissic rocks and along certain fault lines, mixed up with minor Vardar zone rocks, are located downstream of the Sfikia dam, marking a set of thrust slices in the proximity of the border line of the Pelagonian and Vardar zones.

3.2.4.2 Regional structure and faults

The regional structure in the Sfikia and Assomata projects is dominated by the regional faults which have affected the formation of the eastern part of the Pelagonian zone and the formations of the western Vardar zone overthrusted on it.

These faults can be divided into two general categories. The first trends NNW-SSE dipping 10° - 30° NE and with a general displacement toward WSW, and the second trends SW-NE dipping steeply (70°-80°). Among the regional faults (Ambraseys, 1963) fault Nr, (see Fig. 3.13), represents the overthrust of the Vardar zone on the Pelagonian zone. This fault is considered pre-Pleistocene and is followed by a family of such thrust faults, which at least in the Assomata site are well marked by a set of minor such faults with a frequency of 30-60m.

Fault Dr is also of particular importance. It forms the southern boundary of the Neogene to Recent depression of the Kozani Plateau and has facilitated the river course in the lower Aliakmon gorge.

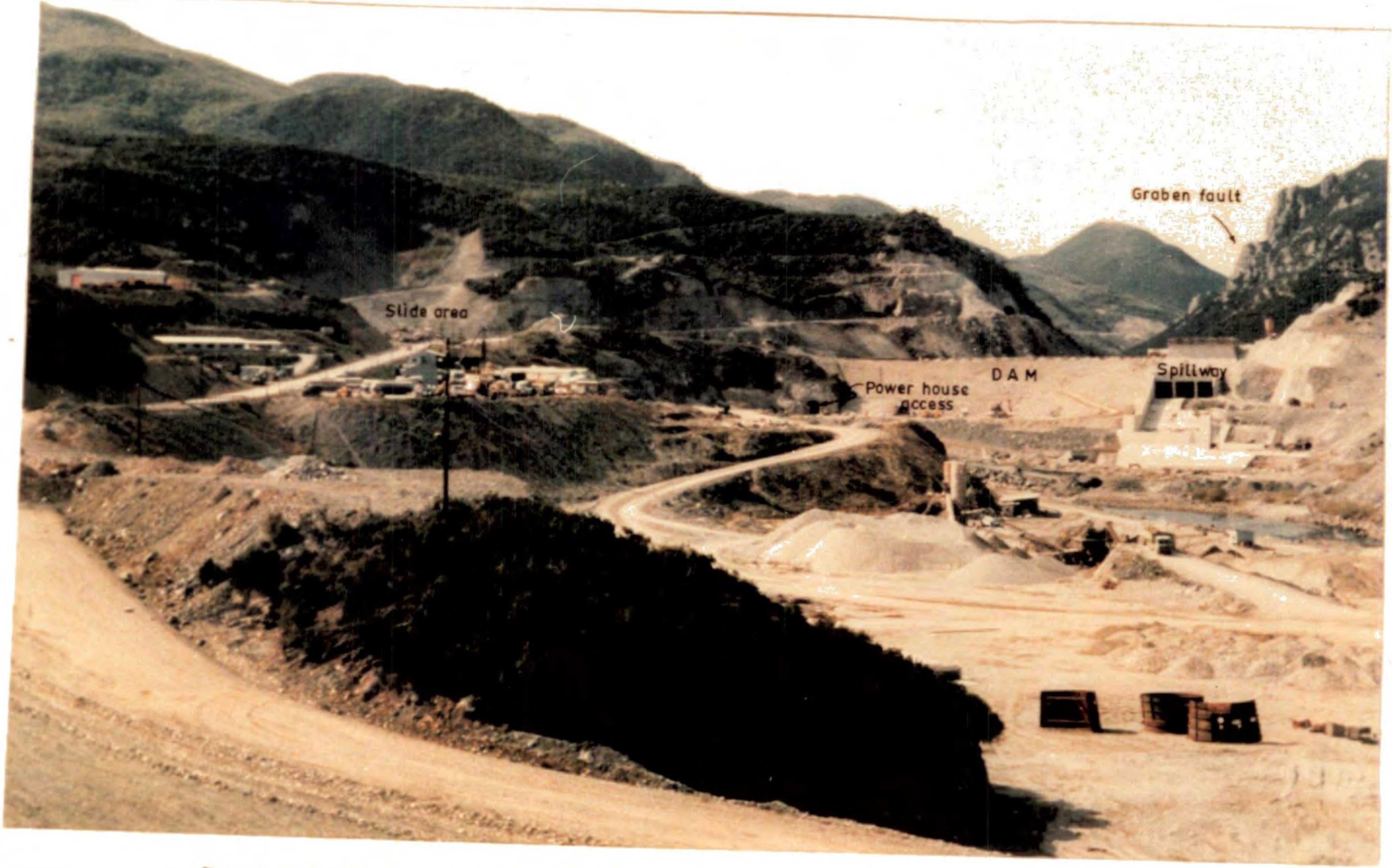


Plate 3.1 Lower Aliakmon River gorge at Assomata damsite.
(Graben fault structure upstream of the dam)

Plate 3.2 Recent movements observed in faults downstream of Assomata damsite.

It is noted (Ambraseys, 1963; Galanopoulos,1967) that, upstream of Polyphyton, fault Dr has received a vertical displacement of 1000 metres since the Neogene age. Some observations, in a new road-cut approximately 1km downstream of the Assomata site, show continuous movement of the region, since fault lines exhibit movements of older crystalline rock rising up through recent (lowest river terraces) deposits (see Plates 3.1 and 3.2).

3.2.5 Seismicity of north-western Greece

Western Greece, and especially the region between 38^{o}-39^{o} latitude, has the highest recorded incidents of seismic events (see Fig.3.14).

According to Drakopoulos (1976, 1980) a study of the isoseismal maps (Fig. 3.15_B) of Greece, based on the isolines of maximum observed intensities over certain periods of earthquake records, shows an elongation of the isolines which mainly trend NNW-SSE and secondarily trend in WSW-ENE directions. This tendency seems to be generally compatible with the geological fault pattern of the area. Galanopoulos (1965) has suggested that the intersection of the Ionian fault zone with the assumed prolongation of the North-Anatolian shear fault, which traverses central Greece, is responsible (see Fig. 3.6_A and 3.7).

Recent studies by Kronberg et al. (1978) on the crustal fracture pattern of the region of Greece, using LANDSAT imagery and bathymetric maps of the surrounding seas, generated the same pattern of fault trends as that given above. Orientations of fault planes calculated by Ritsema (1974) and based on the position of earthquake epicentres are also compatible with the same fault pattern.

For the specific area of central-western Greece the following

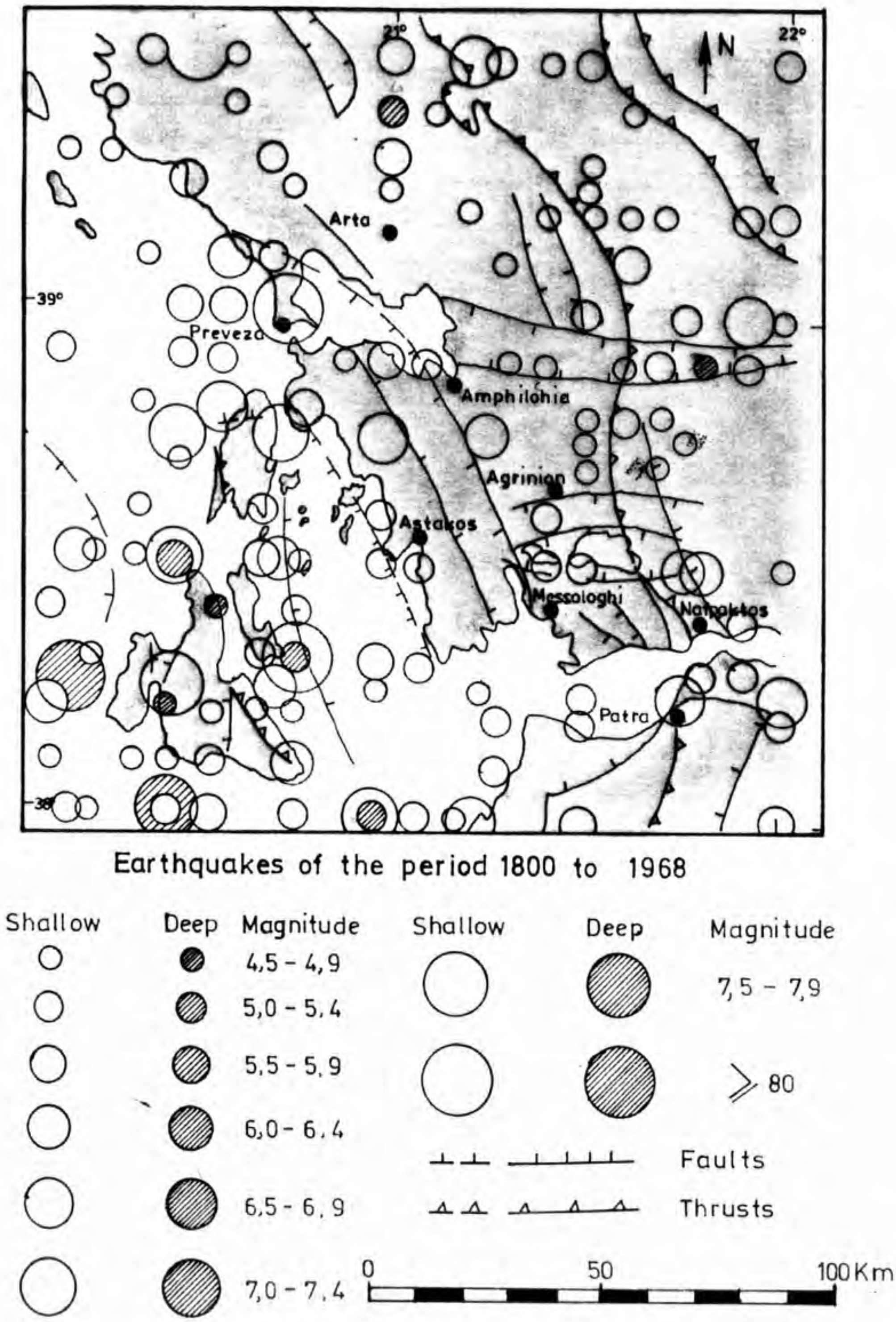


Fig. 3.14 Seismotectonic map of North-Western Greece (after Liakouris, 1971).

factors (after Drakopoulos, 1974 and 1976) contribute to an explanation of the observed intense seismicity:

a) The area lies to the east of the Hellenic trough . This trough is considered to be a newly-developing geosyncline, and the adjacent area is traced by major faults (see Fig. 3.6_A and Fig. 3.7).

b) A recorded 1 to 2 mm per year (or thereabouts) westward movement of the Peloponese along the main Corinthian Gulf-fault, suggests that similar movements are taking place in other parts of Greece.

c) Crustal thickness below the Pindos mountains is about 48 km and is abruptly attenuated towards the Ionian sea to a thickness of 23 km (after Makris, 1973).The distribution of earthquakes with respect to such irregularities in the Moho surface suggest some correspondence between seismicity and deformations of the Mohorovičic boundary.

The predominance of shallow earthquakes in the Ionian seismic zone, and the intense seismic activity released, is associated with a pronounced low of negative Bouguer anomalies; the high sediment supply rate of the rivers into the Ionian sea suggests that a plate accretion must occur in the area (Galanopoulos, 1973).

According to studies by Galanopoulos (1968) the following seismic events and return periods may be predicted for shallow earthquakes (Fig. 3.14) in the Acheloos River areas:

a) Lower Acheloos southern region:

i) One earthquake of $M \geqslant 6$ every ± 12 year period.

ii) One earthquake of $M \geqslant 6\frac{1}{2}$ every ± 45 year period.

b) In the upper Acheloos area (northern region):

i) One earthquake of $M \geqslant 6$ every $\pm$ 22 year period, and ii) One earthquake of $M \geqslant 6\frac{1}{2}$ every $\pm$ 60 years.

It is believed that three seismic events experienced in the area during 1966-1967 were induced earthquakes, possibly triggered by the Kremasta dam (reservoir) construction and specifically the filling of its reservoir (Snow, 1972; Drakopoulos, 1974; among others).

The particular events are:

a) the Kremasta earthquake (5 Feb. 1966);

b) the Katouna earthquake (29 Oct. 1966); and

c) the Drosopighi earthquake (1 May 1967).

Their magnitudes were $M \approx 6.2-6.4$ and the epicentres are shown in Fig. 3.15_A.

Another event, which occurred on Oct. 13, 1969 after the Kastraki dam was filled, is believed to have been an induced earthquake of $M \approx 5.5$ (Snow, 1972). Its epicentre is located in the thrust of the Inachos river valley (see Fig. 3.15_A and 3.11).

Figure 3.15_B comprises the composite isoseismal map of earthquake activity recorded in the Epirus area during the period 1700-1980.

Figure 3.15_C, showing maximum observed intensities in Greece, illustrates that minimal earthquake activity is to be expected in the Lower and Upper Aliakmon River damsites (Veria Town area).

According to Papazachos (1979), although the seismicity of Greece is very high, the seismic risk is lower because:

a) most of the foci of the near-surface earthquakes are deep (deeper than 20 km),

b) the foci of the very high magnitude earthquakes are located in the sea areas around Greece (Greek arc),

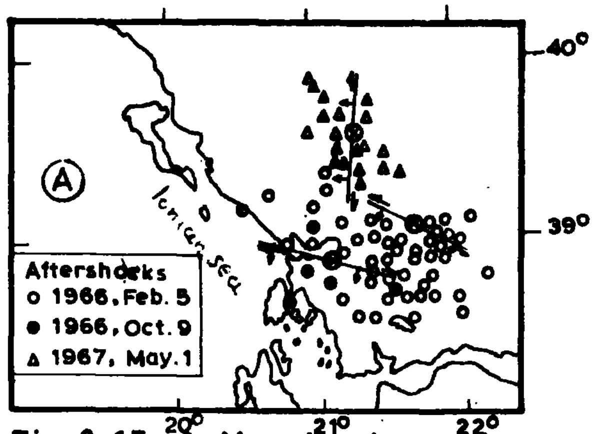


Fig: 3.15 A. Map showing main shocks ⊗ and distribution of aftershocks strong enough to be located by the International Seismological Center–Edinburgh (after Snow 1972)

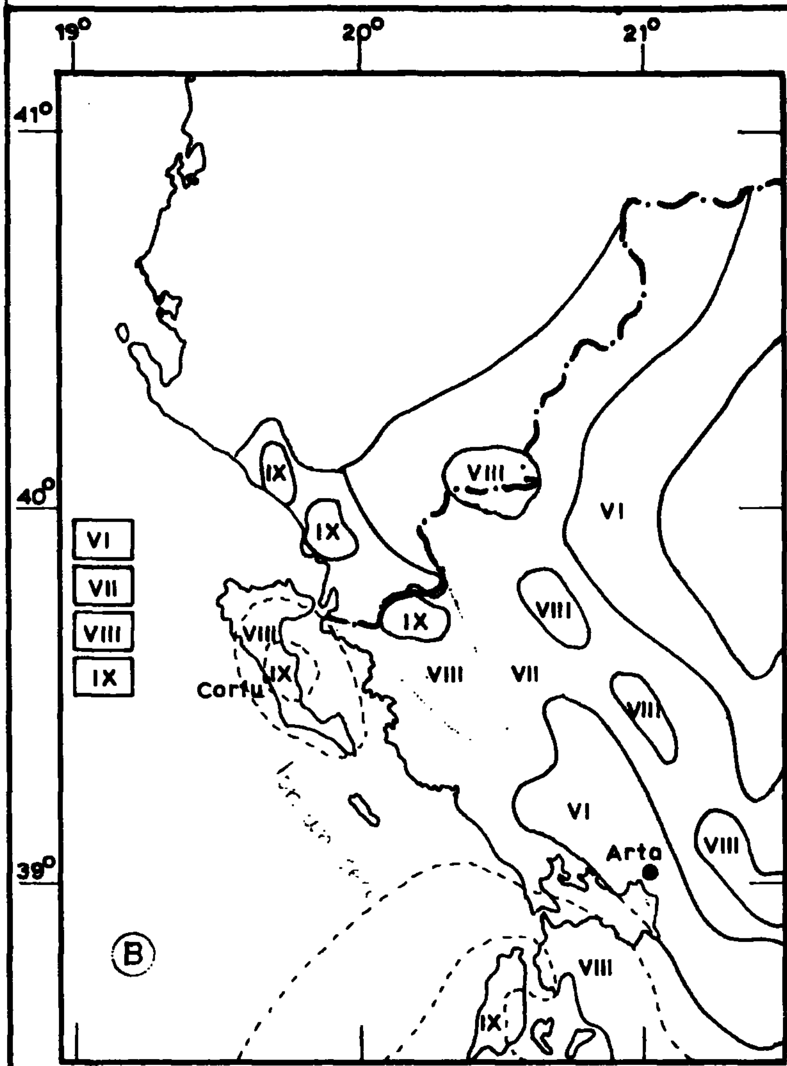
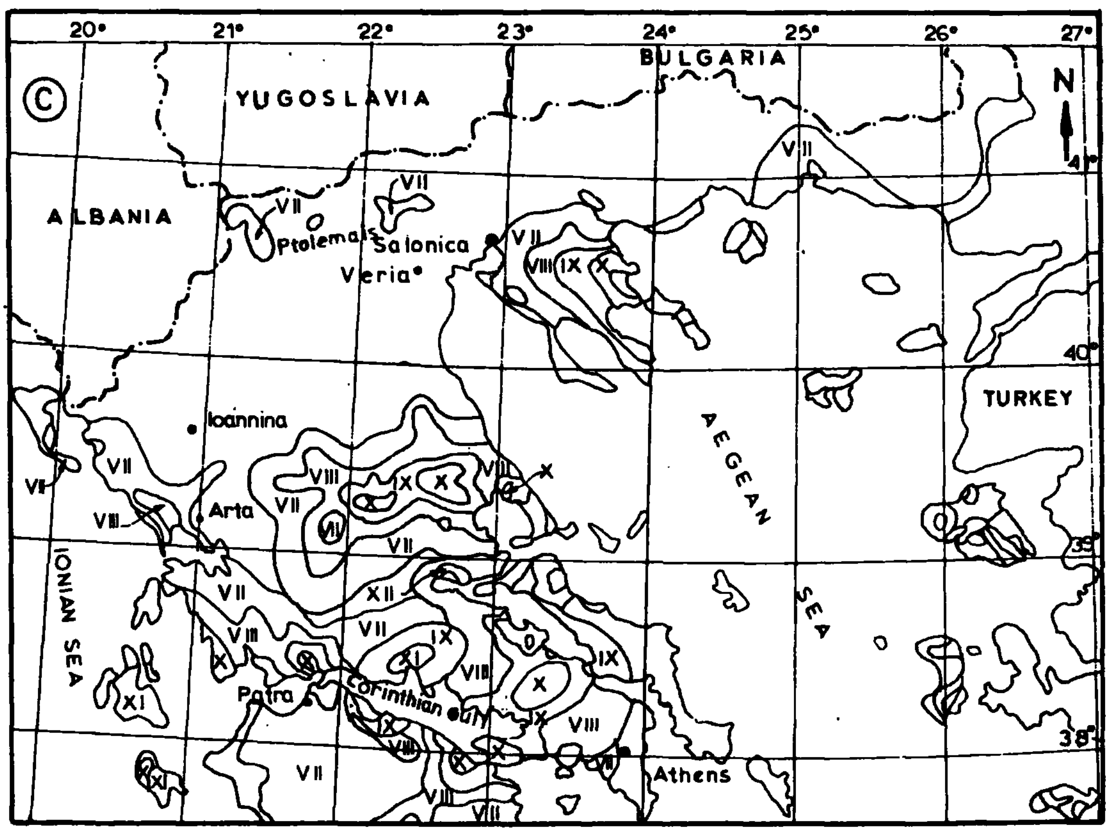


Fig. 3.15 B. Generalized map of maximum observed intensities in Epirus; Period: 1700 – 1980 (after Drakopoulos, 1980)



VII VIII IX X XI XII Intensities on Modified Mercalli Scale.

Fig. 3.15 C. Maximum intensities felt in Greece in historical times (compiled by N. Delibasis and A. Galanopoulos, 1965)

Note: See these Figs with Figs: 3.7, 3.13, 3.1.

and c) there is an attenuation of seismic waves towards the centre of the arc (see also Fig. 3.15).

3.3 Conclusions and remarks

Greece, the southern-most part of the Balkan peninsula, was created by the events of the Alpine and post-Alpine orogeny during the Mesozoic and Cenozoic times.

The geotectonics and geodynamics of the broader region of the eastern Mediterranean suggest that orogenic movements are continuing today, and it is assumed that a new orogenic belt is just developing outside the Hellenic (Greek) arc.

The complexity of geology and the high seismicity records have given rise to three main hypotheses for explaining the Country's creation and structural evolution: the geosyncline interpretation, the plate tectonics interpretation, and the thermodiapiric processes interpretation.

Greece is considered to be the most actively-seismic region in Europe.

The seismic energy released there annually is estimated at two per cent of the seismic energy released annually throughout the world. Relatively aseismic areas in Greece proved to be the Middle and Lower Aliakmon River areas together with the Kozani-Ptolemais-Veria area of Western Macedonia, and these areas are considered geodynamically in tension.

The geological and seismotectonic studies do define two major axes. The first axis strikes NNW-SSE. The second axis strikes WSW-ENE. Along these two axes similarities and differences exist which might be useful for engineering geological zoning. Along the first axis geomorphological, lithological, and structural features are approximately homogeneous in a regional context. But along the second axis the above features exhibit great differences within short distances.

Along this second WSW-ENE axis, which traverses the isopic zones, certain breaks (fault lines) occur. Plate tectonic studies consider that such fault lines are a prolongation of the north-Anatolian fault and seismic activity is observed such as in the Corinthian Gulf and the Amvrakia-Lamia-Volos-north Aegean sea line.

The Greek rivers flow along such axes which constitute major breaks or similarly-formed lithological units (soft or hard strata).

In a similar way, ridges, lakes, thermal springs, chains of karstic limestone series, and gypsum diapiric phenomena are mainly aligned along the first axis.

No matter what the river is or it's flow direction, the axes of the Greek dams are aligned in an approximate NW-SE direction.

CHAPTER 4

Field work

4.1 Introduction

In Chapter 3, the geological, geotectonic and seismicity environment in which Greek dams are mainly located has been briefly reviewed in order to"paint the backcloth"for a consideration of dam foundations in North-Western Greece.

This Chapter presents appropriate geological and geotechnical records which characterize the foundation bedrock particularly with respect to the watertightness (or permeability) of three dam locations, namely:

1) Pournari dam on the Arachthos River.
2) Assomata dam on the Aliakmon River.
3) Sfikia dam on the Aliakmon River.

The main information considered in this fieldwork comprises: a) the recorded permeabilities, from the investigation boreholes, in respect of lithology, structural weaknesses, weathering and depth, and, b) the geological information recorded during the excavation of the core trench, and the grouting and drainage galleries in the grout curtain alignment.

In addition to the above information the drilling methods used, the grout properties, the allowable pressures applied and the completion criteria considered will be discussed briefly. The grouting results will then be presented in terms of the grout curtain geometry and extension, the grouting boreholes drilled and the quantities of grout used, as well as their interrelationships as discussed in the following paragraphs.

Plate 4.1 General view of Pournari dam constuction works.

These grout quantities will provide information which, together with the design-stage information, will facilitate an assessment of the voidness of the foundation bedrock and reveal important geological characteristics.

4.2 Pournari Dam

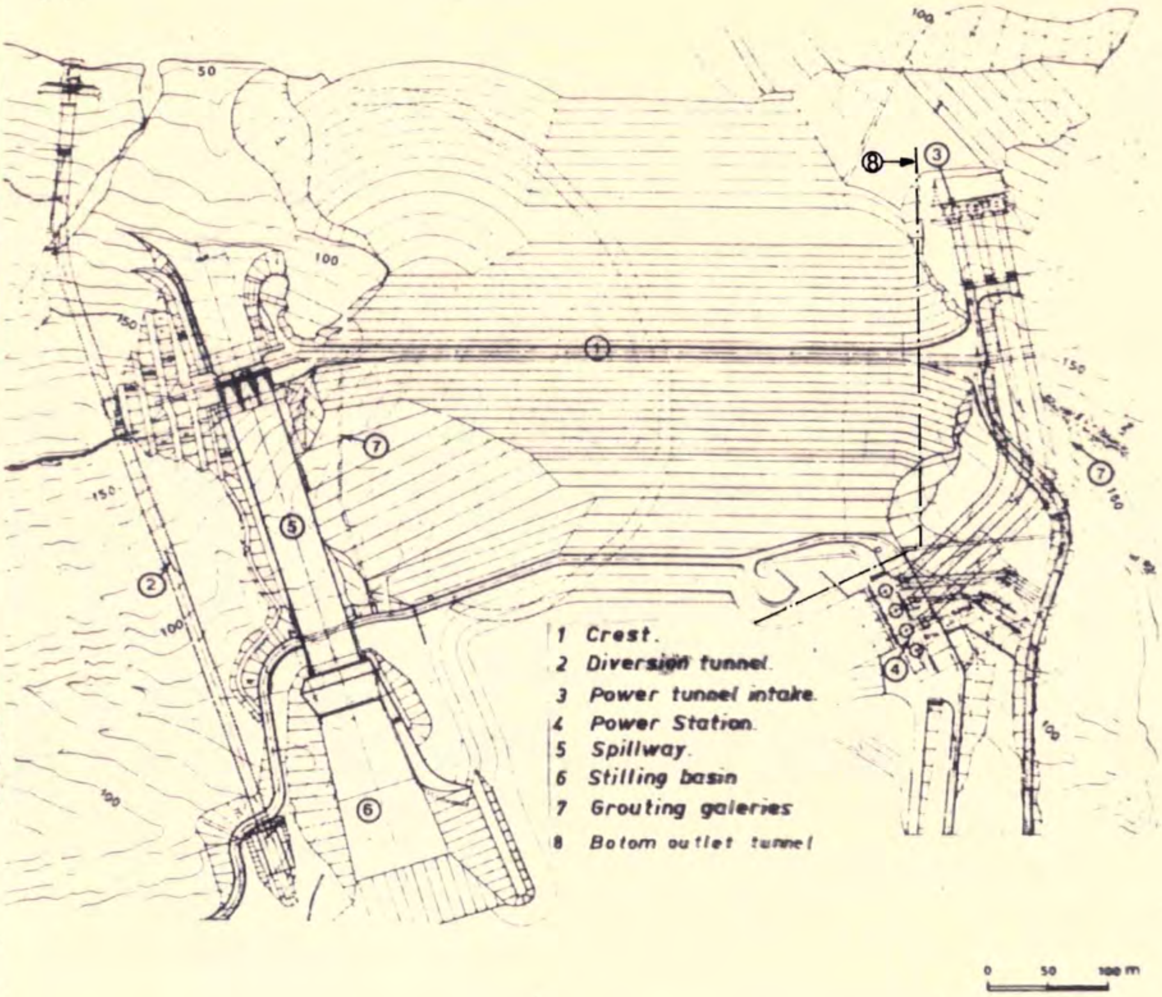
4.2.1 Description of the project

The Pournari dam, completed in 1981, is an earthfill zoned dam with a central clay core and sand-gravel shells, transition filter zones and large stone-block (rip-rap) protection on its upstream face (see Figs.4.1$_{A}$ and $_{B}$ and Plate 4.1).

The core trench, excavated to firm rock, revealed the deepest point of the riverbed at an elevation of 24.07m and the dam superstructure has a maximum constructed height of 103m. Its crest is at an elevation of 127m and its crest length is 574m. The dam volume contains 9.5 x $10^3 m^3$ of construction materials, and its reservoir will retain a total water volume 730 x $10^6 m^3$ with a useful capacity of 340 x $10^6 m^3$. The impounded lake will extend approximately 25 km upstream of the dam.

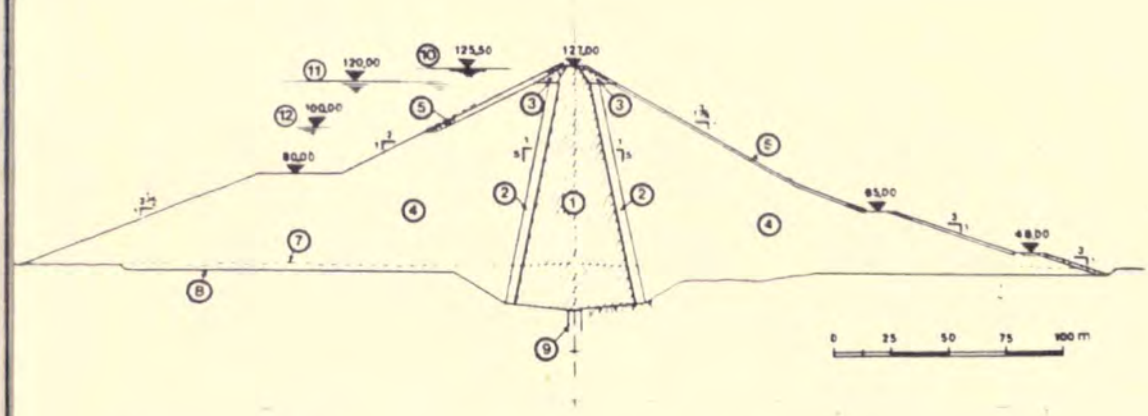
The hydroelectric station, located in the left abutment behind the downstream toe of the dam, includes three units each of 100 MW. The three generating units are independent and are fed from three power tunnels of 366m to 419m length and 7m nominal diameter with concrete and steel reinforcement. Located in the right abutment are the concrete-lined diversion tunnel (720m long and 10.5m diameter) and a chute-type spillway, regulated by three radial gates 12.5m wide and 12.5 high, providing passage for an ultimate flow of 6 100 m^3/sec.

(A) PLANE



1 Crest.
2 Diversion tunnel.
3 Power tunnel intake.
4 Power Station.
5 Spillway.
6 Stilling basin.
7 Grouting galeries
8 Botom outlet tunnel

(B) CROSS-SECTION



1 Impervious core.
2 Transition zone.
3 Transition zone at crest.
4 River sand and gravel shells.
5 Rockfill slope protection.
6 Slope protection layer.
7 Original ground.
8 Dam excavation line.
9 Grout curtain.
10 Maximum flood level.
11 Maximum power pool.
12 Minimum power pool.

Fig. 4·1
Pournari Dam
(Plans and sections)

Prior to the main dam and the upstream cofferdam works, started in 1976, the diversion tunnel and a concrete diaphragm wall of 350m length and a maximum depth of 15m were constructed to allow "dry" conditions for the core-trench excavations. During the main upstream cofferdam works another tunnel (named the "bottom outlet tunnel") was added in the left abutment. It is 350m long and 7m in diameter with concrete reinforcement and was designed to meet emergency conditions.

4.2.2. Foundation bedrock conditions: an assessment prior to grouting

The bedrock conditions and the expected corrective measures for strengthening and sealing of the foundation bedrock have been investigated in some detail. The exploration programme comprised boreholes, adits, trenches, geophysical surveying, in-situ permeability and rock mechanics testing, detailed logging, detailed mapping and photo-interpretation of surface geology, as well as a comprehensive sampling and laboratory testing programme to evaluate local materials for construction.

The outcome of the above studies, together with other (such as economic and environmental) considerations, have led to the construction of the dam in its present layout. This present study will use selective features of the bulk of information collected as above to delineate foundation bedrock conditions as follows:

A. Geomorphological and lithological characteristics of the damsite

The Pournari H.E. project and its reservoir lies on the Oligocene flysch formations of the western limb of the Epirus-Akarnanian syncline of the Ionian zone in the lower Arachthos river, about

4 km east of Arta town (see Chapter 2.3 and Figs.1.1 and 2.11).

The river flow has a general N-S direction, but at the damsite is twisted westwards through a small erosion valley about 300m wide. From upstream to downstream of the dam axis, and from the top of the valley to the bottom, the river has eroded the following main lithological units which are considered in the present study (Figs.4.1$_D$ and $_E$):

i) The moderately-to-thickly-bedded sandstones (SSL unit) belonging to the Peta sandstone series of the area. These sandstones are capping and protecting from erosion the other softer lithological units, which are:

ii) The silty conglomerate (SGL unit, unstratified), which is underlain by

iii) The thinly-bedded siltstone-sandstone sequence (CSL unit).

Due to the cyclothemic character and turbidic nature of the sedimentation processes of the above formations, differentiations within the above units do exist. These differentiations consist of elements of one unit deposited in another in the form of interbedded strata or wide intercalated lenses. The river in its course seems to have eroded the soft member units of the flysch trending N⟶S and occasionally E⟶W. The steepness of the gullies surrounding the site, their relatively steep unweathered (washed-out) fresh slope cuts, the washed-out products graded up to sandstone boulders, and the stormy nature of the rains as well as the torrential character of the river suggested that strong erosion of the surrounding hill currently takes place.

Although the morphology of the foothills in certain areas appears as terrace-like landforms, the stone blocks at the toe and uphill scarps indicate the likelihood of previous landslides. Several minor landslides have been identified in the vicinity of the dam, the most important being the one located in the right abutment at the upstream-toe of the dam which has imposed the rounded shape of the upstream shell, which in fact stabilises this particular area (see Fig. 4.1_A).

B. Bedding and Folds

At the site, beds in general strike N30°W and dip NE at about 25°-45°. The overall development of the strata,kilometres north and south of the site, confirm that no major fault breaks exist nearby. Small variations of the strike and dip of the strata have been noticed within the site, due to the existence of small folds superimposed on the major regional ones (such as that of the Epirus-Akarnania syncline which trends N→S). The most important of these secondary folds was revealed (Fig. 4.1) during the core trench excavations in the river bed, its axis direction being NE - SW, plunging 30°-40° NE towards upstream.

Bedding parting surfaces appear tight, and when separated are infilled with shaly films of sheared clay and sheared siltstone. (Strike slip shearing; see Table 4.1).

C. Joints

Joints are well-developed in systems, either orthogonal or oblique to bedding strikes or dip directions, particularly in the sandstone strata. Major joints exhibit an extension of a few dozen metres and at a frequency of 4-10 metres (Fig. 4.1_D).

The silty conglomerate and the thinly-bedded siltstone-sandstone

TABLE 4.1

Direct shear test laboratory results on selected "clay seam" samples

(Pournari dam)

Sample Geotechnical description	Cohesion intercept c (kg/cm²) (KN/m²)	Peak values of φ'p (degrees)	Residual values of φ'r (degrees)	Sampling code, geological environment and visual description of "clay seam"
Samples of high clay content and high plasticity: range	0.4 (39.23)	20°	16°	*Σ V/15Δ :Joint infilling, 12 cm thick of yellow clay with sand traces, into thickly-bedded SSL unit
	0.2 - 0.3 (19.61-29.42)	18°	14°	Σ III/62Γ: Interbedded material of grey-green clay, extracted from the transition area (contact) of thickly-bedded sandstones (SSL) and the thinly bedded siltstone (CSL)
Samples of medium clay content and plasticity: range	0.6 (58.84)	26.5°	24°	G^4 / 3^4_A : Interbedded clay, sandy, grey-brown, containing fragments of weathered claystone (CSL unit)
	0.2 - 0.6 (19.61-58.84)	24°	12°	G^4/69Γ-: Joint infilling of yellow 5 cm thick clay with sand traces, within thickly-bedded siltstones in the CSL unit. The joint belongs to a set of sheared zones characteristic at this location, with signs of penetrative weathering.
Samples of low clay content and low plasticity: range	0.4 - 0.6 (39.23-58.84)	36°	35°	GV/25A: Interbedded, 4cm thick layer of grey-green clay, in the contact of thickly-bedded sandstones (SSL) and thin layers of claystone (CSL subunit)
	0.5 (49.03)	31°	29°	Σ I/62.6Δ: Joint infilling of yellow brown sandy clay 8 cm thick in a highly-weathered silty conglomerate (SGL unit). This SGL subunit is interfiggered within the SSL unit and it is responsible for the right bank upstream slide.

* Σ V/15_A code number of sampling locations (Σ V = Name of Adit)
/15=Station in metres from portal
Α.Γ.Δ.=The particular sample from the above stationing.

series exhibit small joints of an extension and frequency which range from a few centimetres to a few metres. In the rare instances of the existence of small sheared joints, the rock appears to be shattered. The scattering of these joints in strike and dip (Price, 1959) is due to the existence of the above-mentioned superimposed folds (see Figs.4.1 and 4.6 and 4.7).

The concentration of joints at the apex of the riverbed fold has facilitated the E-W turn of the river and has created by erosion the valley in which the dam is founded. The most important of the recorded joints are those encountered in the upper sandstone beds, and this is particularly due in many cases to the absence of infillings.

D. Shears and faults

Minor faulting and shearing was identified during the exploratory works. It appeared as regularly-spaced shear zones in the right abutment downstream of the dam axis, and within some of the exploratory adits. The measured displacements were of a few centimetres. The core trench and other excavations proved that those shears existed in both abutments in the wings of the anticlinal flexure recorded at the river bed, the most important being those downstream of the right abutment aggravated by deep erosion during undercutting of the riverbed. They strike N50^{o}W to E-W dipping 50^{o} - 85^{o} southerly, being in tension and being infilled with weathering products.

E. Weathering characteristics

At Pournari site weathering, classified in four grades (Hobbs, 1975) from fresh to completely-weathered rock (residual soil) during the investigation stage, turned out to be of six grades

POLAR NET
(Upper Hemisphere Projection)

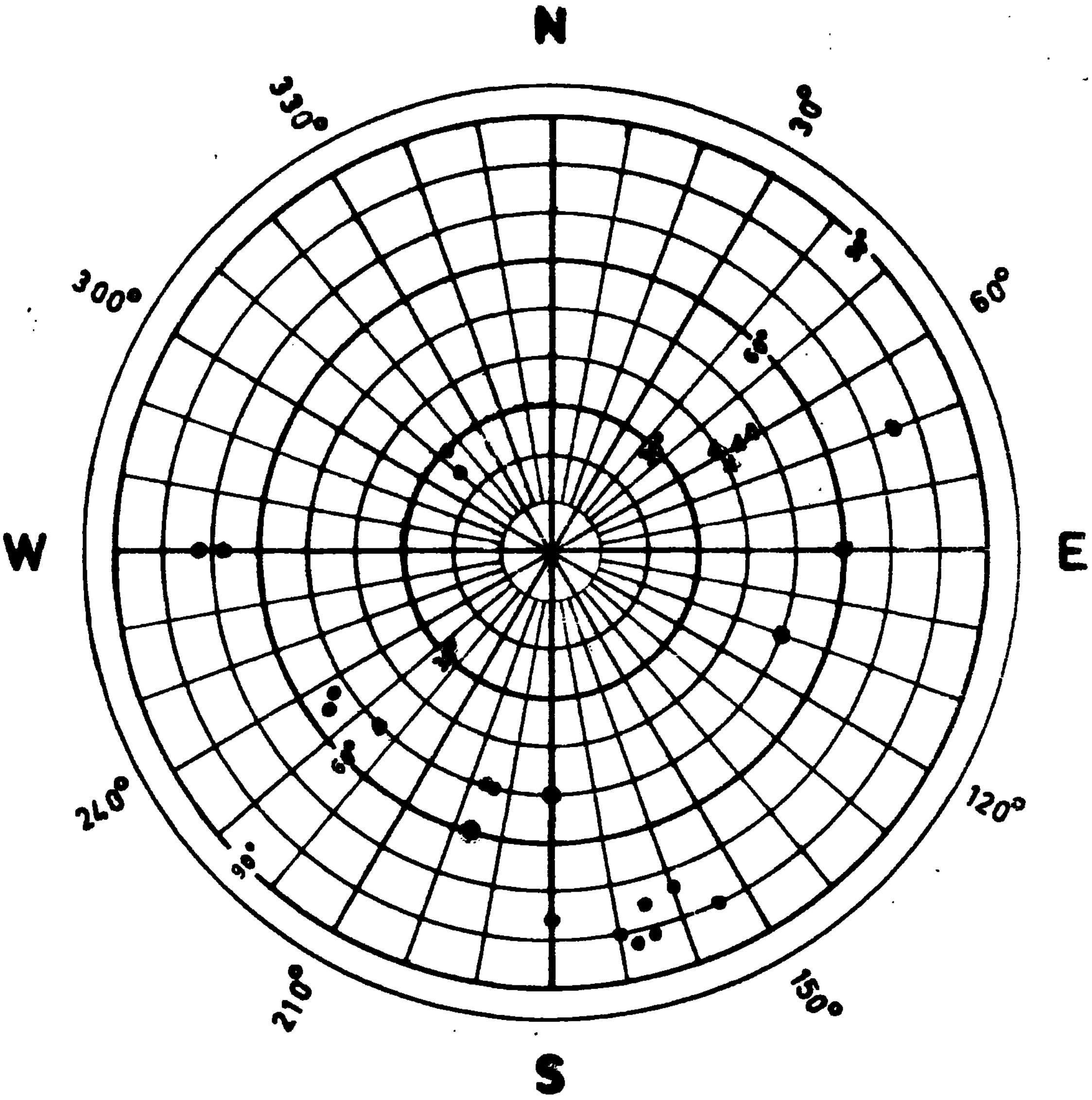


Fig. 4.2 Discontinuity measurements along dam axis (Pournari dam, core trench and abutments sta 0+0,00 to sta 0+200)

LEGEND

- ▲ Bedding planes
- ● Joints
- \+ Major joints (or with noticed displacement)
- ⊕ Shear zones

POLAR NET
(Upper Hemisphere Projection)

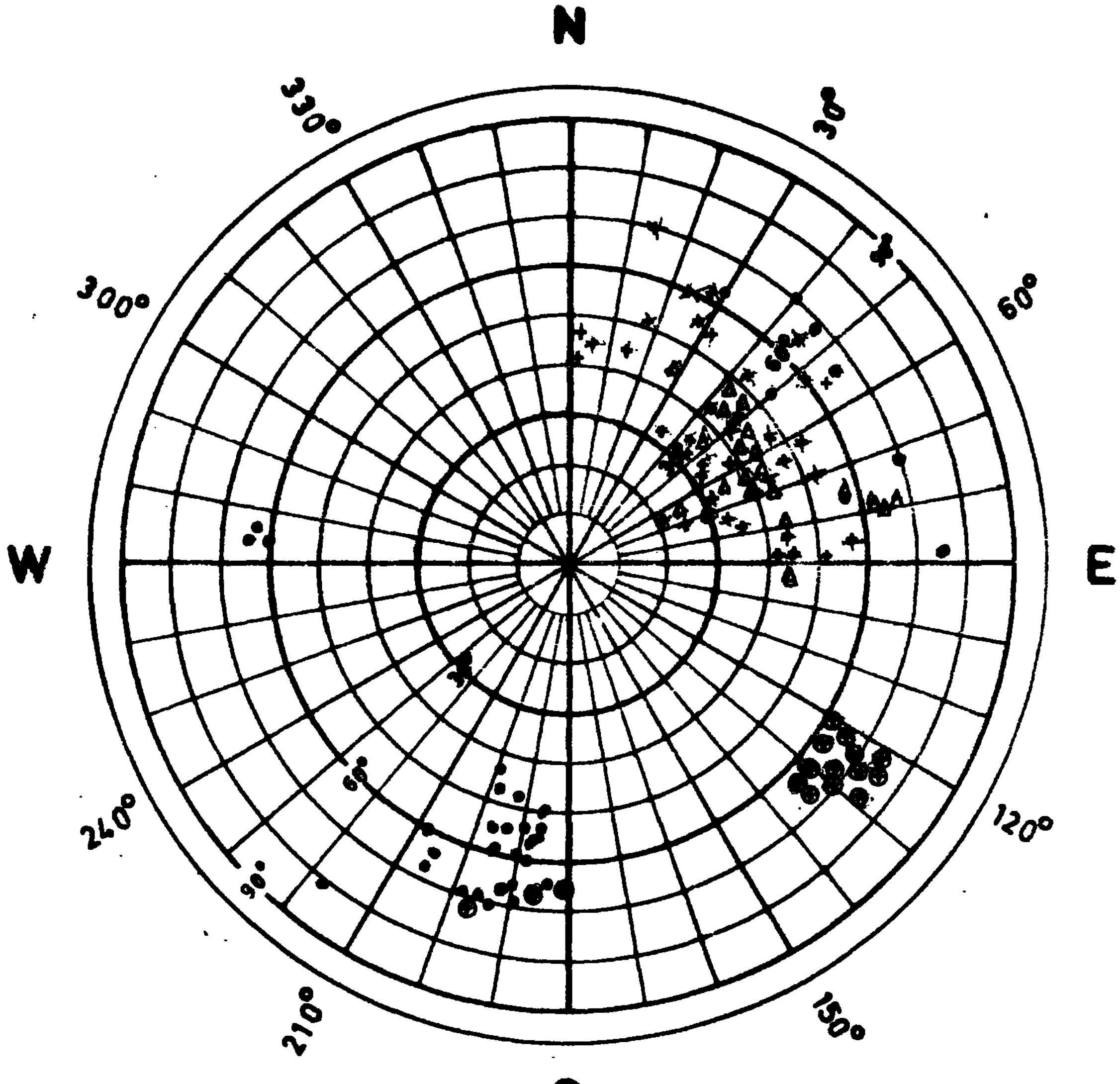


Fig. 4.3 Discontinuity measurements along dam axis (Pournari dam, core trench and abutments, sta 0+200 to sta 0+320).

LEGEND

- ▲ Bedding planes
- ● Joints
- \+ Major joints (or with noticed displacement)
- ⊕ Shear zones

POLAR NET
(Upper Hemisphere Projection)

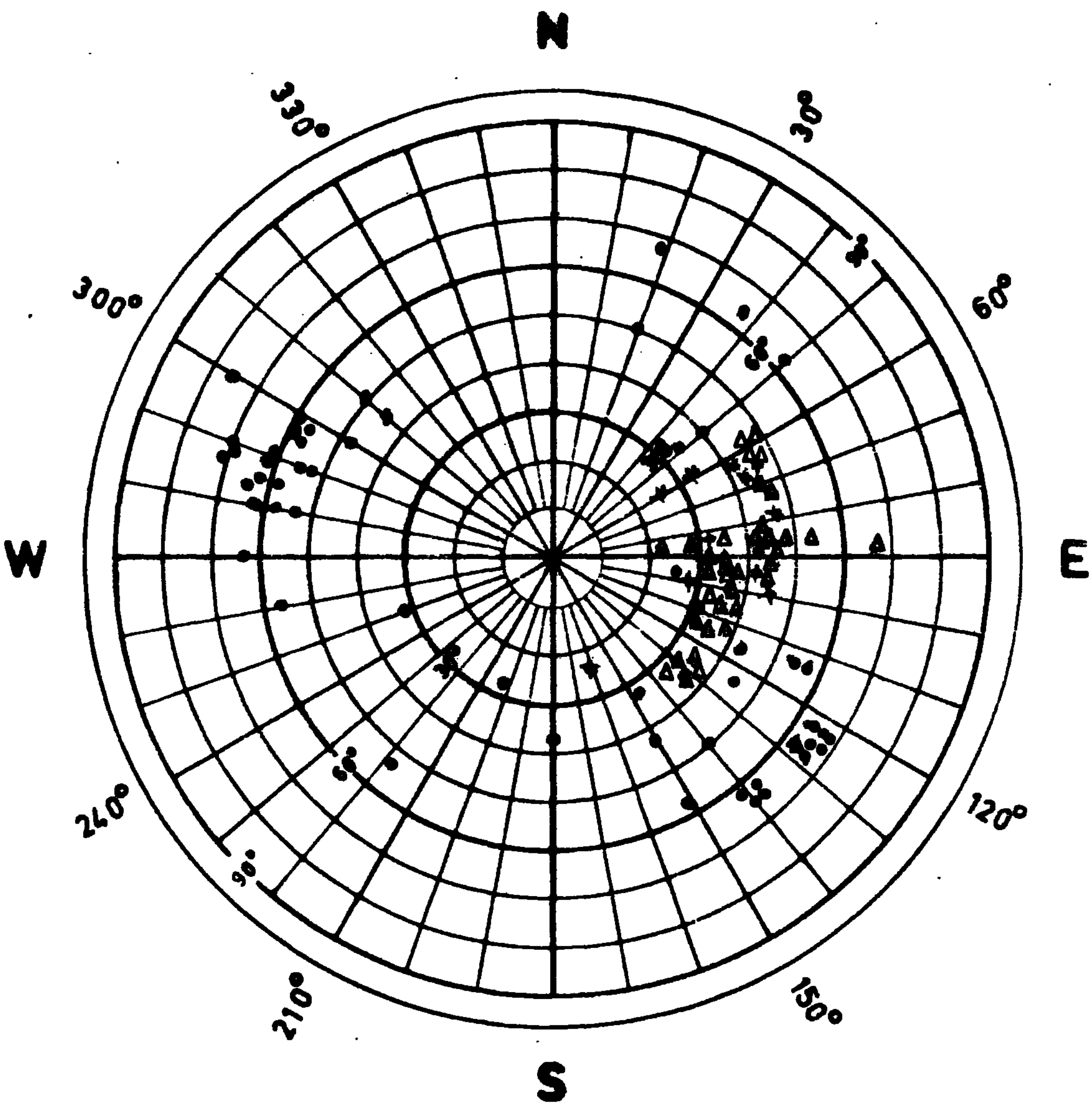


Fig. 4.4 Discontinuity measurements along dam axis (Pournari dam, core trench and abutments, sta 0+320 to sta 0+450; Siltstone - Sandstone sequence).

LEGEND

- Δ Bedding planes
- ● Joints
- + Major joints (or with noticed displacement)
- ⊕ Shear zones

POLAR NET
(Upper Hemisphere Projection)

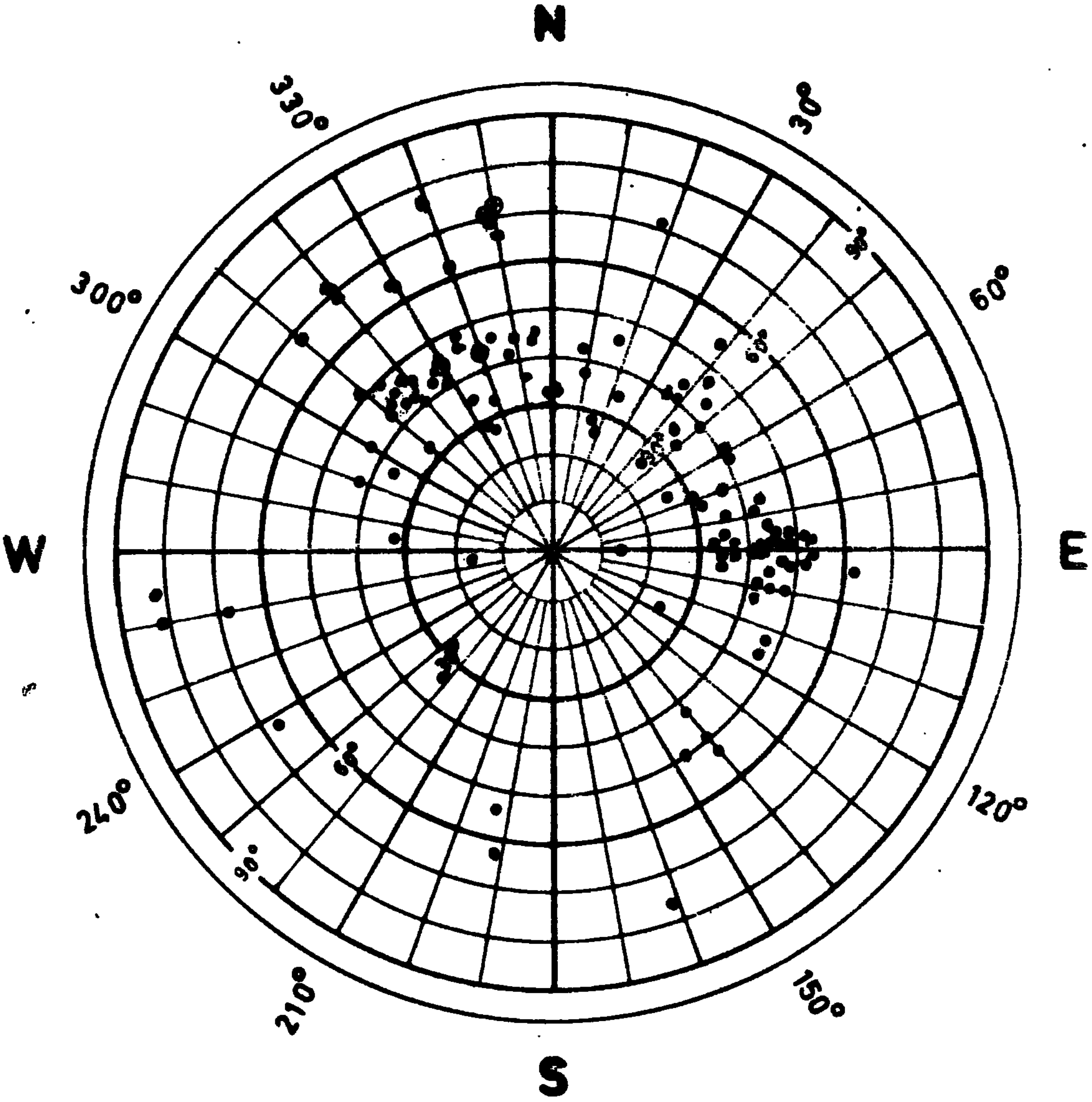


Fig. 4.5 Discontinuity measurements along dam axis (Pournari dam, core trench and abutments, sta 0+320 to sta 0+450; Silty conglomerate).

LEGEND

- △ Bedding planes
- ● Joints
- + Major joints (or with noticed displacement)
- ⊕ Shear zones

POLAR NET

(Upper Hemisphere Projection)

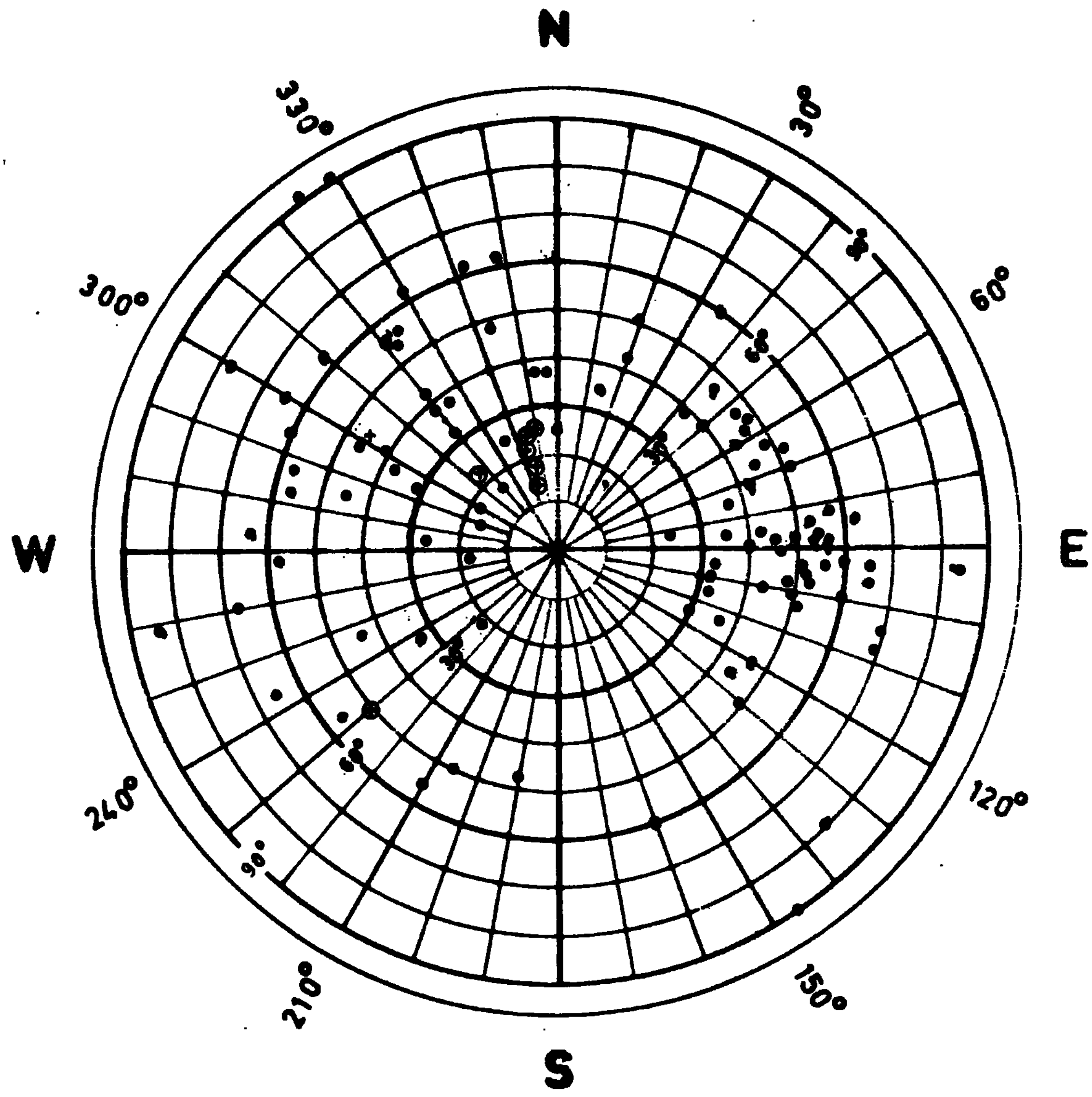


Fig. 4.6 *Discontinuity measurements along dam axis (Pournari dam, core trench and abutments sta 0+450 to sta 0+480).*

LEGEND

- △ Bedding planes
- ● Joints
- \+ Major joints (or with noticed displacement)
- ⊕ Shear zones

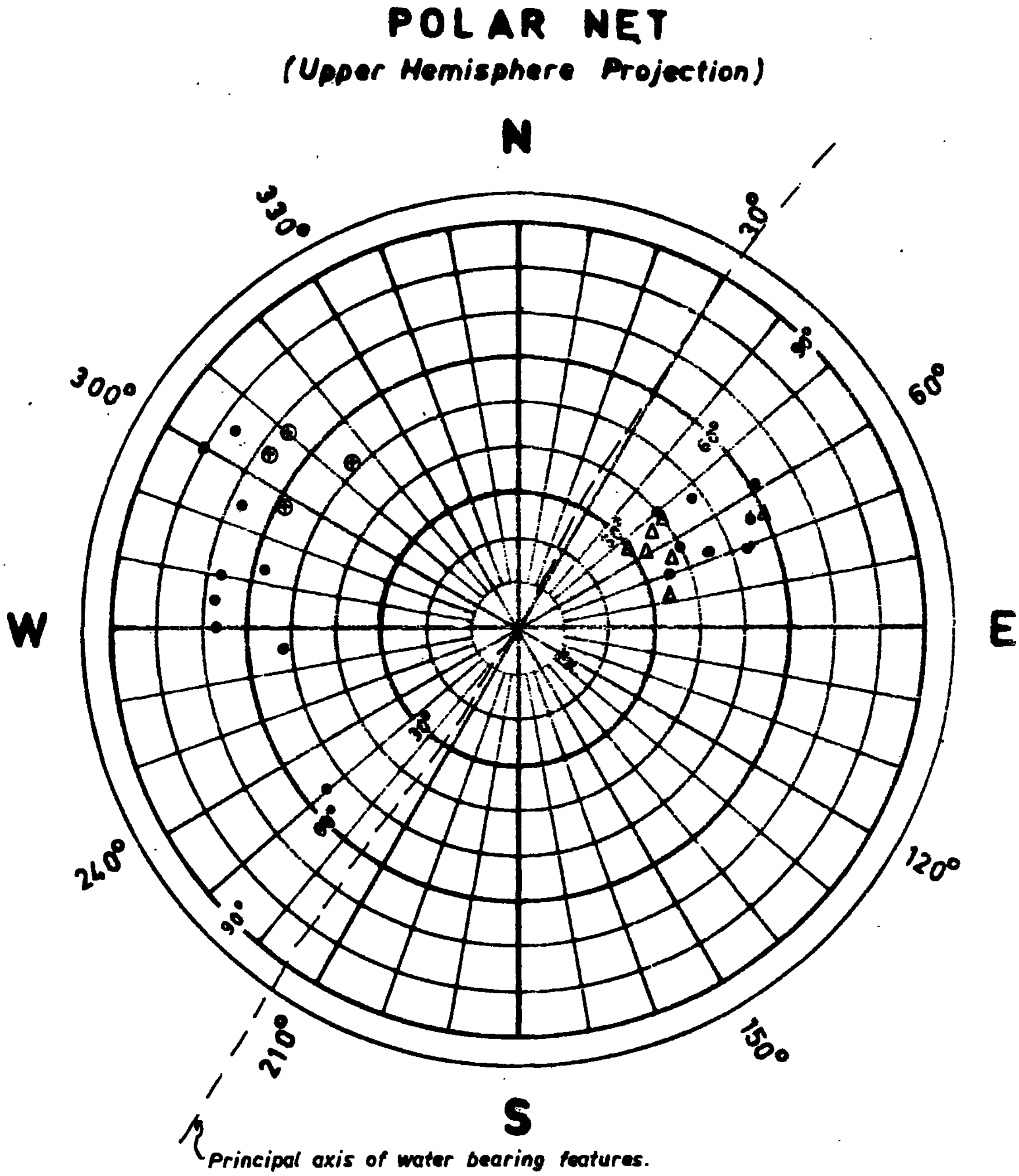


Fig. 4.7 Discontinuity measurements along dam axis (Pournari dam, core trench and abutments sta 0+480 to sta 0+700).

LEGEND

- Δ Bedding planes
- ● Joints
- \+ Major joints (or with noticed displacement)
- ⊕ Shear zones

during construction (Dearman et al., 1972),a good correlation of the two weathering classifications being that quoted by Bell (1978).

Most of the bedrock outcrops at the dam site appeared to be moderately-to highly-weathered, particularly where intense jointing or shearing and sliding had occurred. Samples of clayey infillings, selected along joints or bedding planes, showed that relaxation was deep enough, and weathering was noticed mainly along these discontinuities. In certain areas, particularly in the right abutment, the chemical alteration that had occurred was deep enough and has led to extensive excavation as a result of inferior shear strenth characteristics of these altered rocks. Shear strength characteristics of the infillings are presented in Table 4.1 and some exhibit residual φ_r' values as low as 12^o.

In situ direct shear tests on weathered bedding planes and shears gave higher residual values of $\varphi_r' = 24^o$ to 26^o.

The unconfined compressive strength and the modulus of elasticity of the bedrock were considered high enough for the envisaged static or dynamic loads which might be imposed on the foundations (Table 4.2).

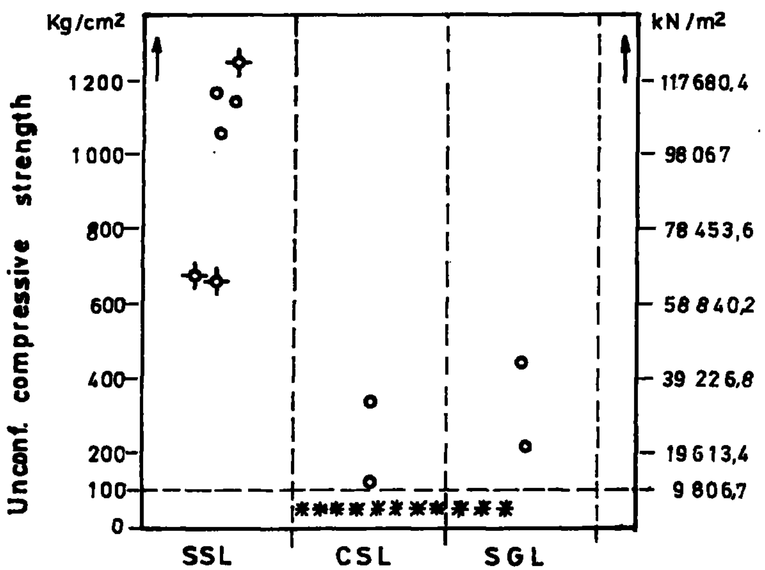
F. Ground water characteristics

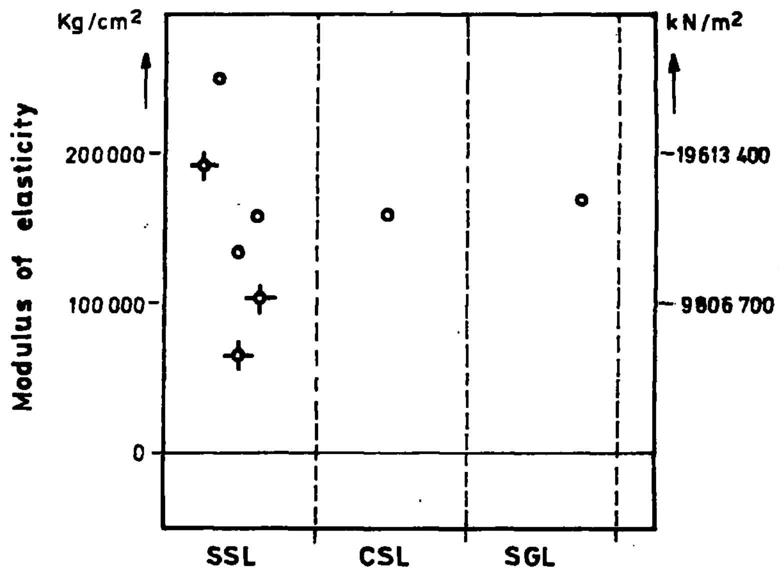
The ground water profile (derived from piezometer readings) proved to follow the topography near the ground surface. It appeared deeper-levelled in the upper sandstone strata. It exhibited an annual fluctuation of several metres, with greater variations noticed in the sandstone strata even during the winter period of rainfall. Water inflows, observed after rainstorms in some adits,

TABLE 4·2

Pournari HEP

Unconfined compressive strength of borehole core samples (Pournari dam)
(after Kotzias Stamatopoulos, 1972)





LEGEND

SSL	Sandstone samples	o	Samples tested as recieved from the jobsite ("dry" samples)
CSL	Siltstone / Sandstone sequence samples	-o-	Samples tested after soaking for 48 hours
SGL	Silty conglomerate samples	*	Samples not tested because they desintegrated during soaking.

indicated that relaxed open joints as well as bedding planes existed. A spring (of a few litres per minute) at the contact between the sandstones and siltstones emerging in the left abutment, as well as minor drippings from shears into the siltstones of the exploratory adits, proved that the siltstones were quite impermeable, although the water levels recorded in the abutments were quite high.

4.2.3 Permeability characteristics

Foundation bedrock permeabilities were investigated with packer pressure tests using clear water and carried out from boreholes drilled along the dam axis in the river bed and the abutments. The results are presented in the Figures 4.8, 4.9, 4.10 and 4.11. The main variable interactions that could be examined were

Permeability versus rock types encountered.

Permeability versus RQD.

Permeability versus weathering.

Permeability versus depth from ground surface.

Each test is symbolized to the main lithological units encountered so as to provide an indication of the interrelations between rock types, weathering and fracture characteristics in terms of RQD and its position in relation to depth. The tests were executed at five metre intervals at allowable pressures not exceeding the overburden pressure, and for very deep holes not exceeding a pressure of 15 kg/cm^2 (1471 kN/m^2).

A first examination of the permeability test data presented in Figs.4.8 to 4.11 indicates that foundation bedrock in the Pournari damsite is quite impermeable. Almost 45 per cent of the tests carried out gave no water takes (zero tests), while another 15 per

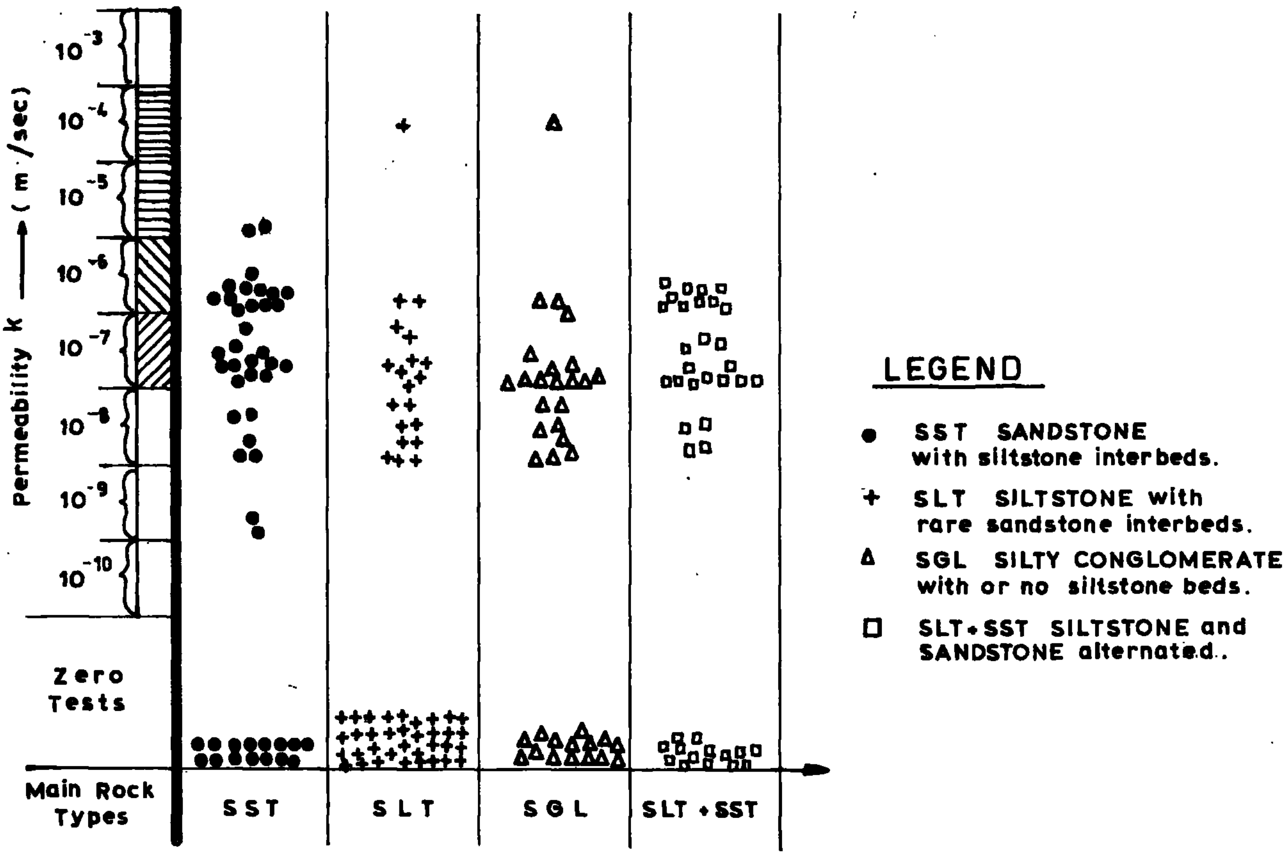


Fig. 4.8 Permeability vs rock types encountered at Pournari dam.

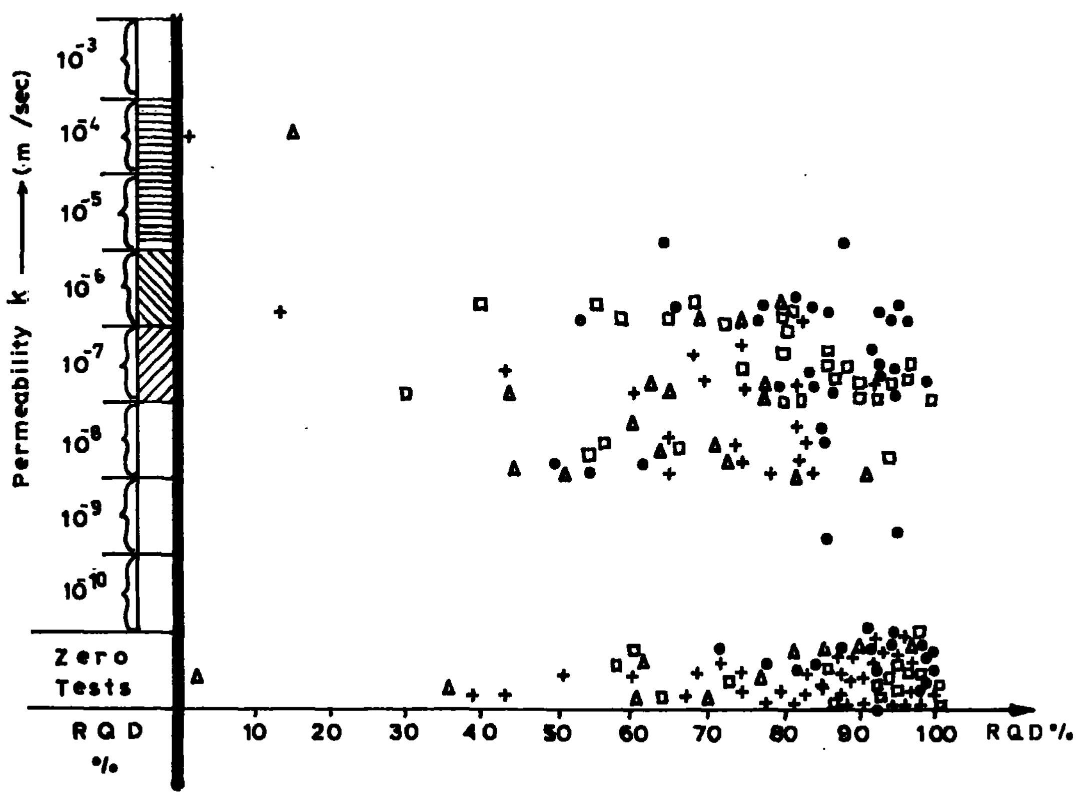


Fig. 4.9 Permeability vs RQD at Pournari dam.

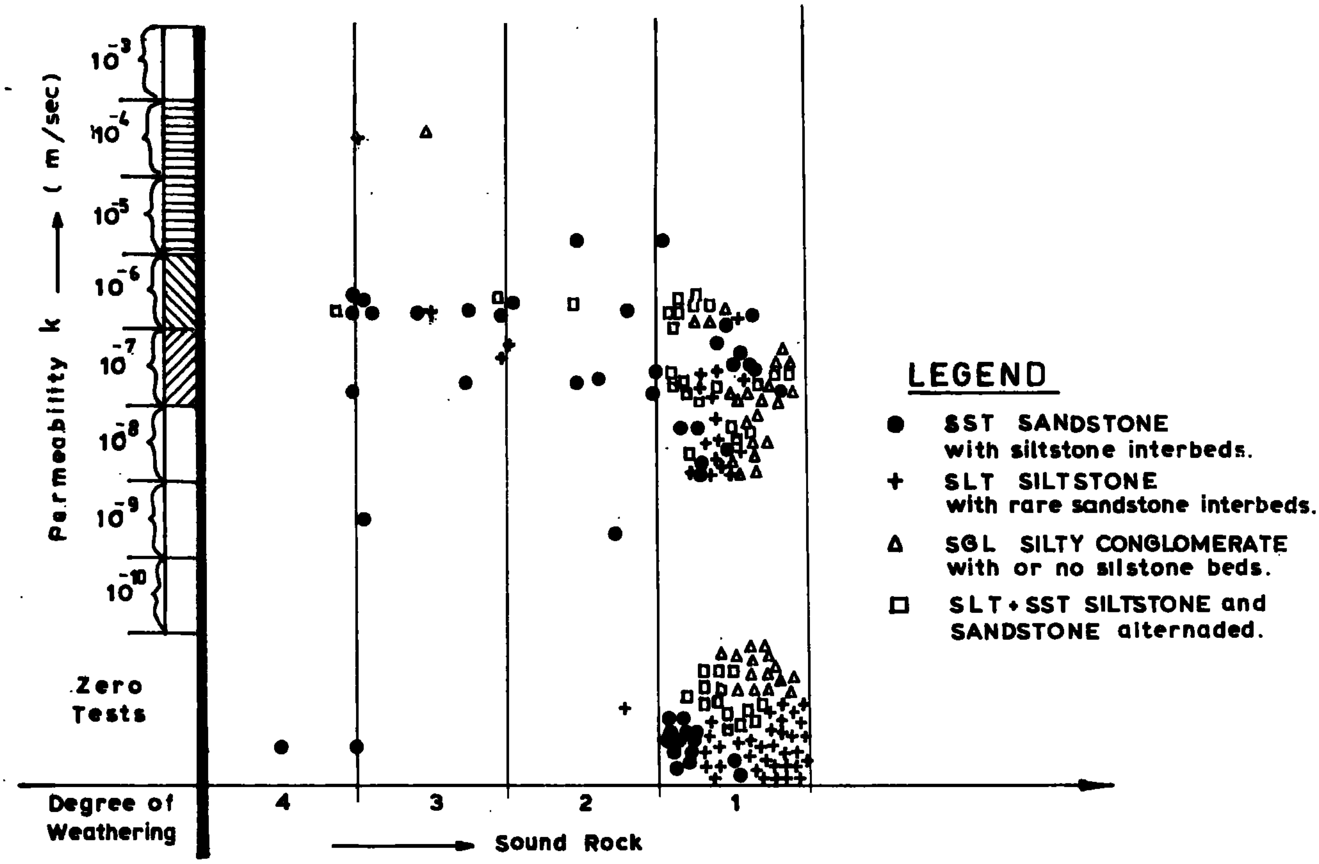


Fig. 4.10 Permeability vs weathering at Pournari dam.

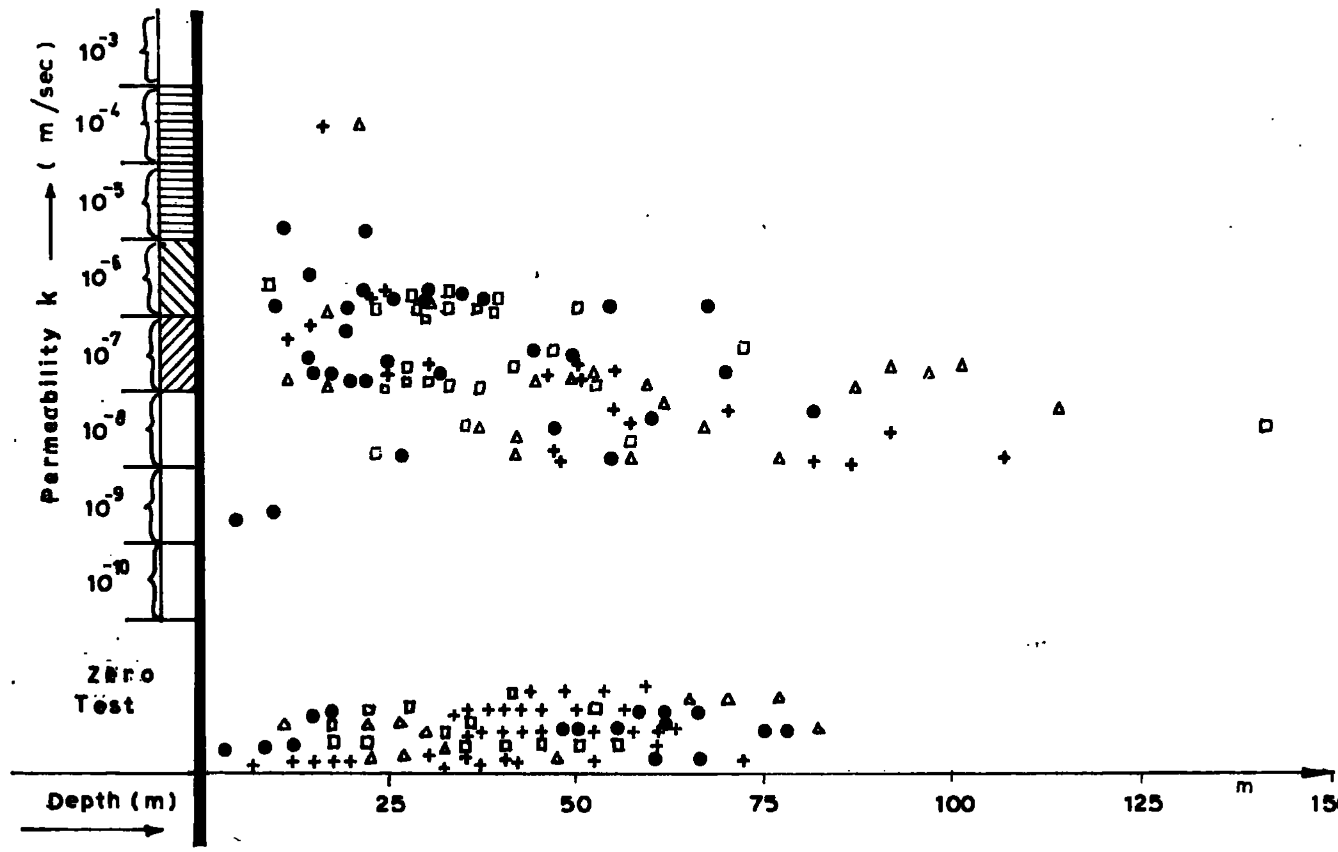


Fig. 4.11 Permeability vs depth at Pournari dam.

cent exhibited permeability values equal to or lower than 1 Lugeon = 1.3×10^{-7}m/s . According to Houlsby (1977,1982), no grouting at all would be required under these permeability conditions.

The main conclusions, which can be drawn from examining the permeabilities recorded in relation to rock type, RQD, weathering and depth are as follows:

Permeabilities are higher in the sandstone (SST) and sandstone bearing (SST + SLT) sequences (Fig. 4.8). This is due to the jointing observed in these sequences and to the existence of water-bearing, bedding/parting surfaces, occasionally occurring there.

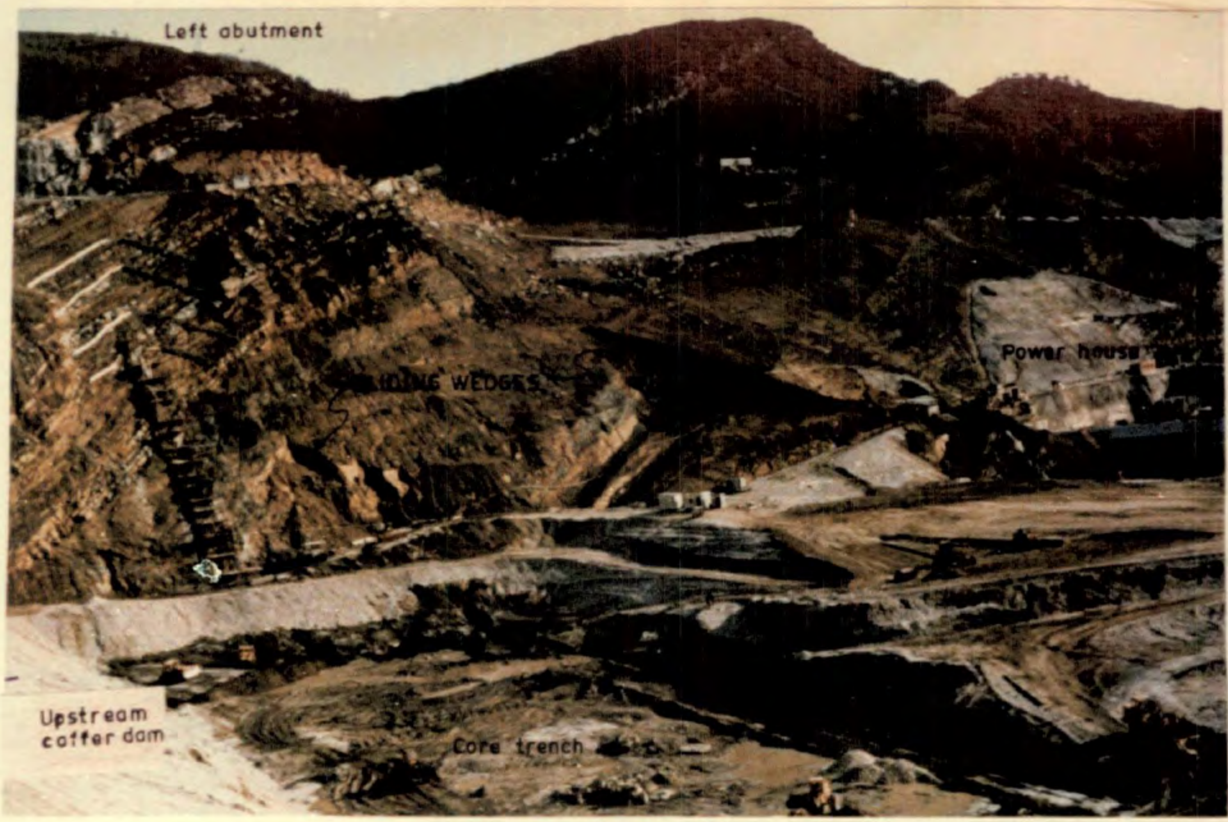
Permeabilities are correspondingly with the lower RQD's observed, although the vast majority of the tested intervals exhibit very high RQD values. Among those rocks showing the higher RQD values are the sandstones which have, from the majority of tests, higher permeability values (Fig. 4.9). The conclusion can be drawn that water is permeating through a few distinct joints or bedding planes. Permeabilities in the most weathered intervals are higher, but about 90 per cent of the intervals were tested within fresh rocks (Fig. 4.10). Most of the intervals which exhibit high weathering belong again to the sandstone-bearing units. These, together with the previous observations, suggest that weathering of joints and relaxation, as well as loose bedding/parting surfaces (which act as water-bearing features), might be expected to be better developed within the sandstone-bearing sequences.

Finally, and as expected, permeability values decrease with depth but, again, the higher values are within the sandstone-bearing sequences. These values proved to be in the range of 10^{-6} to 10^{-7} m/s even at a depth of 30m -70m (Fig. 4.11).

There are several factors emerging from the permeability test results which can explain the transmissibility behaviour of the foundation bedrock.

The sandstone sequence (see Fig. 4.1) is the top-most formation in vertical section along the dam axis. The relaxation observed in these rocks is the highest at the site, and weathering in its joints or bedding planes easily destroys the rock texture by degrading its calcareous cement bonds so that they became easily eroded by circulating water. Its brittleness, too, does allow an open frame - work of joints and bedding planes to be easily created in this sequence. The softer members of the flysch at the site are prone to weathering which destroys their internal cohesion and creates impermeable joint infillings from the break-down products.

Moreover, due to cyclical wetting and drying out, the joint walls exhibit flaking, which infills the joints. Any shearing movement also creates impermeable infillings. The ductile strain behaviour of the softer members, as well as the thin nature of the beds, allows the rock to accommodate the higher stresses.



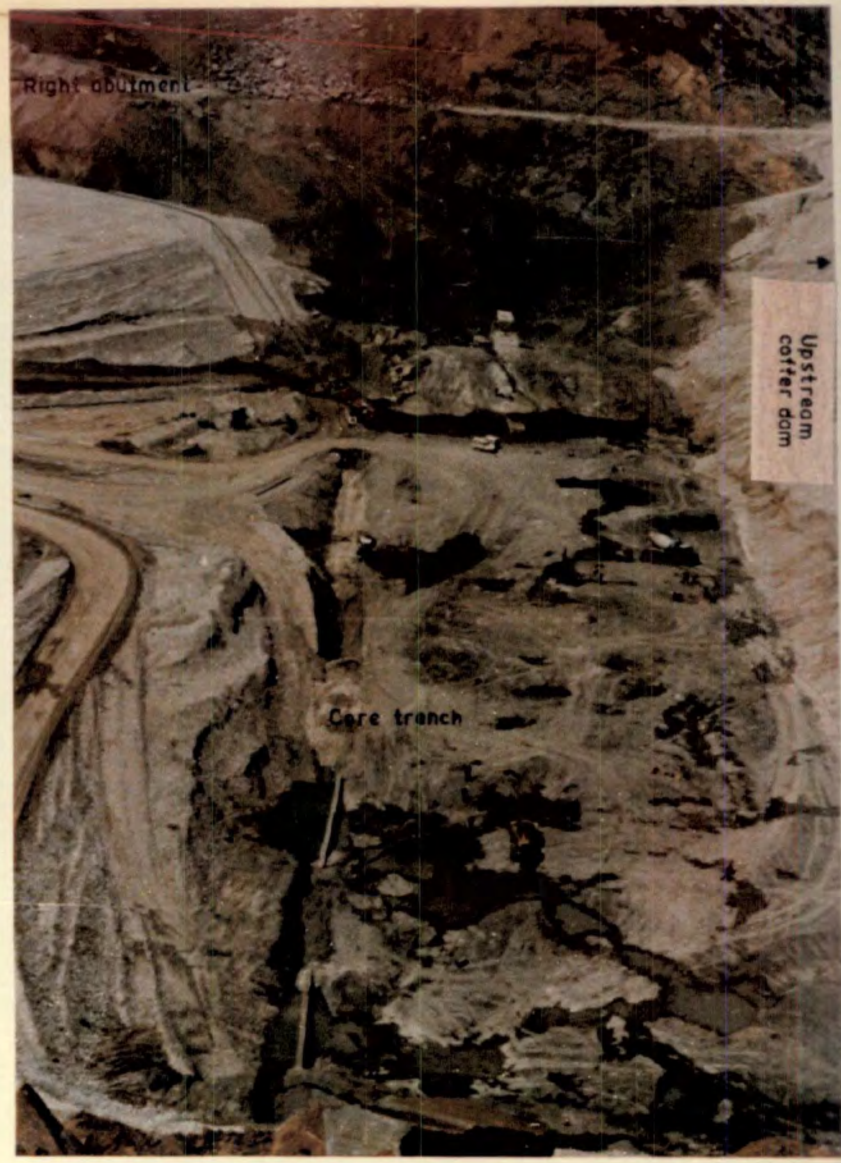


Plate 4.2 Pournari core trench excavation

Plate 4.3 Pournari core trench excavation
(Cleaning and core placement)

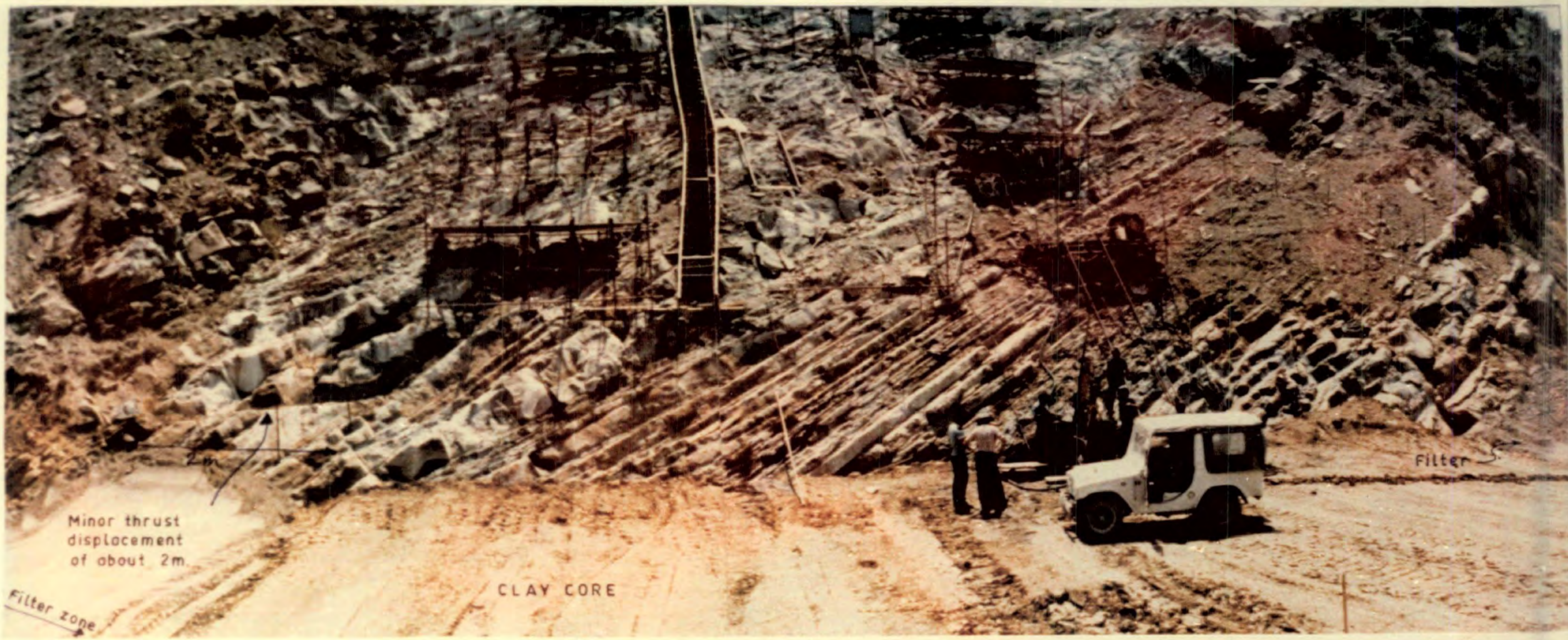


Plate 4.4 Core placement on to the left abutment sandstone (SST) sequence (Pournari dam)

4.2.4 Grouting at Pournari

(a) General

Grouting at Pournari was carried out for two main reasons:

1) to reduce leakages under the dam and from the abutments,
2) to reduce the uplift pressures into the abutments.

In this second case, in addition to the grouting a drainage curtain was constructed from the same galleries which were also used for the construction of the grout curtain.

Grouts, drilling methods and the grouting procedures adopted are the same for Pournari, Assomata and Sfikia. Details are given in Appendix C.

(b) Grouting parameters and grouting results

Grouting results presented here concern the main grout curtain, while mention of grouting results of auxiliary holes, off the main curtain, will be made where necessary to denote important geological factors conditioning the evaluation of the results.

The detailed grouting field results are presented in the "as built drawings" which are given in Volume II. (The main grout absorptions, permeabilities, the geological formations drilled and construction times are presented in Plate 4.5).

Grouting results have been presented here by considering three main quantities:

1) the area of the curtain in a vertical sense (in m^2),
2) the amount of the grout used (in litres of grout or in kg of cement), and

Plate 4.5 Summary curtain grouting results, permeabilities, geological section, and curtain construction time (Pournari dam)

Note. The complete plan is included in the supplementary volume

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3) the length of grouting boreholes drilled (in metres-run).

The above quantities and their interrelations are used to designate the defective character (openings) of the treated foundation bedrock.

The grouting data have been grouped in basic segments within the curtain over an area of 50m horizontal distance and extending vertically to the depth of the limit line of the curtain, and they have been given in the form of grouting parameters, defined as follows:

i) Grout take This is the total weight (in kg of cement) of grout injected within a section of the curtain. When volumes are considered next, then these volumes are expressed in litres of grout of the 1 : 1 water/cement ratio (see Table AC-1 in Appendix C).

ii) Borehole length This is the total length (in metres) of grouting boreholes drilled within a segment.

iii) Grout take per metre-run of borehole This is the weight of grout injected over the total length of grout holes drilled within a segment. It is a parameter that is sensitive to the fracture condition of the ground, but can also be influenced by grout fluidity (w:c ratio) and pump pressure.

iv) Grout take per area of segment This is the weight of grout placed within a section of the curtain (segment) divided by the section area. It takes account of the averaged distribution of the grout within the section, but is indicative of the relative fracture density in the rock over the different segments. This indication of fracture density is weakened because no account is taken in this parameter of grout borehole density; the greater the number of boreholes (and their length) per unit area of segment, the greater is the chance of grouting more cracks in the foundation rock.

v) Grout take per m-run per area of segment This is the weight of grout accepted by the rock expressed in terms both of the borehole length and the segment area. It is a parameter that normalizes the grout weight to curtain area (parameter iv) for comparisons between segments, but which has the advantage of overcoming the objection in (iv) by incorporating the ground fracture sensitivity factors (iii).

vi) Volume of grout used per m-run per area of segment This parameter offers the advantages of (v) parameter above, but also has the advantages of a dimensionless parameter by converting grout weight to grout volume by either assuming an average grout density or by taking account of specific w:c ratios*.

vii) Succession of grouting boreholes

P : Primary S : Secondary T : Tertiary Q : Quaternary	These are the stages of boreholes put down for grouting.

* The more fluid the grout (the higher the water-cement ratio) the more likely is the grout from the numerous boreholes to spread differentially out of the plane of the dam axis section. In such circumstances the validity of the grouting parameters detailed above becomes less tenable.
A substantial percentage of the field grouting used a 1:1 (low mobility) water-cement grout. This w:c ratio is used for the calculation in this work.

TABLE 4.3

Main Grout Curtain Results

Segment Nº / Grouted surface (m^2)		1 / 724	2 / 4222	3 / 3569	4 / 3625	5 / 4134	6 / 2955	7 / 2800	8 / 2560	9 / 2510	10 / 2500	11 / 2500	12 / 2500	13 / 2500	14 / 2555	15 / 2900	16 / 3160	17 / 4150	18 / 4030	19 / 2850
Grout Take per Segment (kg)		65	1389	4995	9507	17468	307	991	2633	3095	6878	5006	6576	7985	10390	17615	15372	18468	15678	7568
Grouting B.H. Length (m)		66	625	552	636	917	465	432	545	498	636	530	612	603	742	978	869	1264	1088	571
Density of B.H (m/m^2)		0.091	0.15	0.15	0.17	0.21	0.15	0.15	0.21	0.20	0.25	0.21	0.24	0.24	0.29	0.34	0.27	0.30	0.27	0.24
Grout Take per m-run of B.H (kg/m)		0.98	2.22	8.97	14.95	19.05	0.66	2.29	4.46	6.21	10.34	9.44	10.74	13.24	14.00	17.80	17.68	14.84	14.41	13.25
Grout Take per Area of Segment ($kg\ m^2$)		0.089	0.33	1.40	2.62	4.22	0.10	0.35	0.95	1.23	2.75	2.00	2.6	3.19	4.06	6.00	4.86	4.45	3.85	3.22
Grout Take per m-run per Area of Segment (kg/m^3)		8.1×10^{-3}	49.5×10^{-3}	210.0×10^{-3}	445.4×10^{-3}	886.2×10^{-3}	15.0×10^{-3}	52.50×10^{-3}	199.5×10^{-3}	266.0×10^{-3}	687.1×10^{-3}	422.0×10^{-3}	626.0×10^{-3}	765.6×10^{-3}	1177.4×10^{-3}	2040.0×10^{-3}	1312.2×10^{-3}	1335.0×10^{-3}	1039.5×10^{-3}	772.8×10^{-3}
Volume of grout used per m-run per Area of Segment. (m^3/m^3 = unit)		10.78×10^{-3}	63.91×10^{-3}	279.6×10^{-3}	593×10^{-3}	1180.2×10^{-3}	19.9×10^{-3}	69.9×10^{-3}	263.6×10^{-3}	327.5×10^{-3}	914.0×10^{-3}	561.9×10^{-3}	830.9×10^{-3}	1019.4×10^{-3}	1567.7×10^{-3}	2716.5×10^{-3}	1747.2×10^{-3}	1777.6×10^{-3}	1386.1×10^{-3}	1029.0×10^{-3}
P	Grout Take (kg)	50	811	1960	2437	6661	190	242	860	552	2312	1720	2905	3950	3195	1885	6310	7312	7548	2657
P	Length of B.H. (m)	20	345	300	268	376	254	212	181	158	250	200	200	205	210	180	267	609	319	196
S	Grout Take (kg)	5	351	2270	1205	8125	67	669	650	698	1645	1076	1210	1229	2025	4705	3511	7634	3265	2201
S	Length of B.H. (m)	8	85	130	113	209	128	125	150	140	160	120	120	169	133	161	189	329	231	212
T	Grout Take (kg)	10	227	765	5237	2552	50	80	718	1290	1355	980	743	1695	2735	6362	3210	3112	2784	2710
T	Length of B.H. (m)	38	195	122	183	210	83	95	158	122	168	72	133	157	179	273	176	623	411	163
Q	Grout Take (kg)	Ø	Ø	Ø	498	1108	Ø	Ø	135	385	1200	875	80	60	775	2638	1606	Ø	2101	Ø
Q	Length of B.H. (m)	Ø	Ø	Ø	50	96	Ø	Ø	32	48	68	88	24	26	98	210	155	Ø	127	Ø
E and C	Grout Take (kg)	Ø	Ø	Ø	130	130	Ø	Ø	70	170	365	355	1638	1051	1460	2025	735	410	Ø	Ø
E and C	Length of B.H (m)	Ø	Ø	Ø	22	30	Ø	Ø	24	30	32	50	135	68	130	154	102	83	Ø	Ø

TABLE 4·4

Main Grout Curtain Results of Primary (P) Boreholes

Segment N°	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	
Grouted Surface Area (m^2)	724	4222	3569	3625	4134	2955	2800	2560	2510	2500	2500	2500	2500	2555	2900	3160	4150	4056	2350	
Grout Take (kg cement)	50	811	1960	2437	6661	190	242	860	552	2315	1720	2905	3950	3195	1885	6310	7312	7548	2657	
Grouting B.H. Length (m)	20	345	300	268	374	254	212	181	158	250	200	200	205	210	180	247	409	319	196	
Density of B.H. (m/m^2)	0.028	0.081	0.084	0.073	0.090	0.085	0.075	0.070	0.062	0.100	0.080	0.080	0.082	0.082	0.062	0.078	0.098	0.078	0.083	
Grout Take per m-run of B.H. (kg/m)	2.5	2.35	6.53	9.09	17.81	0.74	1.14	4.75	3.49	0.26	8.6	14.52	19.26	15.21	10.47	25.54	17.87	23.66	13.55	
Grout Take per Area of Segment. (kg/m^2)	0.069	0.19	0.54	0.67	1.61	0.06	0.08	0.33	0.21	0.92	0.68	1.16	1.58	1.25	0.65	1.99	1.76	1.86	1.13	
Grout Take per m-run per Area of Segment (kg/m^3)	1.93×10^{-3}	15.4×10^{-3}	45.36×10^{-3}	48.9×10^{-3}	144.9×10^{-3}	5.1×10^{-3}	6.0×10^{-3}	23.1×10^{-3}	130.2×10^{-3}	92×10^{-3}	54.4×10^{-3}	92.8×10^{-3}	129.5×10^{-3}	102.5×10^{-3}	40.3×10^{-3}	155.2×10^{-3}	172.5×10^{-3}	145.1×10^{-3}	93.8×10^{-3}	
Volume of grout used per m-run per Area of Segment. (m^3/m^3 unit)	2.57×10^{-3}	20.5×10^{-3}	60.4×10^{-3}	65.1×10^{-3}	192.9×10^{-3}	6.8×10^{-3}	7.98×10^{-3}	30.75×10^{-3}	173.3×10^{-3}	122×10^{-3}	72.43×10^{-3}	123.5×10^{-3}	172.4×10^{-3}	136.5×10^{-3}	53.66×10^{-3}	206.6×10^{-3}	229.7×10^{-3}	193.2×10^{-3}	124.9×10^{-3}	

TABLE 4·5

Main Grout Curtain Results of Secondary (S) Boreholes

Segment N°	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	
Grouted Surface Area (m^2)	724	4222	3569	3625	4134	2955	2800	2560	2510	2500	2500	2500	2500	2555	2900	3160	4150	4050	2350	
Grout Take (kg cement)	5	351	2270	2205	8125	67	669	650	698	1645	1076	1210	1229	2025	4705	3511	7634	3245	2201	
Grouting B.H. Length (m)	8	85	130	113	209	128	125	150	140	160	120	120	149	133	161	189	329	231	212	
Density of B.H. (m/m^2)	0,011	0,020	0,036	0,031	0,050	0,043	0,044	0,058	0,055	0,064	0,048	0,048	0,059	0,052	0,055	0,059	0,079	0,057	0,090	
Grout Take per m-run of B.H. (kg/m)	0.62	4.12	17,46	10,66	38,87	0.35	5.35	4.33	4.98	10,28	8.96	10,08	8.24	15,22	29,22	18,57	23,20	14,04	10,38	
Grout Take per Area of Segment. (kg/m^2)	0,006	0,083	0,636	0,332	1,965	0,022	0,238	0,254	0,278	0,658	0,430	0,484	0,491	0,792	1,622	1,111	1,839	0,801	0,936	
Grout Take per m-run per Area of Segment (kg/m^3)	0,066 x 10^{-3}	1.66 x 10^{-3}	22.8 x 10^{-3}	10.3 x 10^{-3}	9,825 x 10^{-3}	0,946 x 10^{-3}	10.5 x 10^{-3}	14.7 x 10^{-3}	15.3 x 10^{-3}	42.1 x 10^{-3}	20.6 x 10^{-3}	23.2 x 10^{-3}	44.2 x 10^{-3}	41.2 x 10^{-3}	89.2 x 10^{-3}	65.5 x 10^{-3}	145,3 x 10^{-3}	45.6 x 10^{-3}	84.2 x 10^{-3}	
Volume of grout used per m-run per Area of Segment. (m^3/m^2 unit)	0,087 x 10^{-3}	2.21 x 10^{-3}	30,35 x 10^{-3}	13.7 x 10^{-3}	13,08 x 10^{-3}	1.26 x 10^{-3}	13,98 x 10^{-3}	19,57 x 10^{-3}	21,54 x 10^{-3}	56,06 x 10^{-3}	27,43 x 10^{-3}	30.9 x 10^{-3}	58.8 x 10^{-3}	54,86 x 10^{-3}	11,87 x 10^{-3}	87.2 x 10^{-3}	193,5 x 10^{-3}	60.7 x 10^{-3}	112,1 x 10^{-3}	

TABLE 4·6

Main Grout Curtain Results of Tertiary (T) Boreholes

Segment N°	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	
Grouted Surface Area (m^2)	724	4222	3569	3625	4134	2955	2800	2560	2510	2500	2500	2500	2500	2555	2900	3160	4150	4050	2350	
Grout Take (kg cement)	10	227	765	5237	2552	50	80	718	1290	1355	980	743	1695	2735	6362	3210	3112	2784	2710	
Grouting B.H. Length (m)	38	195	122	183	210	83	95	158	122	148	72	133	157	179	273	176	423	411	163	?
Density of B.H. (m/m^2)	0.052	0.046	0.034	0.050	0.050	0.028	0.033	0.061	0.048	0.059	0.028	0.053	0.062	0.070	0.094	0.055	0.101	0.101	0.069	
Grout Take per m-run of B.H. (kg/m)	0.26	1.16	6.27	28.61	12.15	0.60	0.84	4.54	10.57	6.75	13.61	5.58	10.79	15.28	23.30	18.23	7.35	6.77	16.62	
Grout Take per Area of Segment (kg/m^2)	0.013	0.053	0.214	1.444	0.617	0.016	0.028	0.280	0.513	0.542	0.392	0.297	0.678	1.070	2.193	1.015	0.749	0.687	1.153	
Grout Take per m-run per Area of Segment (kg/m^3)	0.67×10^{-3}	2.43×10^{-3}	7.27×10^{-3}	72.2×10^{-3}	30.85×10^{-3}	0.45×10^{-3}	0.92×10^{-3}	17.1×10^{-3}	14.6×10^{-3}	31.9×10^{-3}	10.9×10^{-3}	15.7×10^{-3}	42.0×10^{-3}	74.9×10^{-3}	206.1×10^{-3}	55.8×10^{-3}	75.6×10^{-3}	69.38×10^{-3}	77.97×10^{-3}	
Volume of grout used per m-run per Area of Segment (m^3/m^3 unit)	0.89×10^{-3}	3.23×10^{-3}	9.68×10^{-3}	96.14×10^{-3}	41.1×10^{-3}	0.6×10^{-3}	1.22×10^{-3}	22.76×10^{-3}	32.75×10^{-3}	42.5×10^{-3}	14.5×10^{-3}	20.9×10^{-3}	55.9×10^{-3}	99.7×10^{-3}	274.4×10^{-3}	74.3×10^{-3}	100.69×10^{-3}	92.38×10^{-3}	103.82×10^{-3}	

TABLE 4·7

Main Grout Curtain Results of Quaternary (Q) Boreholes

Segment N°	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
Grouted Surface Area (m^2)	724	4222	3569	3625	4134	2955	2800	2560	2510	2500	2500	2500	2500	2555	2900	3160	4150	4050	2350
Grout Take (kg cement)	Ø	Ø	Ø	498	1108	Ø	Ø	135	385	1200	875	80	60	775	2438	1606	Ø	2101	Ø
Grouting B.H. Length (m)	Ø	Ø	Ø	50	94	Ø	Ø	32	48	48	88	24	24	98	210	155	Ø	127	Ø
Density of B.H. (m/m^2)	Ø	Ø	Ø	0.013	0.022	Ø	Ø	0.012	0.019	0.019	0.035	0.009	0.009	0.038	0.072	0.049	Ø	0.031	Ø
Grout Take per m-run of B.H. (kg/m^3)	Ø	Ø	Ø	9.96	11.78	Ø	Ø	4.21	8.02	25.00	9.94	3.33	2.50	7.90	11.60	10.36	Ø	16.54	Ø
Grout Take per Area of Segment (kg/m^2)	Ø	Ø	Ø	0.137	0.268	Ø	Ø	0.052	0.153	0.480	0.350	0.032	0.024	0.303	0.840	0.508	Ø	0.518	Ø
Grout Take per m-run per Area of Segment (kg/m^4)	Ø	Ø	Ø	1.8×10^{-3}	5.9×10^{-3}	Ø	Ø	0.62×10^{-3}	2.9×10^{-3}	9.12×10^{-3}	12.25×10^{-3}	0.28×10^{-3}	0.21×10^{-3}	11.5×10^{-3}	60.5×10^{-3}	24.8×10^{-3}	Ø	16.06×10^{-3}	Ø
Volume of grout used per m-run per Area of Segment (m^3/m^3 unit)	Ø	Ø	Ø	2.4×10^{-3}	7.85×10^{-3}	Ø	Ø	0.82×10^{-3}	3.86×10^{-3}	12.4×10^{-3}	16.31×10^{-3}	0.38×10^{-3}	0.28×10^{-3}	15.3×10^{-3}	80.5×10^{-3}	33.0×10^{-3}	Ø	21.4×10^{-3}	Ø

TABLE 48

Main Grout Curtain Results of Check (E and C) Boreholes

Segment N°	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	
Grouted Surface Area (m^2)	724	4222	3569	3625	4134	2955	2800	2560	2510	2500	2500	2500	2500	2555	2900	3160	4150	4050	2350	
Grout Take (kg cement)	Ø	Ø	Ø	130	130	Ø	Ø	70	170	365	355	1638	1051	1460	2095	735	410	Ø	Ø	
Grouting B.H. Length (m)	Ø	Ø	Ø	22	30	Ø	Ø	24	30	32	50	135	68	130	154	102	83	Ø	Ø	
Density of B.H. (m/m^2)	Ø	Ø	Ø	0.006	0.007	Ø	Ø	0.009	0.012	0.013	0.02	0.054	0.027	0.050	0.053	0.032	0.02	Ø	Ø	
Grout Take per m-run of B.H. (kg/m)	Ø	Ø	Ø	5.90	4.33	Ø	Ø	2.91	5.66	11.40	7.10	10.13	15.45	11.23	13.15	7.20	4.94	Ø	Ø	
Grout Take per Area of Segment. (kg/m^2)	Ø	Ø	Ø	0.035	0.031	Ø	Ø	0.027	0.067	0.146	0.142	0.655	0.420	0.571	0.698	0.232	0.098	Ø	Ø	
Grout Take per m-run per Area of Segment (kg/m^3)	Ø	Ø	Ø	0.21×10^{-3}	0.21×10^{-3}	Ø	Ø	0.24×10^{-3}	0.8×10^{-3}	1.9×10^{-3}	2.8×10^{-3}	35.3×10^{-3}	11.3×10^{-3}	28.5×10^{-3}	36.9×10^{-3}	7.4×10^{-3}	1.96×10^{-3}	Ø	Ø	
Volume of grout used per m-run per Area of Segment. (m^3/m^3 unit)	Ø	Ø	Ø	0.28×10^{-3}	0.28×10^{-3}	Ø	Ø	0.32×10^{-3}	1.06×10^{-3}	2.53×10^{-3}	3.7×10^{-3}	47.0×10^{-3}	15.0×10^{-3}	37.9×10^{-3}	49.13×10^{-3}	9.8×10^{-3}	2.6×10^{-3}	Ø	Ø	

E : non-cored	)		These are the final boreholes
	)	Check	put down for checking the
C : cored	)	holes	completeness of the earlier grouting

For evaluation purposes, the detailed grouting field results are gathered together in Table 4.3, and are given as appropriate grouting parameters (defined previously) for each basic segment. The grouting results in the succession of Primary, Secondary, Tertiary, Quaternary and Check holes are given in Tables 4.4 to 4.8.

The averaged results of grout takes in kg of cement per m^2, boreholes drilled in m-run per m^2, and the volume of grout used per m-run per area of segment (dimensionless parameter), as well as the treated area in m^2 of each segment, are illustrated in Fig. 4.12. In this Figure, differences such as those exhibited in segments 1, 5, 6 and 10, or similarities as those exhibited in segments 16, 17, 18 and 19, facilitate the evaluation of the predominant geological factors involved.

The effectiveness of grouting treatment in the succession of P, S, T, Q, E and C grouting stages, are presented in Figure 4.13. Differences, such as those of segments 4, 5 and 6, are indicative of depth and frequency of the most important groutable discontinuities and the lithological horizons cited in the foregoing paragraphs.

Finally, Figures 4.14 and 4.15 illustrate the effectiveness of the drilling and grouting effort in the succession of the P, S, T, Q stages and the checking stages E and C. The effectiveness achieved as illustrated by the P, S, T, Q, E and C curves act as warnings and guidance towards the appropriate future measures needed to supplement the grouting works already performed.

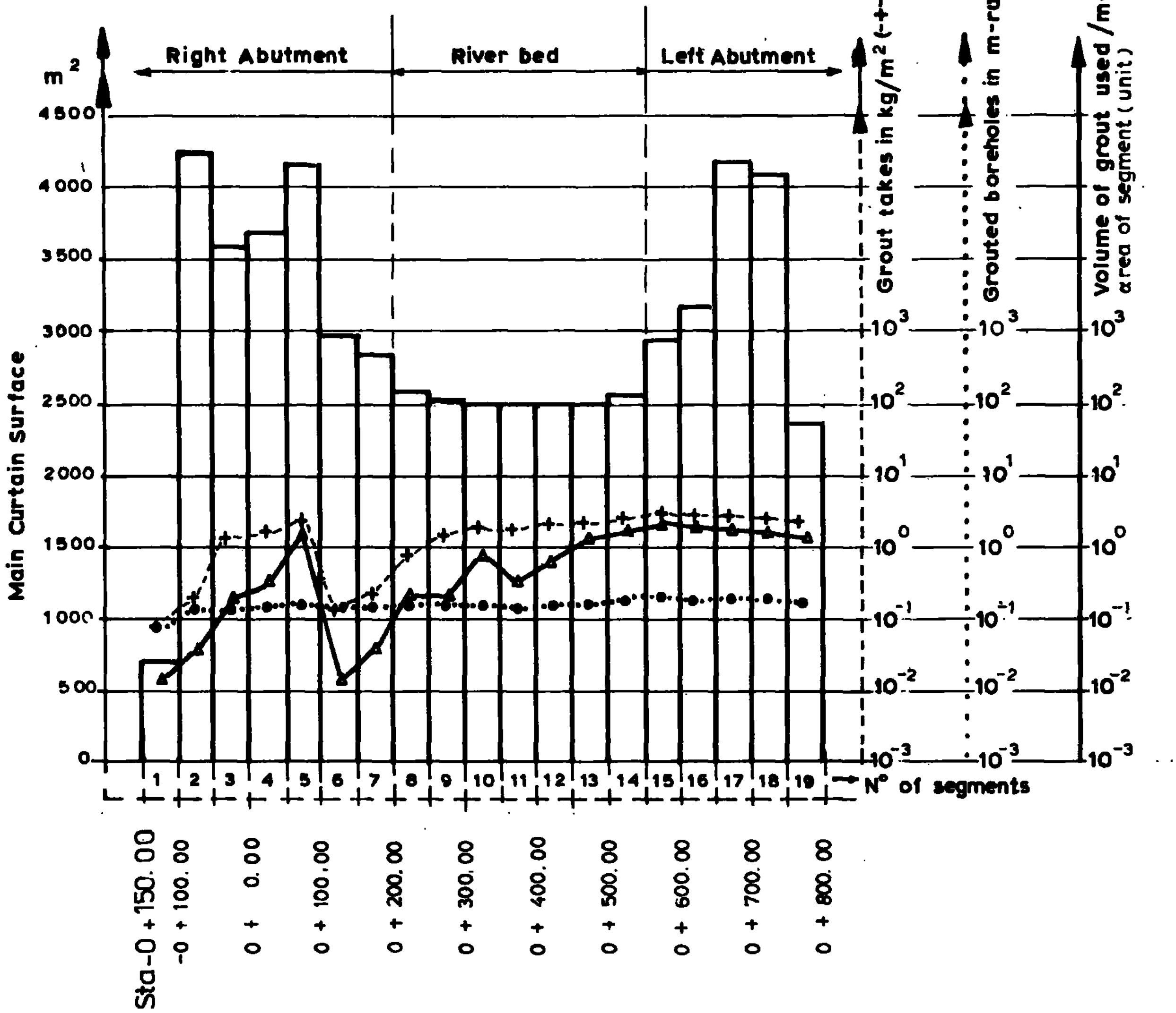


Fig. 4 : 12 Foundation grouting treatment (Pournari dam) and behavour in terms of.

1. Main grout curtain extension along dam axis(m^2)
2. Borehole density in m-run per m^2 of curtain.
3. Grout takes per m^2 of the curtain.
4. Vol. of grout / m-run of bh. / area of segment (unit)

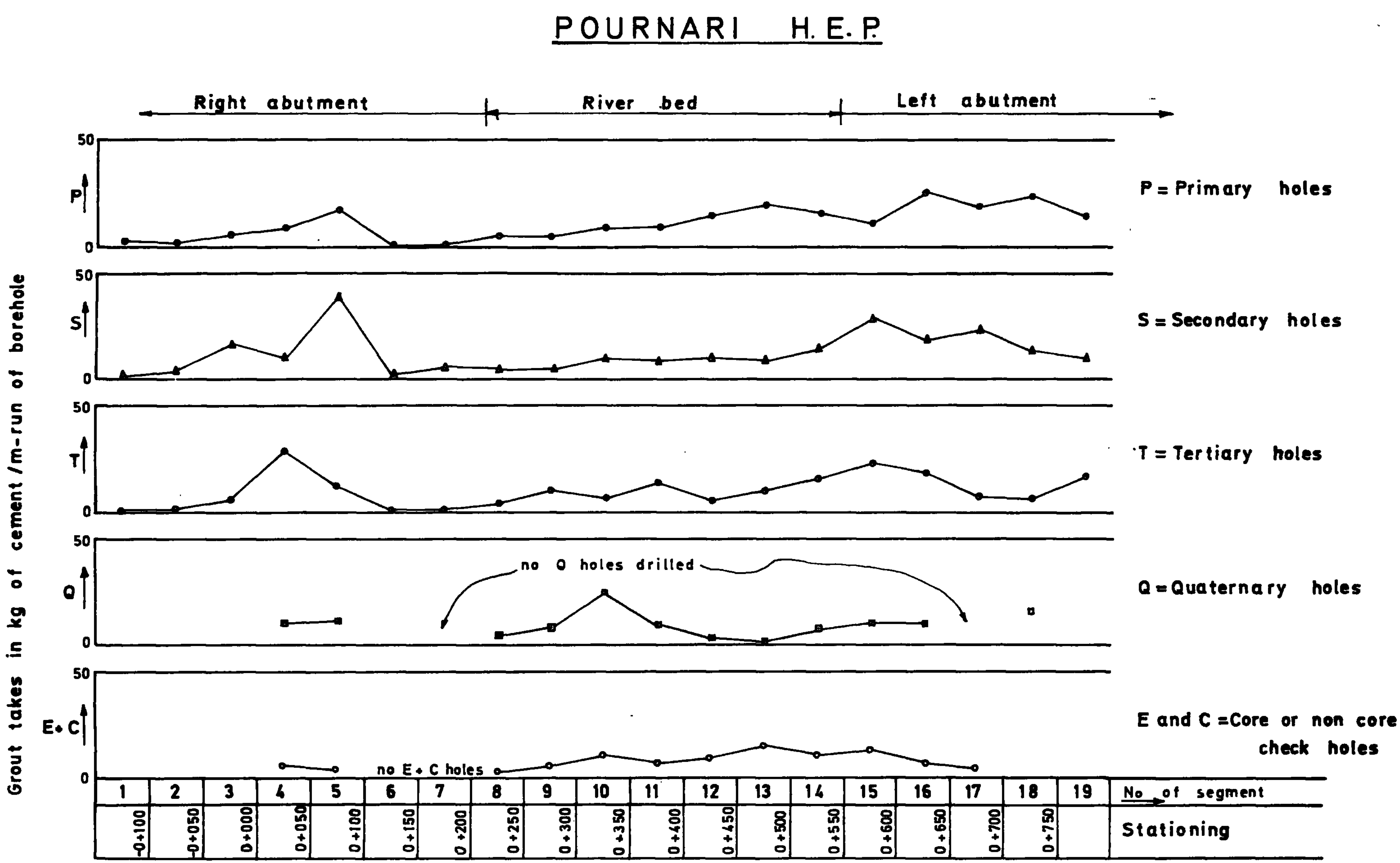


Fig. 4·13 Comparative results of foundation bedrock behaviour in grouting treatment along the main Pournari grout curtain (Plots represent the average grout take in kg of cement per m-run of borehole for each segment considered).

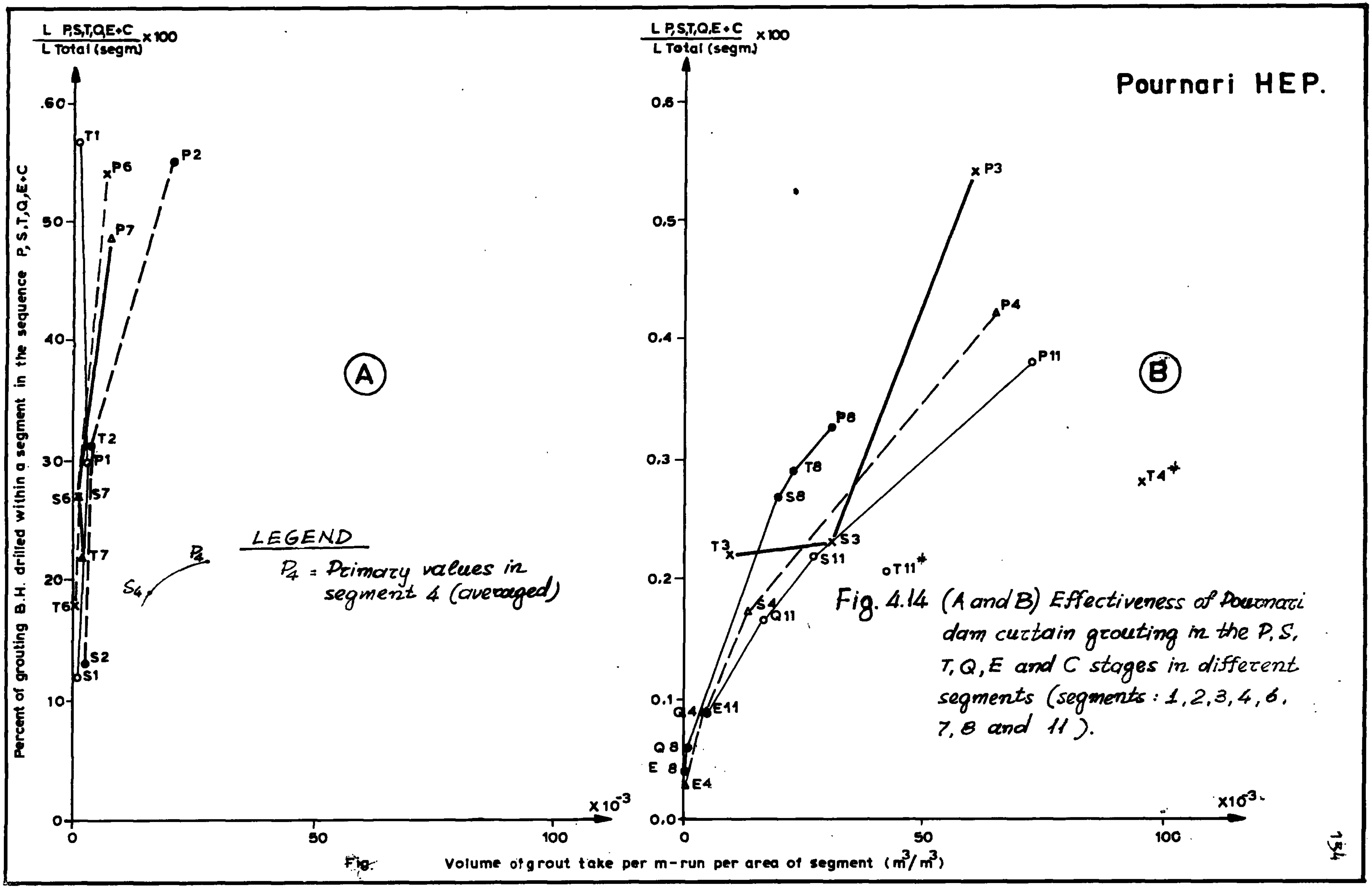


Fig. 4.14 (A and B) Effectiveness of Pournari dam curtain grouting in the P, S, T, Q, E and C stages in different segments (segments: 1, 2, 3, 4, 6, 7, 8 and 11).

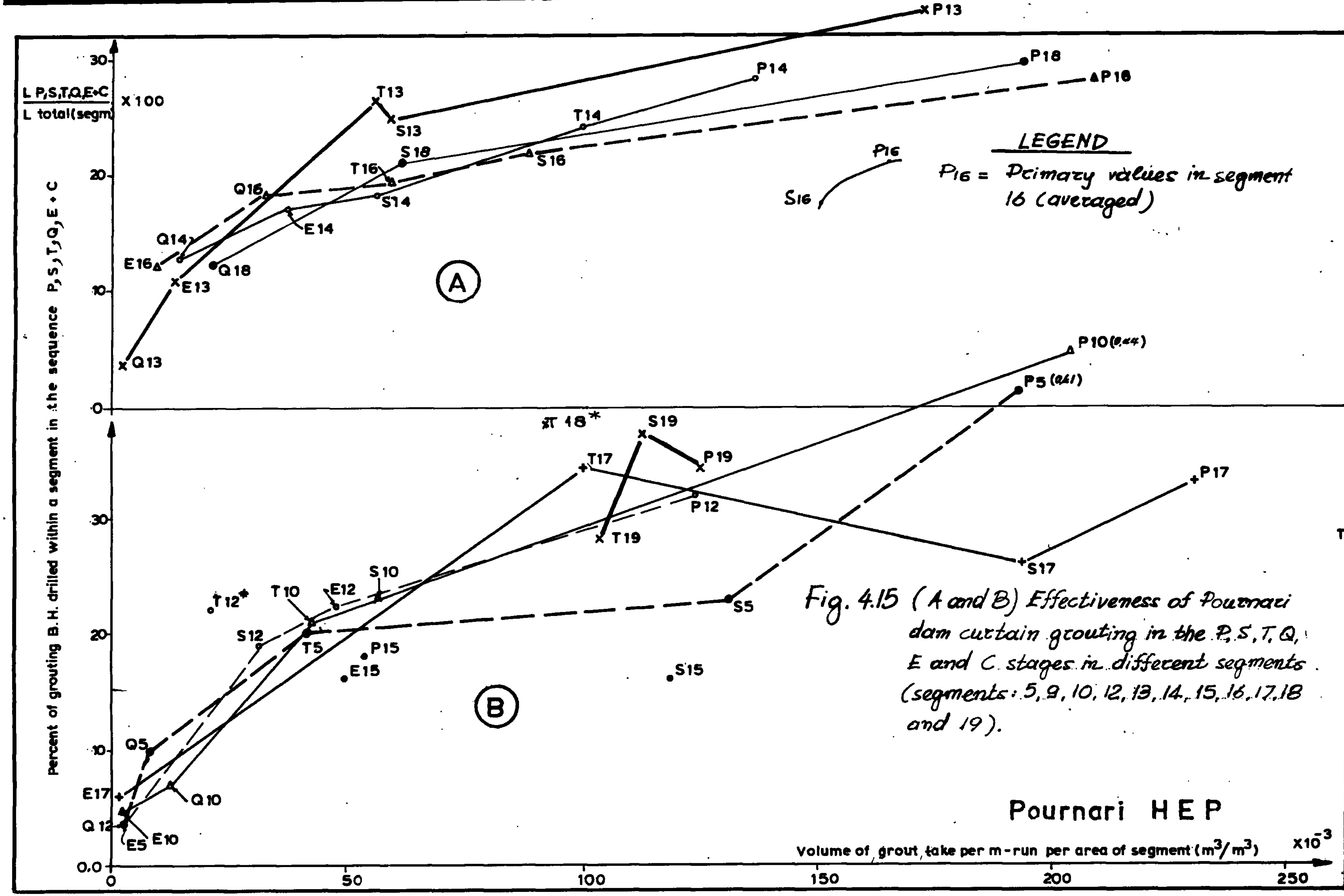


Fig. 4.15 (A and B) Effectiveness of Pournari dam curtain grouting in the P, S, T, Q, E and C stages in different segments (segments: 5, 9, 10, 12, 13, 14, 15, 16, 17, 18 and 19).

It is thus possible to isolate and specify the existing or assumed geological factors involved.

It is noted that it is often necessary to perform further grouting or drainage works after completion of the initial work. An assessment of the foundation bedrock discontinuity characteristics with respect to grout take will benefit both the design of the operation and its cost effectiveness.

4.2.5 Summary discussion and conclusions on Pournari dam

On the basis of the grouting results and other field information available, as presented in this Section, a number of conclusions can be drawn, in spite of the complex nature of the variables considered, and the uncertainties which usually accompany the geological factors involved. In the evaluation of the grouting results, some correlations can be made between

a) the grouting parameters (quantities), and the foundation conditions (rock voidness), and

b) the extent of the effectiveness of the grouting works and the preset completion criteria (see also, Appendix C).

On the other hand a number of ambiguities and the need for additional information from specific tests, geological records and observations are brought to light when an attempt is made

a) to evaluate the foundation properties and their response to the methods of improvement applied, and

b) to grade their interrelations.

Because of (in some cases) restricted facilities in the field, and the employment of the writer in the exploratory programme (final design and bidding contract documents) of the Pigae damsite on the Aoos River, these tests and the other information needed could not be carried out. Suggestions for further research will be detailed in due course.

The curves (Fig. 4.12) of the average results of the grout used per unit surface (m^2) of the basic segmental divisions of the grout curtain indicate that the quantities of grout used are

a) higher in segments 5, 15, 16, 17, 18 and 19 due mainly to presence of the sandstone formations and their similarities of jointing and weathering, and
b) lower in segments 1, 2, 6 and 7 in which sandstone-bearing strata are absent or weathered and relaxed strata have been excavated to fresh rock.

The density of the boreholes drilled is about the same in all segments except in segment 1. Variations in the quantities of the grout used, such as those in segments 10 and 15 or 6 and 7, must be attributed to factors pertaining to these segments.
The factors are:

a) The joint concentration on the apex of the river bed fold.
b) The near-vertical and dilated joints in the abrupt cliff of the left abutment (rebound joints).
c) The absence of open joints in the silty conglomerates of the siltstone sequence, particularly at depth (10m) below the surface.

Generally, the grout takes shown in the "as built" drawings and the conclusions drawn out from the evaluation of the investigation stage permeability results are in agreement with the response of the ground to the grouting treatment applied.

In Figure 4.13 it is evident that the effectiveness of the succession of the grouting stages of P, S, T, Q, E and C holes in sealing the bedrock voids presents some anomalies. These anomalies comprise

a) the high grout takes of the Tertiary holes in segment 4 which can be attributed to construction (see the detailed "as built" drawings), and

b) the high grout takes of the Quaternary holes in segment 10 which have been caused by the strong and closely-spaced jointing due to the fold present in this segment.

Other anomalies, similar to the above, which concern the remainder of the segments are minimal and are attributed to the relaxation of the near surface rocks (particularly where shattered rocks occurred) and to the losses of grout in the effort to seal the near surface (upper) grouted sections of the holes.

Finally, the effectiveness of the drilling and grouting effort in the succession of the P, S, T, Q stages and the checking stages E and C, is illustrated in Figures 4.14 and 4.15.

In particular, the curves of the Fig. 4.14 (A and B) relate to tight bedrock conditions and the isolated high grout takes are attributed to construction defects (concrete lining). The curves indicate that although the grouting borehole density is high (as high as in the other segments; see Table 4.3 and Fig. 4.12), the reduction in the values of the dimensionless parameter of "volume of grout take per metre run per area of segment" is minimal. This minimal grout absorption implies a very tight bedrock (the silty conglomerates and siltstone sequence) and that at least the existing joints are still in compression under stress and only the near-surface (about 0-10m deep) rocks exhibited some dilation and stress relief.

The curves of Figs.4.15$_{A \text{ and } B}$ indicate that the groutable openings in the foundation bedrock have effectively reduced (at least after the Tertiary holes grouting stage), but some anomalies indicate that some small open joints in segments 15 and 19 still exist. Drainage holes in the left abutment have indicated the existence of small leakages (a few litres of water per minute) in occasional holes.

The above remarks warn about the potential danger of a build-up of pore pressures if a similar behaviour of segments is to be encountered in other curtains elsewhere and that properly-designed drainage curtains should prove to be more cost effective than incurring the need to resort to further grouting subsequent to the main programme.

4.3 Assomata dam

4.3.1 Description of Project

Assomata project is still under construction (at the time of writing) and due to start generating electric power during the second half of 1984.

It comprises an earthfill dam 66 metres in height from the river-bed foundation bedrock (elevation 26 metres) to crest (elevation 92 metres). Between the very abrupt (almost vertical) right abutment and the flatter left abutment, the river gorge at the dam axis exhibits a width of 60 metres at riverbed foundation bedrock level and a width of 200 metres at crest. The clay core and the supporting fill abut on the right hand concrete wall of the spillway (see Fig. 4.16 and Plate 4.6).

FIGURE 4.16

Assomata dam layout

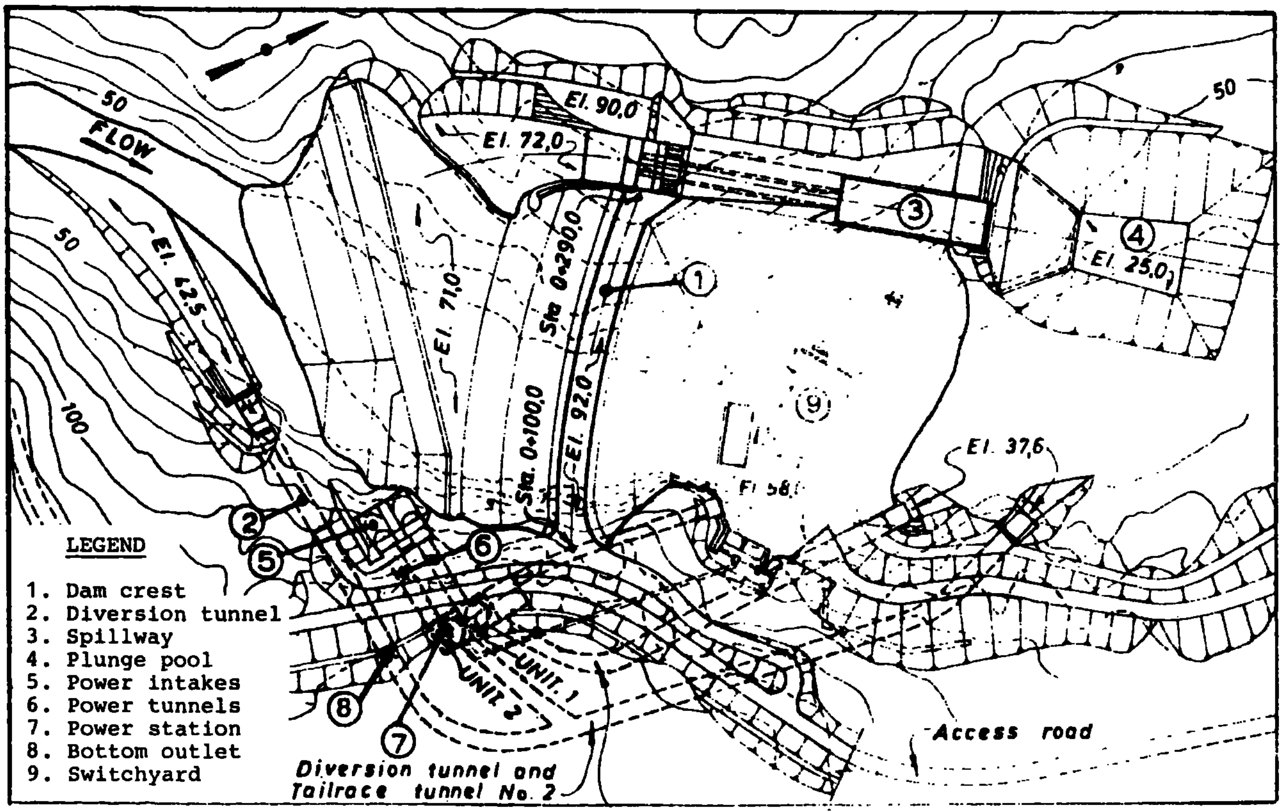


Plate 4.6 Assomata dam construction

Within the strong right abutment are located the concrete lined diversion tunnel, 500 metres long and 7.5 metres in diameter, and the underground power house with its appurtenances (power waterways, access tunnel and so on). The final half of the diversion tunnel is due to be utilized as the tailrace tunnel No 2 (Fig. 4.16).

The spillway is located on the left abutment and part of its chute consists of a cut and cover concrete box section, which protects it from potential instability of the sheared serpentinites of the left abutment.

The dam volume contains 1.5 x $10^6 m^3$ of construction materials.

The outer zone of its downstream gravel shell includes oversized quarried limestones as protection in case of limited overtopping.

4.3.2 Foundation bedrock conditions: an assessment prior to grouting

Bedrock conditions and the anticipated corrective measures for strengthening and sealing of the foundation bedrock were appraised and conducted in a similar manner to that programme adopted for Pournari dam (see Section 4.2.2). From the bulk of the information collected as above, selective features will be used in this study to delineate foundation bedrock conditions as follows.

A. Geomorphological and lithological characteristics of the damsite

Assomata damsite lies 8 kilometres south of Veria town at the exit of the lower Aliakmon River gorge.
From upstream to downstream the following lithological units are observed:

a) Thickly-bedded to massive crystalline limestones, which form the upstream bound of the dam, dipping north-easterly under the damsite. The limestones in the right abutment are located immediately upstream of the diversion tunnel entrance. At the left abutment they form part of the upstream coffer-dam foundation rocks. A contact fault separates the limestones from:

b) The dam foundation rocks (which include all dam structures) are a complex of ophiolitic serpentinites of basaltic origin (Campbell, 1976) and include several distinct rock types (see Figs. 4.17 and 4.18) and

c) Gneissic rocks outcropping a few hundred metres down-stream of the dam site (see Plate 3.2).

The following remarks concerning the foundation rocks along the dam axis during the core trench excavations are necessary for a proper assessment of the permeability and grouting data cited in the next paragraphs:

Right abutment rocks of the core trench consist of massive, strong, fresh to slightly-weathered greenish agglomeritic serpentinites. At elevation 70-80 metres a band of younger extrusive volcanic tuff rock, embedded within the older agglomerates, forms part of the under-ground powerhouse roof and portal rocks of the power intake tunnels.

River bed rocks consist of transitional serpentinitic schistose agglomerates. At the back of the right abutment they are separated from the strong overlying rocks by a sheared contact. Towards the left abutment the transi-tional agglomerates are gradually replaced by tuff bands.

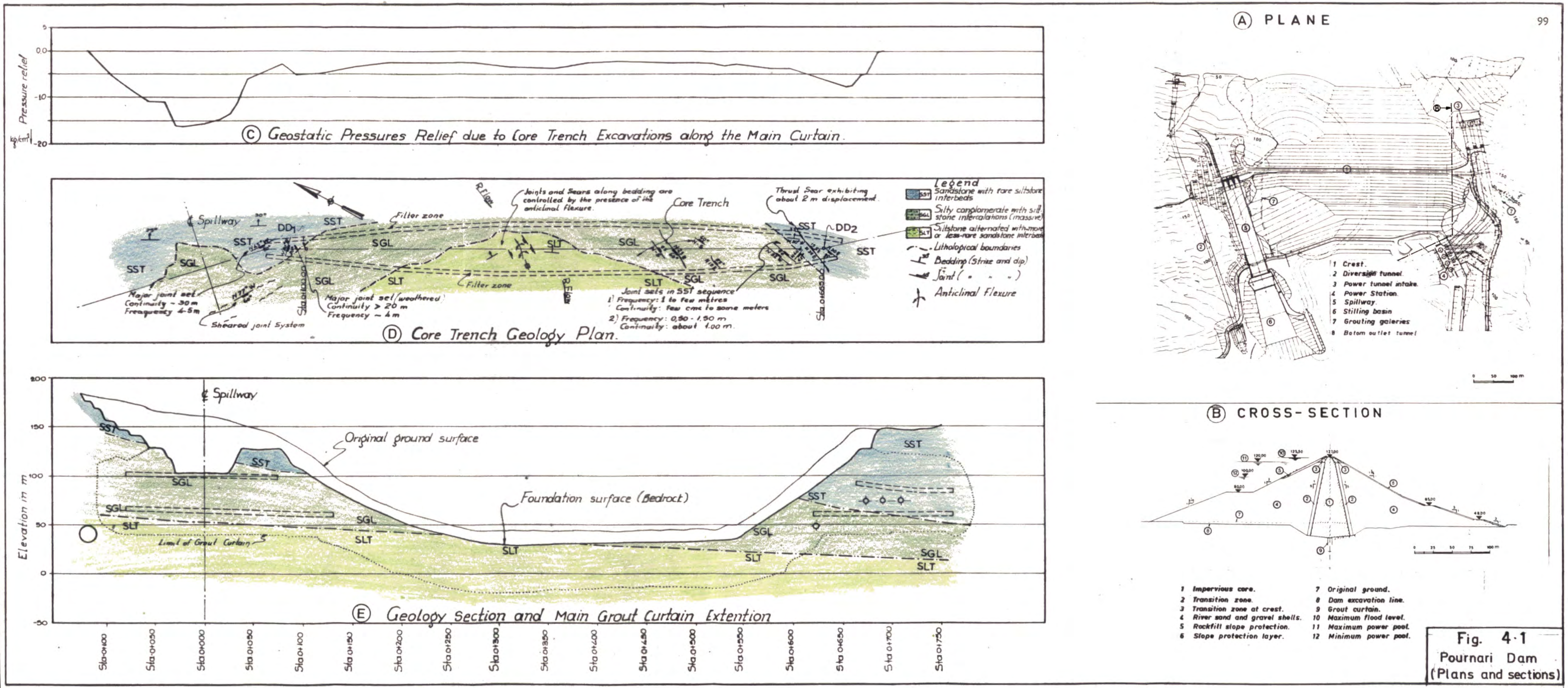
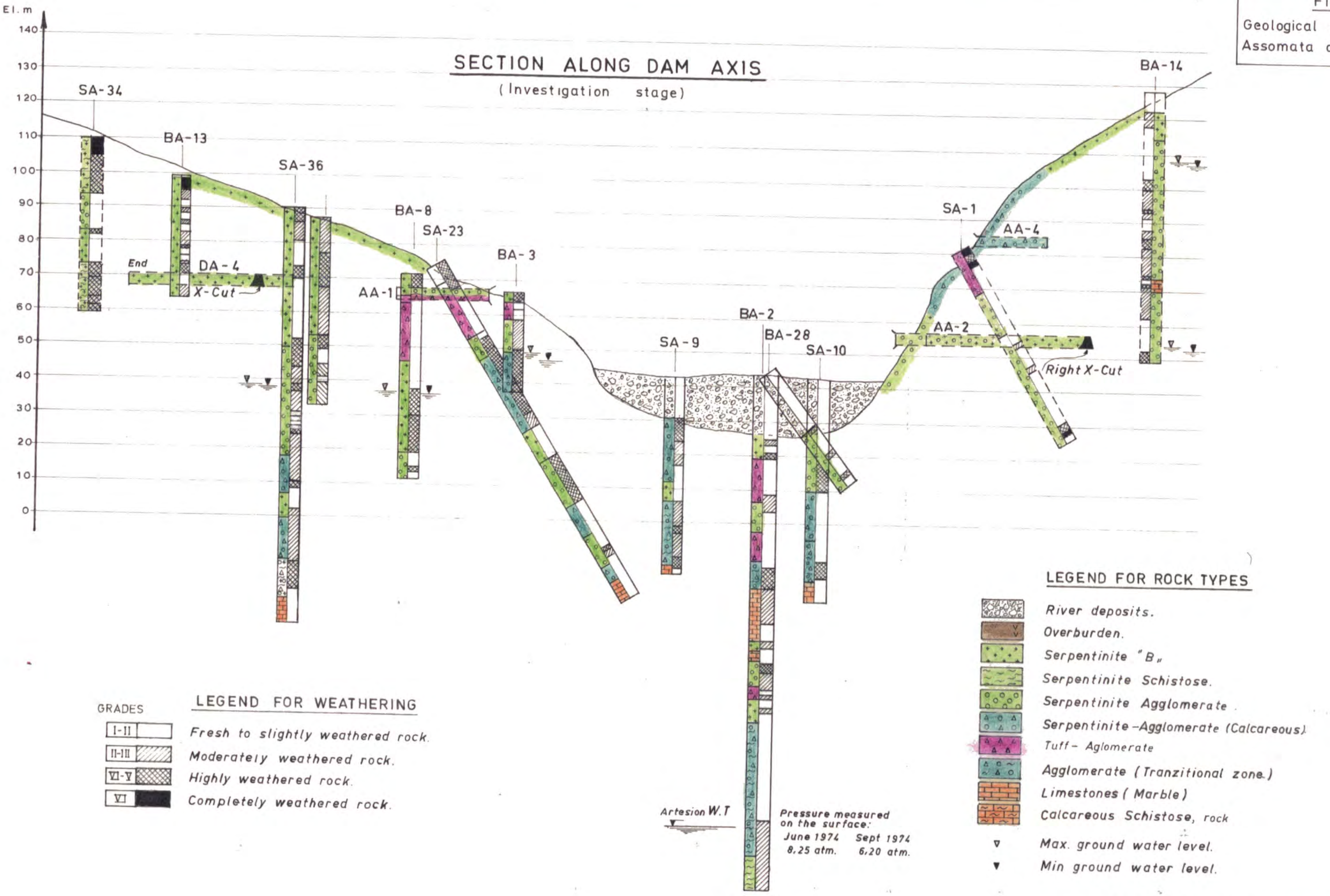


Fig. 4·1
Pournari Dam
(Plans and sections)

Fig. 4 · 17

Geological section along the Assomata dam axis.



ASSOMATA H-E. PROJECT

APRIL 1983, MD

Fig 4.18 ASSOMATA DAM
Core Trench Geology

LEGEND

V Overburden.

Serpentinite.

r) Basalt (Serpentinized)

)Lst Agglomerate (Serpentinitic Lst with limestones erratics)

3s) Agglomerate (Basaltic)

Ag) Basalt (Agglomeratic)

An) Tuff (Andesitic - agglomeratic)

Shear zone.

Limestone.

Sk. Serpentinite (Skin of snake' texture).

Phyllite.

Lithological boundaries.

Rock grade boundaries.

/30 Fault/Shear.

/65 Joint

/65 Shear joint/contact joint.

+++ Slide limit

o/35 Exposured traces of shears faults


ROCK WEATHERING GRADES
(I) Fresh (F)
(II) Slightly weathered (SW)
(III) Moderatery weathered (MW)
(IV) Highly weathered (HW)
(V) Completely weathered (CW)
(VI) Residual soil (R)
Access Road to Sfikia
River flow
N
0 10 20 30 40 50 m
Scale
El. 90.00
El 102.00
Bs (Ser) (III)
Ser (III)
Ag (II-III)
BS (Ag) (III)
Tuff
Tuffous rock

These bands are in a continuous mass of grey tuff agglomerates at the base of the left abutment. These tuff agglomerate rocks comprise:

the left abutment basement rocks up to elevation 40 metres and are traced by steeply-inclined (almost vertical) strongly brecciated narrow (20-40 cms) fault zones transversing the full width of the core trench.

Higher up, between elevation 40-60 m, the 20-25 m thick fault zone (main thrust at the left abutment) is overlain by a second band of relatively-sound tuff agglomerate and a massive serpentinite thick band on which the spillway gate structure is founded.

At the top of the damsite and the surrounding hills, terrace-like landforms exist. These are separated by steep gullies. Several minor or major landslides are recognized along the course of these gullies which surround the damsite.

The most important landslide is in the right abutment. This created some problems during construction and led to extensive excavations of the area above and adjacent to the outlet of the diversion tunnel.

A second landslide, relatively serious, is apparent in the left abutment in front of the spillway approach channel. This landslide was cleared and formed the foundation enclave of the main upstream cofferdam. In the same abutment two more minor landslides were identified but had been washed out by the river.

The landslides and the strong shearing of the left abutment suggest that high relaxation occurred there. This was proved during excavation by the discovery of open cracks and voids from a few centimetres to tens of centimetres wide (see Plate 4.7).

Plate 4.7 Assomata Dam. Riverbed and left abutment core trench excavations.

B. Bedding/schistocity and folds

Except for the limestones, no other strata exhibit bedding. The upstream bedded limestones show a dip of 50^{o}-65^{o} in the NE direction. A similar direction of dip, but at lower angles, has been assumed for the limestone horizons detected by holes below the river bed. Schistocity appears along major or minor faults present at the site, parallel or subparallel to the dip directions of these faults. Folds have not been identified within the dam site, but small schistocity variations in dip and dip direction are noticed.

In the massive serpentinitic and tuff rocks a banding is observed, but it is due only to variations in the fragment size, that is, coarser or finer inter-particles of these rocks. The size of the fragments ranges from millimetres to several centimetres, and in some cases to some metres (large size limestone erratics).

C. Joints

Joints at the site are better developed within the stronger lithological units. They exhibit a wide range of dips and dip directions, as well as frequency and extension, particularly influenced by the presence of minor or major faults.

The more systematically developed are those joints which are feather-like or oblique to the strike and dip of major faults.

These joints usually appear re-healed, with calcitic and serpentine veins. Locally they contain clayey infillings. These features make them quite impermeable, particularly in the right abutment. Joints adjacent to tuff rocks appear to be the most questionable from the point of view of permeability.

These joints are usually open and clean or with brecciated rock, and rare clayey or calcite infillings. Their spatial distribution together with the faults recorded is presented in Figs. 4.19 to 4.21 (see also Fig. 4.18).

D. Faults and shears

Numerous shears and faults trace the site as well as the surrounding area. The distinct difference in the appearance of the two abutments must be attributed to the faulting, which is present there, as well as the creation of the river bed by erosion.

Main faults and shear zones are illustrated in Fig. 4.18. These faults can be categorized as follows:

i. Right Abutment

Minor faults and shears (of a few centimetres thickness) and dipping upstream or downstream (Plate 4.8).

ii. River bed

Shallow dipping contact faults dipping beneath the right abutment (Plate 4.8).

iii. Left abutment

There are two groups. The one group at the base of the left abutment is nearly vertical, striking E-W and dipping towards the abutment. The second group concerns three main thrust faults. The major fault controls the sheared serpentinites of the left abutment (from elevation 40 m to elevation 60 m; see also Plate 4.9, and Fig. 4.18). The second fault is in the spillway gate foundation rocks (with a thickness of 1 to 3 metres). The third fault is downstream of the dam axis beyond the flip bucket of the spillway.

Plate 4.8 River bed and right abutment core trench excavations (Assomata dam)

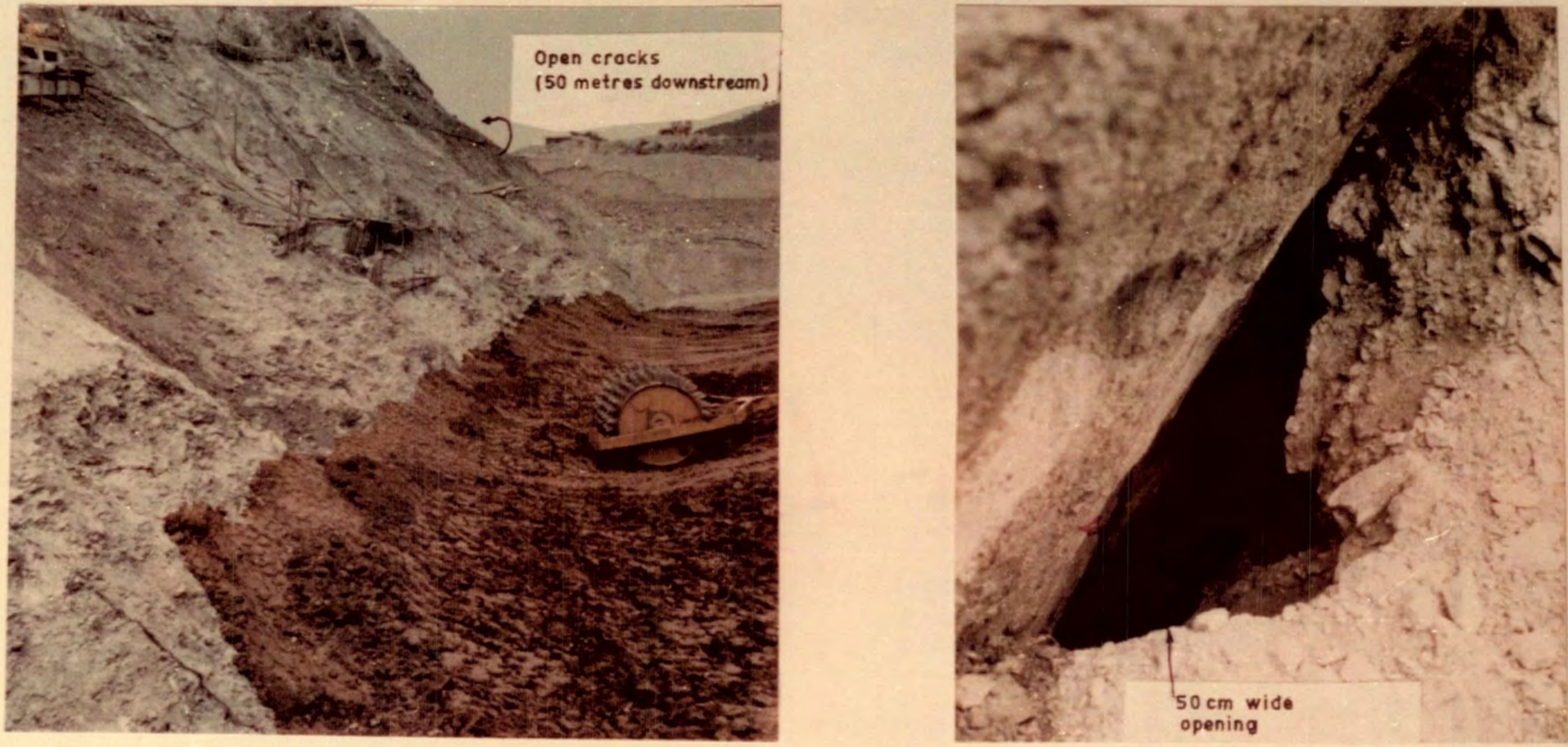


Plate 4.9 Clay core placement on the main fault zone of the left abutment and wide open crack downstream (Assomata dam)

POLAR NET
(Upper Hemisphere Projection)

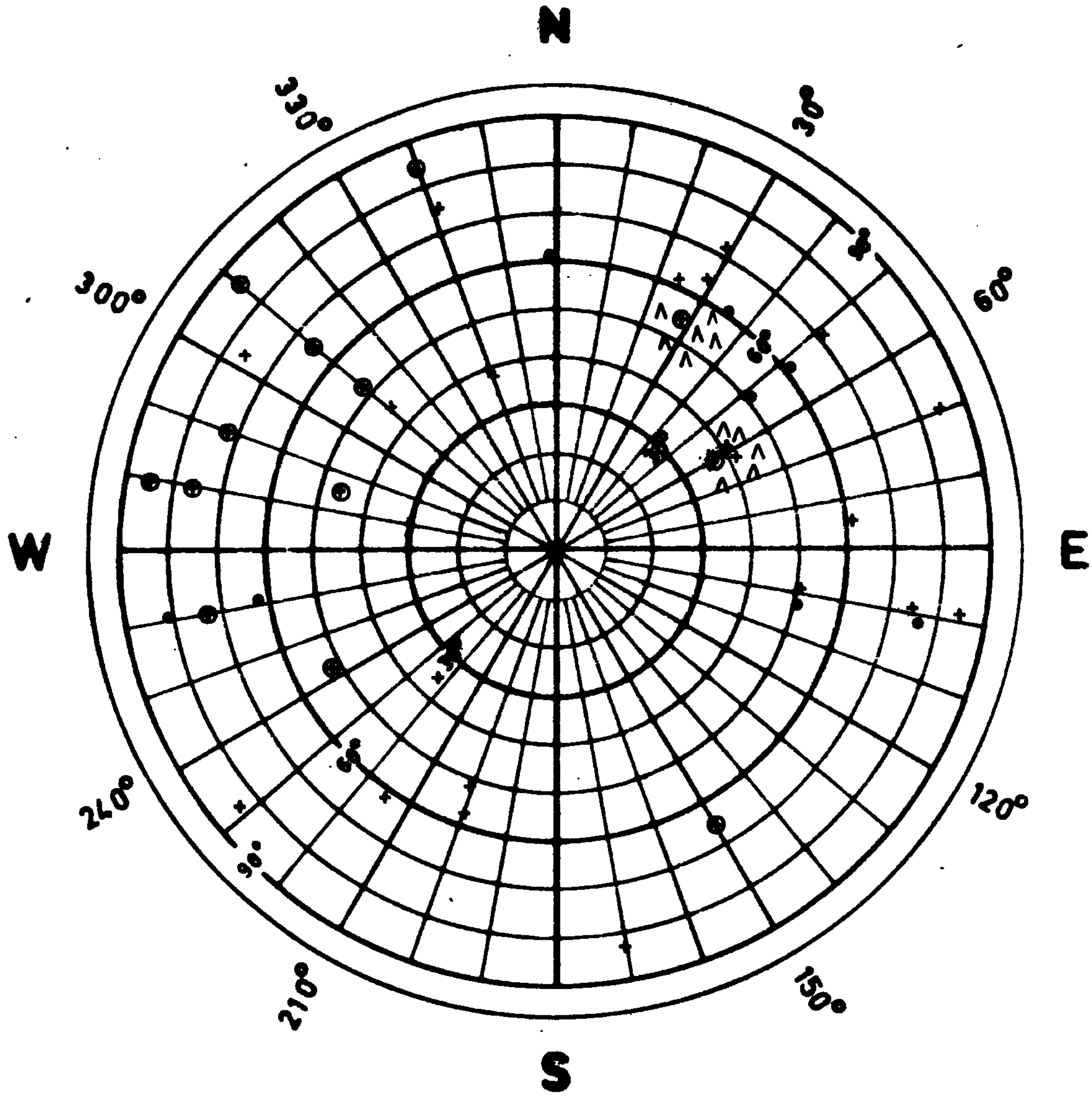


Fig. 4.19 *Discontinuity measurements along dam axis (Left abutment of Assomata dam).*

<u>*LEGEND*</u>

- • *Joints (Major)*
- + *Minor faults*
- ⊕ *Major faults (traversing the core trench).*
- ∧ *Foliation along sear zones*

POLAR NET
(Upper Hemisphere Projection)

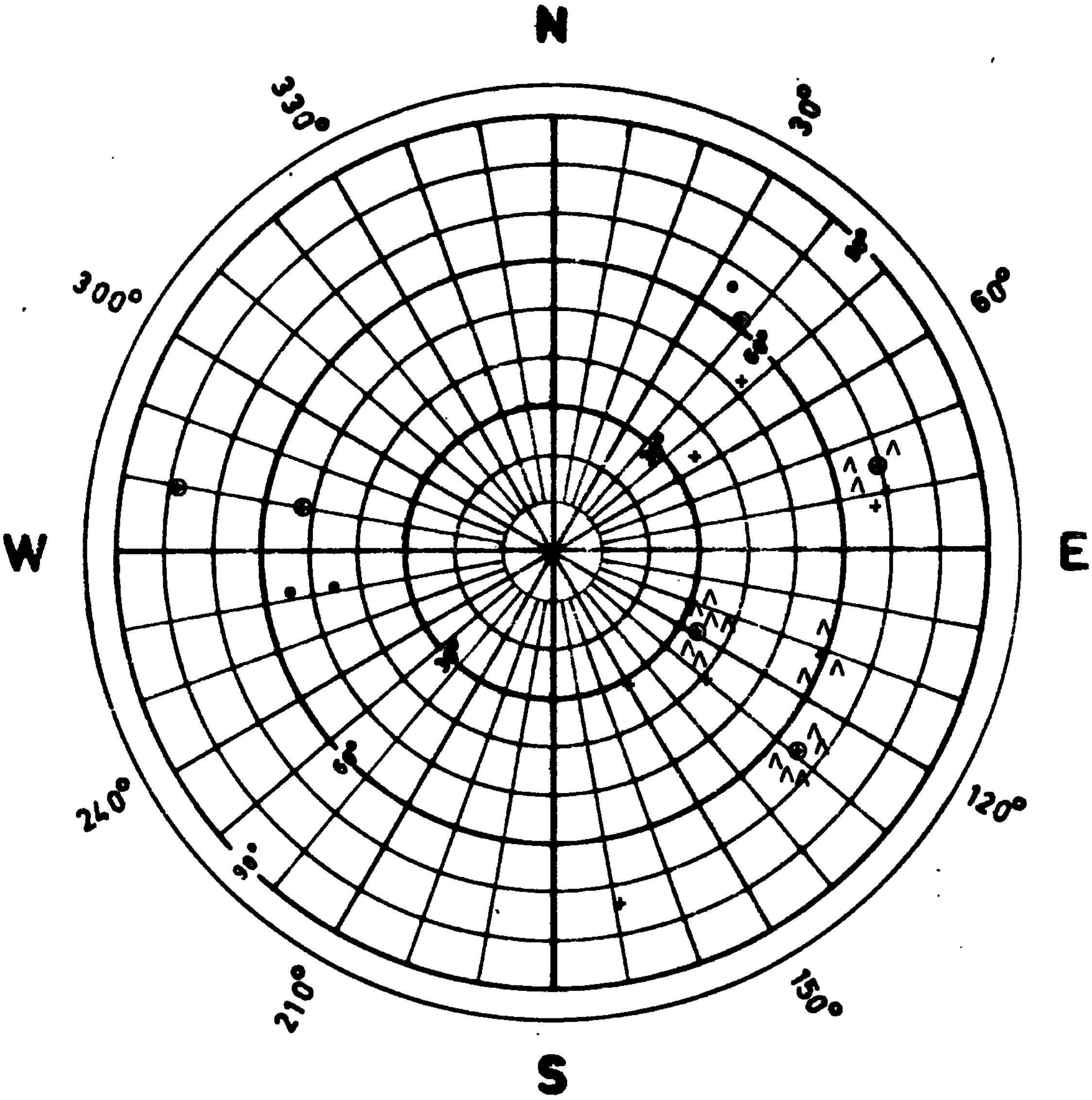


Fig. 4.20 *Discontinuity measurements along dam axis (Riverbed of Assomata dam).*

LEGEND

- • Joints (Major)
- + Minor faults
- ⊕ Major faults (traversing the core trench).
- ∧ Foliation along sear zones

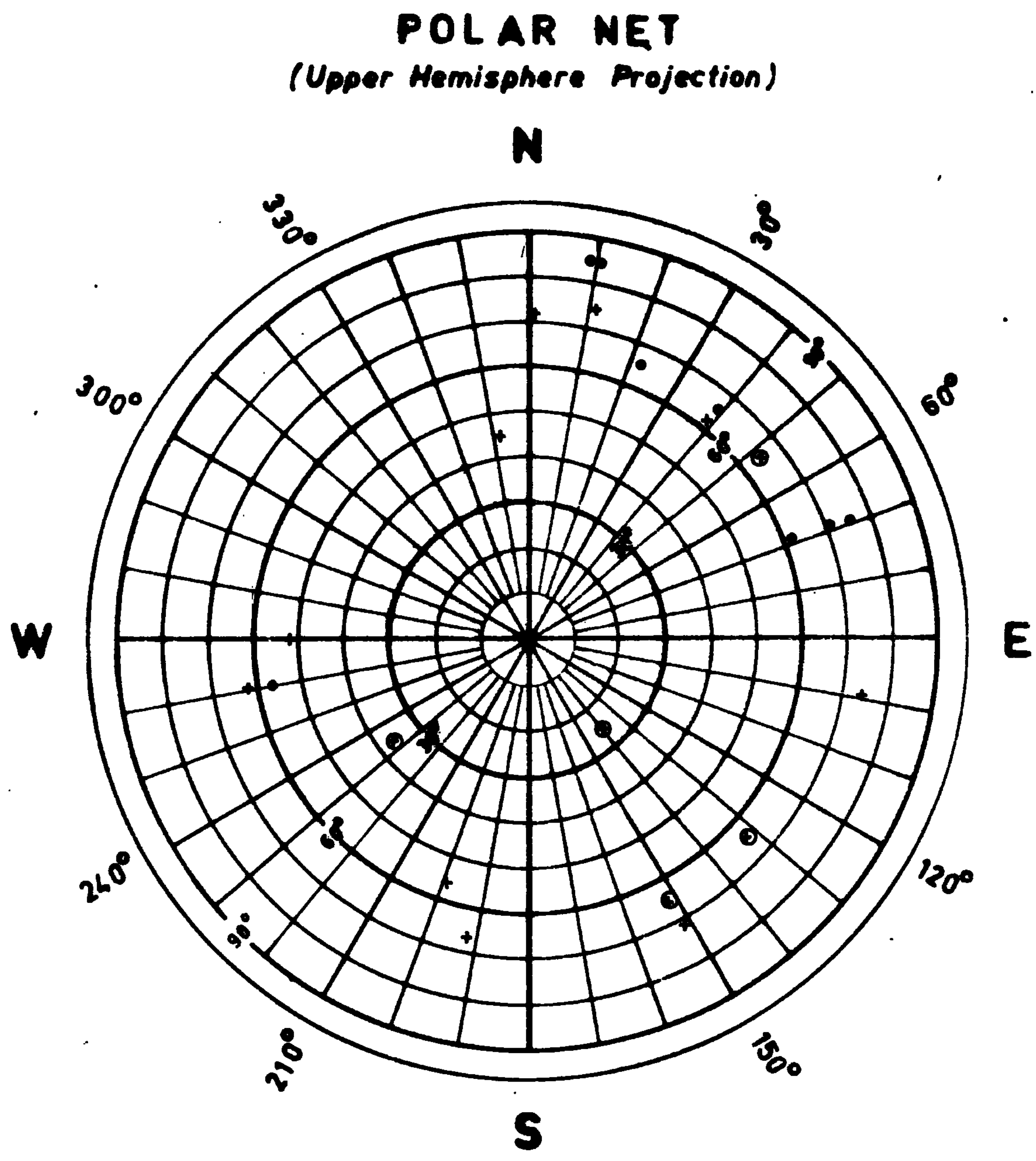


Fig. 4.21 *Discontinuity measurements along dam axis (Right abutment of Assomata dam).*

LEGEND
- • Joints (Major)
- + Minor faults
- ⊕ Major faults (traversing the core trench).

These three thrust faults are compatible with the regional geotectonic structure of the area, their dip directions being north-eastwards and their dips about 25^{o}-45^{o}.

It must be pointed out that characteristics such as the thickness of sheared or infilling material within faults or shears, the dip and dip direction, as well as the tightness of the fault and the disturbance of adjacent rocks, can be materially different within a few metres along their trace. It is noted also that almost all major shears and faults traverse the full width of the core trench.

E. Weathering characteristics

Rocks at the Assomata site exhibit deep penetrative weathering, particularly along the main structural features. Chemical and hydrothermal alterations have formed clayey and soil-like residual products. Minor alteration products are observed for the fault breccias of the tuff rocks.

Asbestos, chlorites, talc and other minerals have been observed along joints, shears and faults as alteration products of serpentinites. There is also a secondary deposition which has refilled open joints with calcite veins from millimetres to several centimetres in thickness. This phenomenon is frequently observed in the site. The whole appearance of such features suggests a high degree of relaxation as a result of geotectonic movements and associated hydrothermal action, particularly for the left abutment rocks (see Chapter 3,Section 3.2.4).
Chemical alteration and fracturing of the rocks, particularly of the left abutment, gave the low strength characteristics of the

materials tested. Clayey infillings exhibited φ_r' values ranging from 12° to 22°. Tested core specimens exhibited unconfined compressive strengths in the range

11 kg/cm^2 (1 078.7 kN/m^2) for sheared serpentinites,

152-230 kg/cm^2 (14 906.2-2 255.4 kN/m^2) for relatively sound serpentinites, and

1 110 kg/cm^2 (108 854.37 kN/m^2) for high quality tuff agglomerate rocks.

The most characteristic feature noted during the failure of the specimens was that in the serpentinites the pre-existing micro-fracturing and alteration (caused by regional tectonic movements) were acting to reduce the brittleness, so that failure took a more ductile form.

F. Ground water characteristics

Water tables, as indicated by the installed piezometres at the site, are illustrated in Fig. 4.17 and suggest a very small annual fluctuation of about 1-2 metres. The most characteristic features of these measurements are perched water tables in both abutments and artesian water-tables detected by deep holes in the river bed. At higher elevations up to the top of the hills surrounding the dam site, surface springs have been noticed in the alignment of fault lines. These suggested that certain rock bands or faults and shear zones act as natural barriers to the water flow channels in the bedrock. In this case, the permeabilities of the damsite bedrock must be interpreted as channelled ones.

4.3.3 Permeability characteristics

Foundation bedrock permeabilities were investigated with packer pressure tests using clear water and carried out from boreholes drilled along the dam axis in the river bed and the abutments. The results are presented in Figures 4.22, 4.23, 4.24 and 4.25.

The main variable interactions that could be examined were:

Permeability versus weathering

Permeability versus rock types encountered

Permeability versus RQD, and

Permeability versus depth (from ground surfaces).

The tests were executed at five-metre intervals at allowable pressures not exceeding the overburden pressure, and for very deep holes not exceeding a pressure of 15 Kg/cm^2 (1 471 kN/m^2). Each test is symbolized to represent the main lithological units encountered so as to provide an indication of the interrelations between rock types, weathering and fracture characteristics in terms of RQD and position in relation to depth.

A first examination of the permeability test data presented in Figs. 4.22 to 4.25 inclusive indicates that foundation bedrock in the Assomata damsite is quite permeable. Almost 60 per cent of the tests carried out exhibited permeability values higher than 1 Lugeon = 1.3×10^{-7} m/sec and thus an improvement (by grouting) is required (Houlsby, 1982).

Only three tests gave no water takes (zero tests) and if that is compared with the Pournari permeability tests (almost 50 per cent zero tests; see Section 4.2.3), it is obvious that the Assomata dam foundation will exhibit different grouting behaviour (higher grout takes most probable) from those in the Pournari dam foundation (see Sections 4.2.4 and 4.2.5).

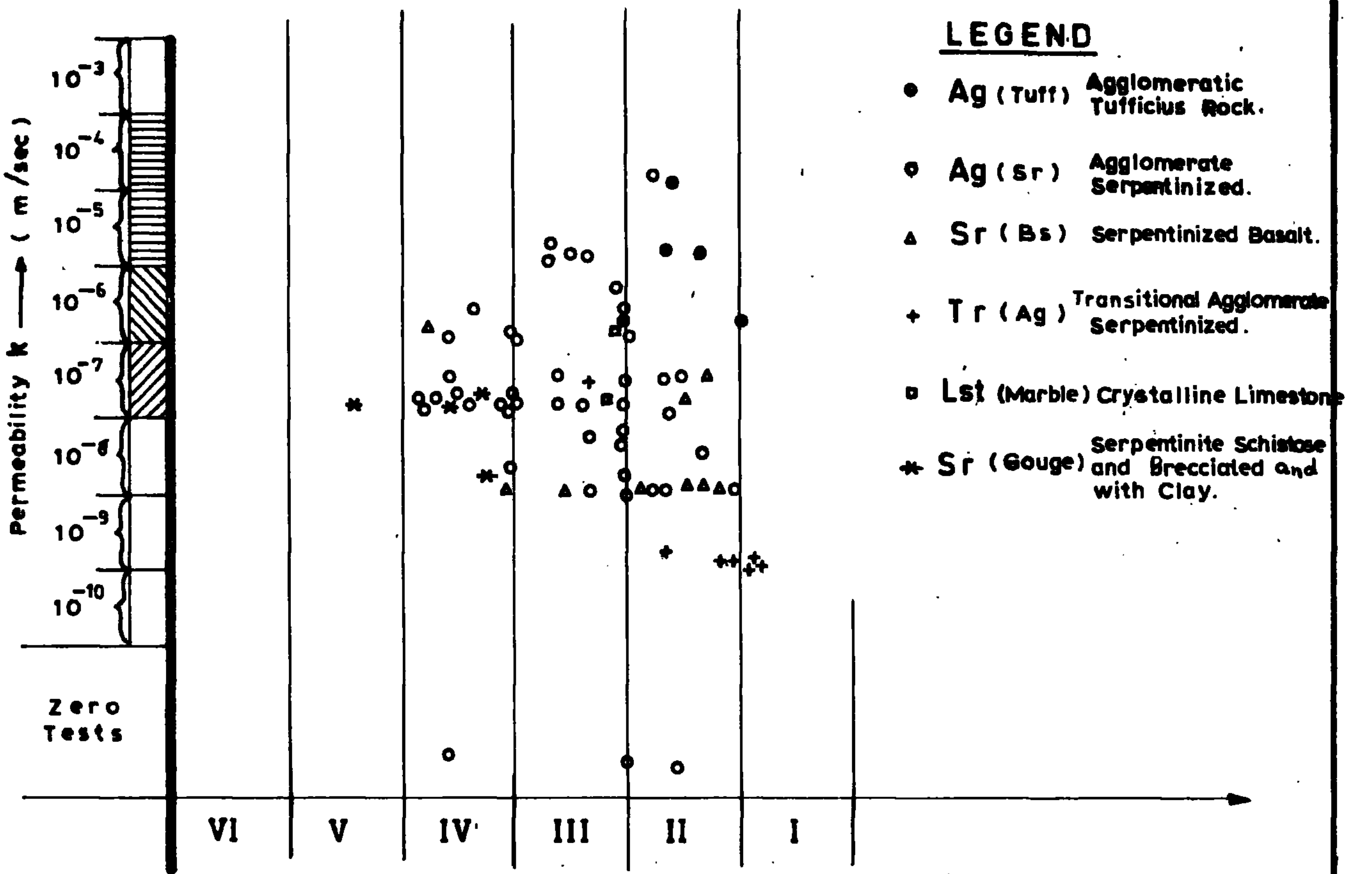


Fig. 4·22 Permeability vs weathering

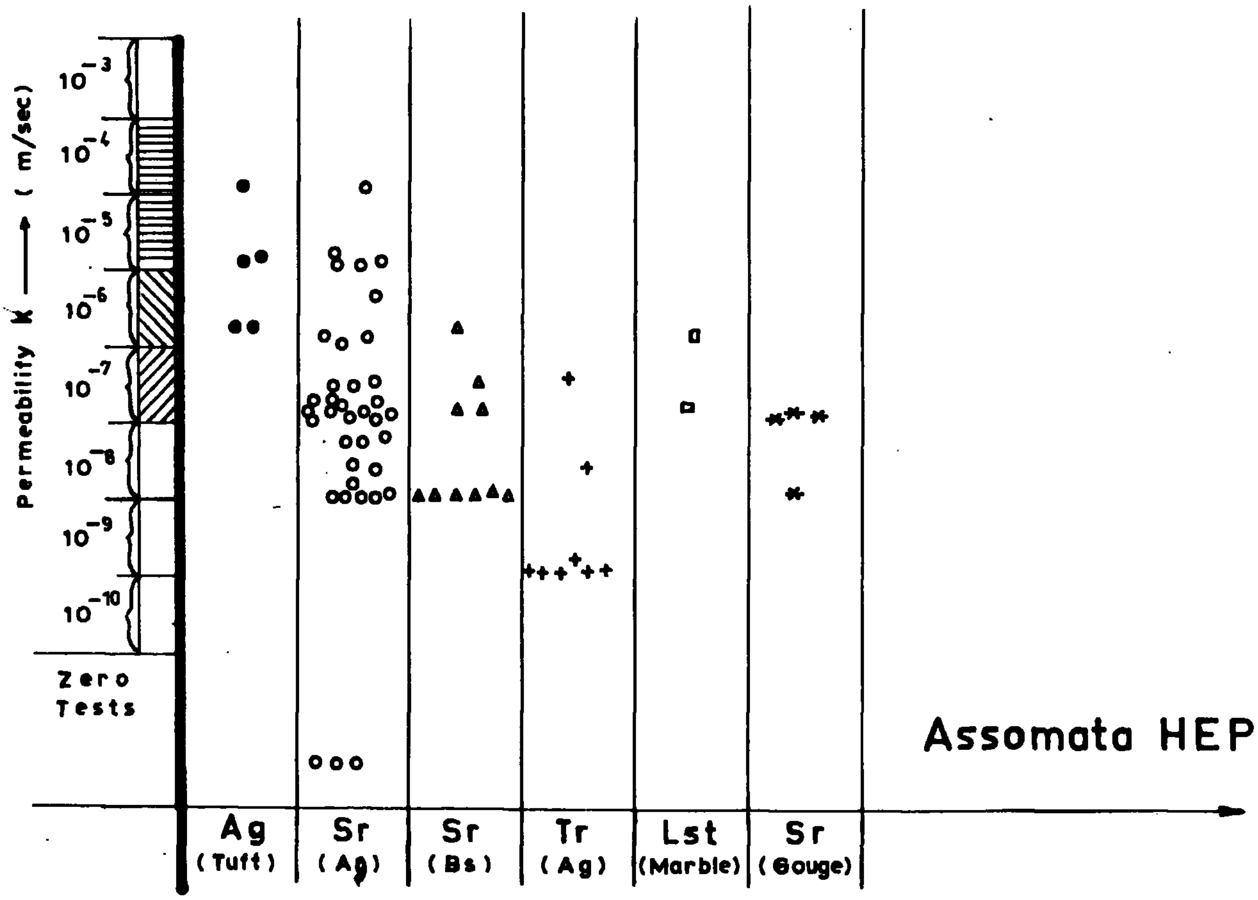


Fig. 4.23 Permeability vs rock types encountered

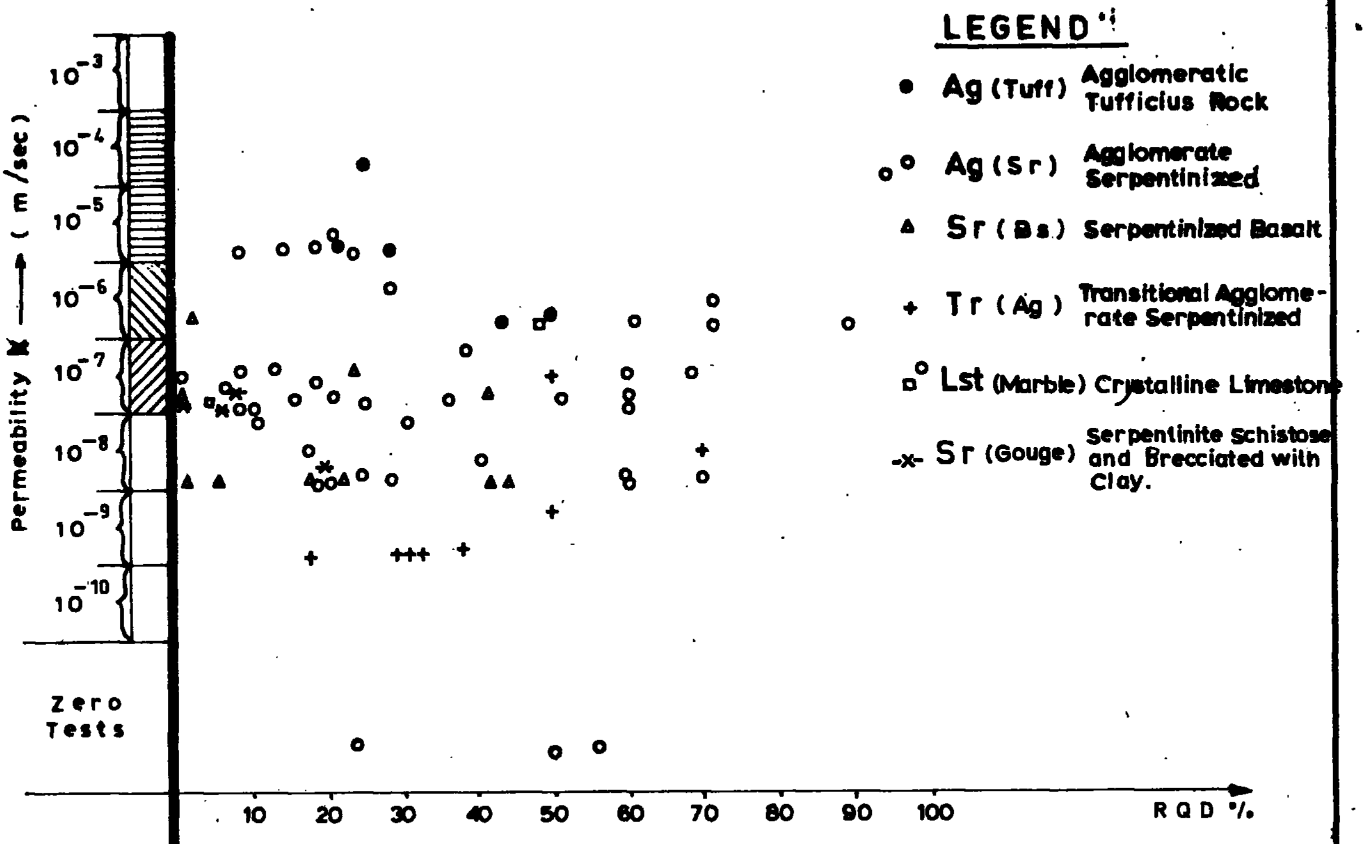


Fig.4.24 Permeability vs RQD

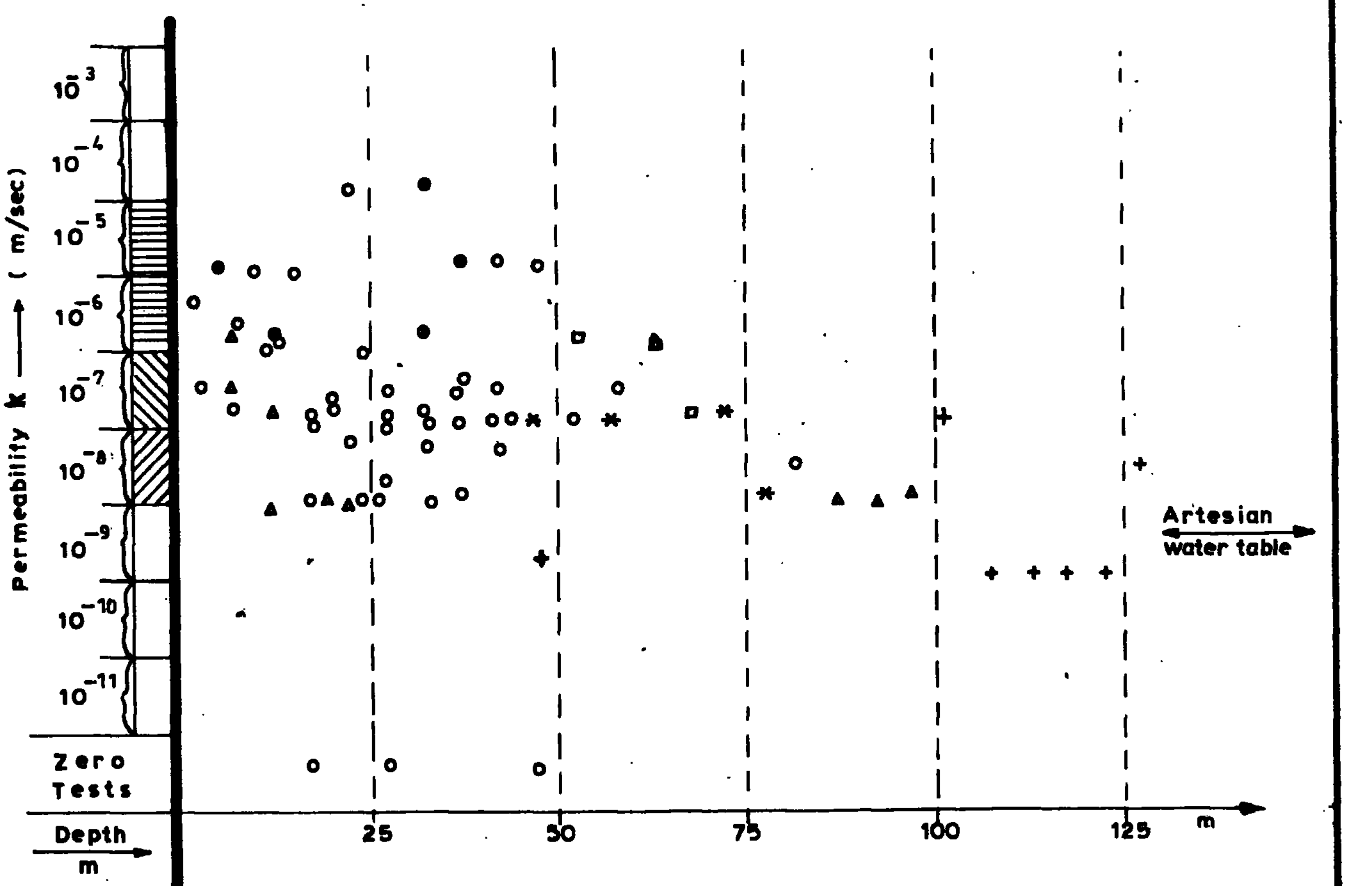


Fig. 4.25 Permeability vs depth

<u>The main conclusions</u> which can be drawn from examining the permeabilities recorded in relation to rock type, weathering, RQD and depth are as follows.

Permeabilities are higher in the tuff and the serpentinite agglomerates (Fig.4.23). The lower permeabilities recorded are those of the transitional agglomerates in which the artesian water tables were detected (<u>see</u> Fig. 4.17). The three zero tests must be attributed to the massive nature of the rocks (in which they were carried out) and also to the clayey infilling of their joints.

In Figure 4.22 an interesting parabolic envelope can be traced around the recorded tests.

The apex of this envelope is formed from the most weathered rock intervals tested. The following remarks seem appropriate for that particular arrangement of the permeability values in Fig. 4.22.

i) Open cracks have to be expected in the strong massive tuff and serpentinitic agglomerates which tend to appear infilled in the most weathered and altered rock formations.

ii) Small or tight cracks with low permeability values are observed in the most sound and massive rocks.The low permeability difference (one order of magnitude) of the tight transitional zone strata and the zoned low permeability values of all the other rocks has to be considered in the construction of the grout curtain.

iii) There is an obvious trend towards a reduction of permeability values with depth (Fig. 4.25).

Schistose (transitional) and sheared rocks (fault zones) seem to be tighter than the other most permeable rocks. In the case of the over-thrusts, the overlying rocks are more disturbed.

In Fig. 4.24 the results show that among the stronger tuff and serpentinitic agglomerates, those rocks which exhibit lower RQD values are most permeable.

Relatively high permeability values are exhibited also in higher RQD values, and in this case the explanation is that open isolated joints must exist.

There are several factors emerging from the permeability test results which can explain the transmissibility behaviour of the foundation bedrock.

There is a particular water-tight horizon which is the serpentinitic transitional agglomerate. Also, there are the thick sheared and schistose fault zones which exhibit relatively low permeability values. The artesian water table and the perched water tables (Fig. 4.17), as well as the permeability results cited above, show a channelled transmissibility behaviour in the foundation bedrock.

The most permeable rocks are the tuff agglomerate rocks, which are the younger formations. They are related to the stronger faults at the site and are extruded through these faults (see also Fig. 4.18).

Their inherent cracking as a result of rapid cooling has been worsened by subsequent geotectonic movements. Open cracks, observed after the core trench excavations (particularly within the stronger rock formations), vertical faults and joints, as well as the high alteration in the older formations, give a picture of the complex geotectonic history of the site (see Section 3.2.4 in Chapter 3).

4.3.4 Grouting at Assomata

(a) General

Grouting at Assomata is needed for three main reasons:

1) to reduce leakages under the dam and from the abutments (main grout curtain),
2) to strengthen (by consolidation grouting, see Plates 4.10 and 4.11) the foundation rock under the core (and secondarily to assist the performance of the grout curtain by sealing off all discontinuities traversing the core trench and to eliminate any danger of erosion of soft foundation rock or core material), and
3) to reduce any uplift pressures into the abutments.

In this third case, in addition to the construction of a drainage curtain immediately downstream of the dam axis, the following provisions have been made:

a) Drainage holes ("relief wells"),eighteen in total of 8 inches diameter, filled with coarse sand and pea gravel, have been drilled in the foundation bedrock under the downstream shell along the major fault line of the left abutment.
b) In the lower of the grouting galleries of the left abutment, and in the margins of the main fault, a "T"-shaped end of the grouting gallery has been built to be used for additional grouting if it is needed, and

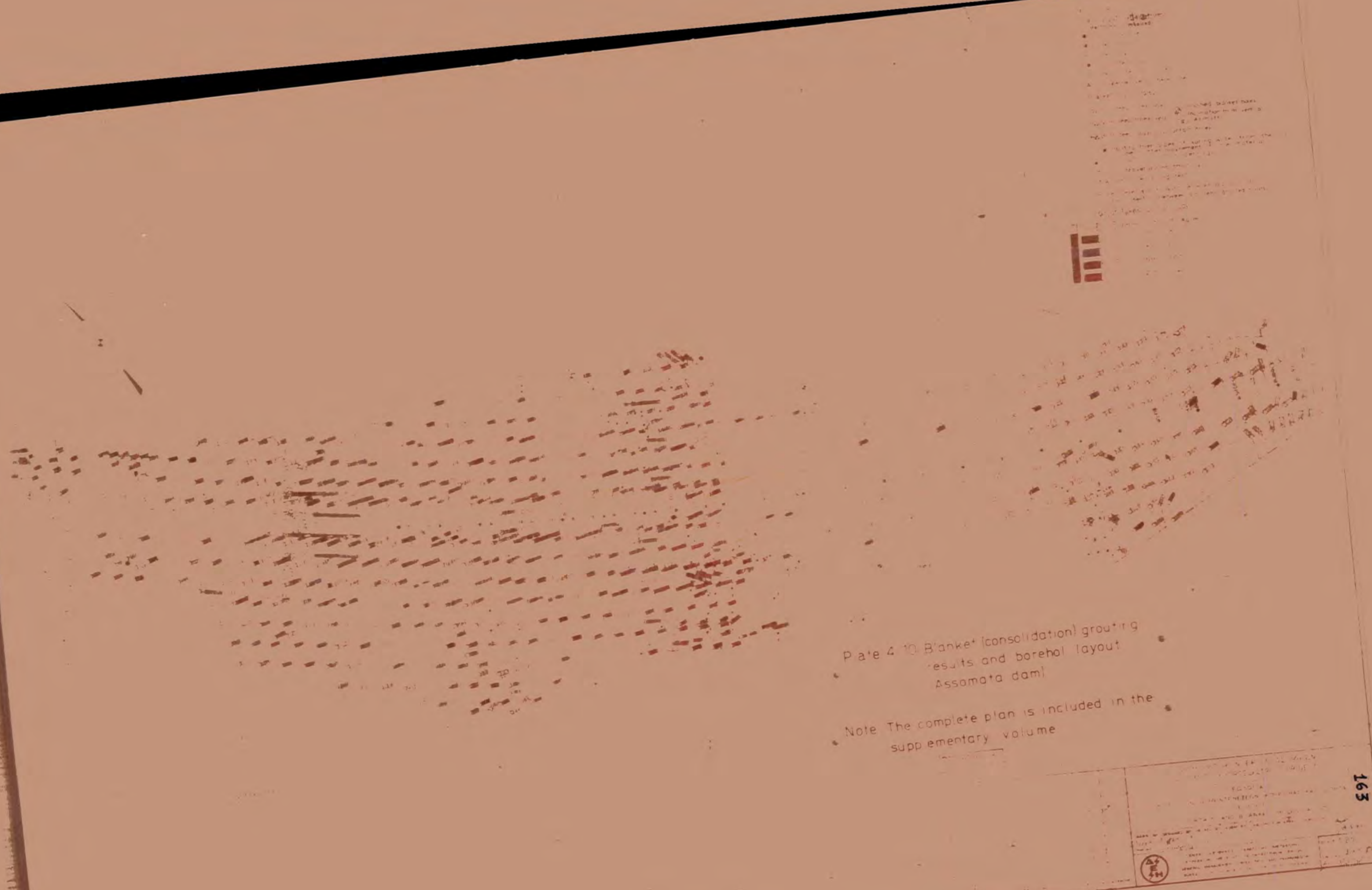


Plate 4.10 Blanket (consolidation) grouting results and borehol layout (Assomata dam)

Note: The complete plan is included in the supplementary volume

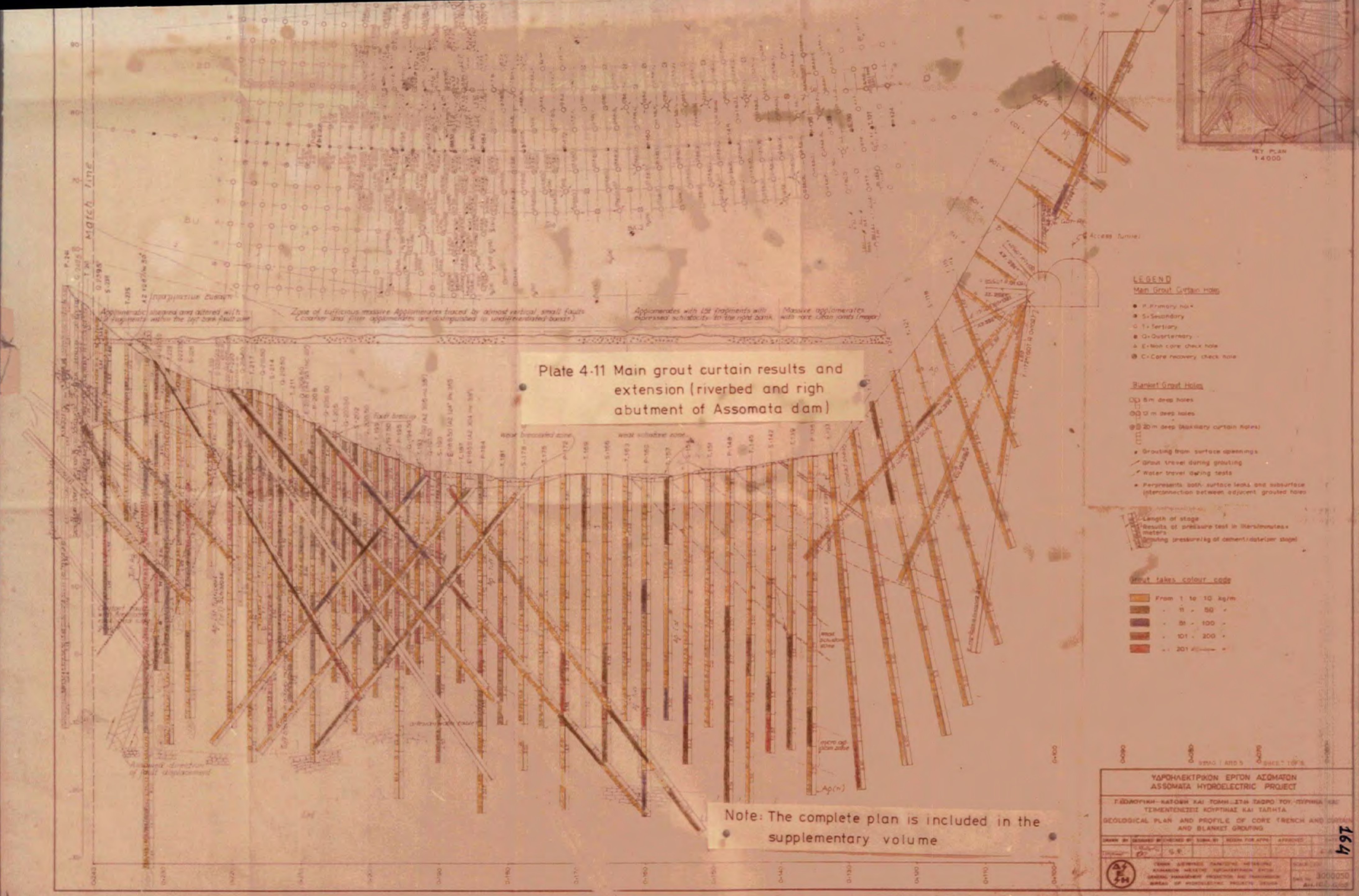


Plate 4.11 Main grout curtain results and extension (riverbed and righ abutment of Assomata dam)

Note: The complete plan is included in the supplementary volume

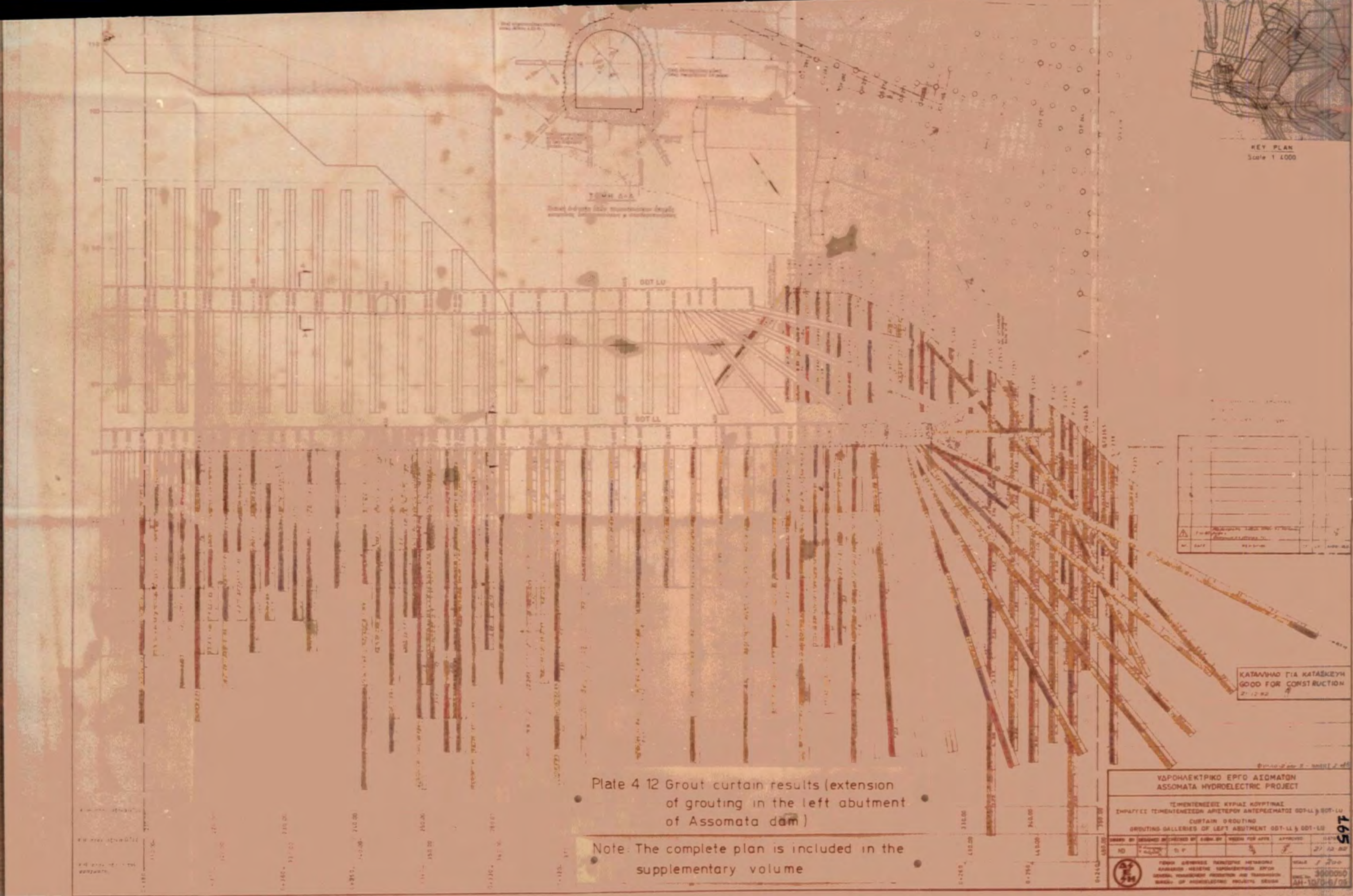


Plate 4 12 Grout curtain results (extension of grouting in the left abutment of Assomata dam)

Note: The complete plan is included in the supplementary volume

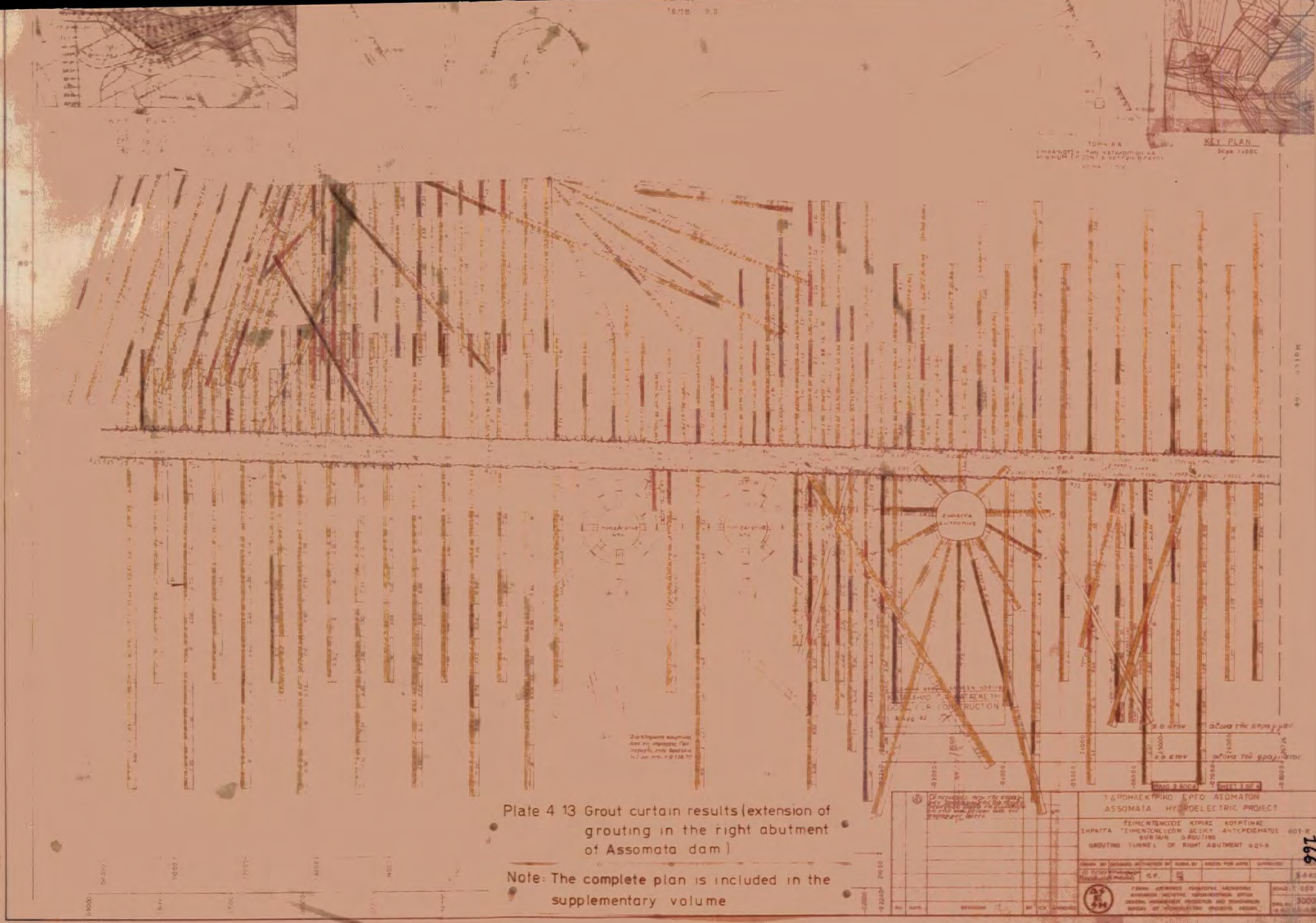


Plate 4 13 Grout curtain results (extension of grouting in the right abutment of Assomata dam)

Note: The complete plan is included in the supplementary volume

TABLE 4·9

Main Grout Curtain Results

	Segment N° / Grouted surface (m²)	1	2	3	4	5	6	7	8	9	10	11	12	13	14
	Grouted surface (m²)	660	3050	3050	2905	2894	2395	2825	2515	2218	4560	4100	3950	3500	1850
	Grout Take per Segment (kg)				146407	68198	83825	267204	17457	5079	44788	25213	31028	4416	
	Grouting B. H Length (m)				892	684	1234	1638	783	716	1426	980	1331	753	
	Density of B. H (m/m²)						0.515	0.58	0.311	0.323	0.313	0.239	0336		
	Grout Take per m-run of B. H. (kg/m)						67.93	163.13	22.89	7.09	31.41	25.73	23.31		
	Grout Take per Area of Segment (kg/m²)						35.00	94.58	6.94	2.29	9.82	6.15	7.86		
	Grout Take per m-run per Area of Segment (kg/m³)						18030 X 10^{-3}	54850 X 10^{-3}	2160 X 10^{-3}	740 X 10^{-3}	3670 X 10^{-3}	1470 X 10^{-3}	2640 X 10^{-3}		
	Volume of grout used per m run per Area of Segment. (m³/m³ = unit)						24000 X 10^{-3}	73040 X 10^{-3}	2870 X 10^{-3}	980 X 10^{-3}	4090 X 10^{-3}	1960 X 10^{-3}	351 X 10^{-3}		
P	Grout Take (kg)				61025	49687	26899	100789	7328	290	9380	13282	18604	3139	
	Length of B.H. (m)				240	334	340	285	200	144	456	341	590	408	
S	Grout Take (kg)				56273	13111	16667	26770	6030	1811	11096	7161	9280	1277	
	Length of B.H. (m)				197	220	327	203	172	151	375	271	402	345	
T	Grout Take (kg)				28175	4780	33124	66459	3401	2297	7364	4433	2287		
	Length of B.H. (m)				340	113	320	356	311	266	324	317	235		
Q	Grout Take (kg)				934	620	5898	60094	Ø	Ø	11917	47	Ø		
	Length of B.H. (m)				115	17	127	377	Ø	Ø	138	8	Ø		
E and C	Grout Take (kg)						1237	13092	698	681	5031	290	857		
	Length of B.H. (m)						120	417	100	155	133	43	104		

TABLE 4 : 10

Main Grout Curtain Results of Primary (P) Boreholes

Segment N°	1	2	3	4	5	6	7	8	9	10	11	12	13	14	
Grouted Surface Area (m^2)	660	3050	3050	2905	2894	2395	2825	2515	2218	4560	4100	3950	3500	1850	
Grout Take (kg cement)						26899	100789	7328	290	9380	13282	18604			
Grouting B.H. Length (m)						340	285	200	144	456	341	590			
Density of B.H. (m/m^2)						0.14	0.10	0.08	0.06	0.10	0.08	0.15			
Grout Take per m-run of B.H. (kg/m)						79.11	353.64	36.64	2.01	20.57	38.95	31.53			
Grout Take per Area of Segment (kg/m^2)						11.23	35.68	2.91	0.13	2.06	3.24	4.71			
Grout Take per m-run per Area of Segment. (kg/m^3)						1570×10^{-3}	3570×10^{-3}	233×10^{-3}	8×10^{-3}	206×10^{-3}	260×10^{-3}	706×10^{-3}			
Volume of grout used per m-run per Area of Segment. (m^3/m^3= unit)						2090×10^{-3}	4750×10^{-3}	310×10^{-3}	100×10^{-3}	270×10^{-3}	350×10^{-3}	940×10^{-3}			

TABLE 4·11

Main Grout Curtain Results of Secondary (S) Boreholes

Segment N°	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Grouted Surface Area (m^2)	660	3050	3050	2905	2894	2395	2825	2515	2218	4560	4100	3950	3500	1850
Grout Take (kg cement)						16667	26770	6030	1811	11096	7161	9280		
Grouting B.H. Length (m)						327	203	172	151	375	271	402		
Density of B.H. (m/m^2)						0.136	0.07	0.07	0.07	0.08	0.066	0.10		
Grout Take per m-run of B.H. (kg/m)						50.97	131.87	2.39	11.99	29.56	26.42	23.08		
Grout Take per Area of Segment (kg/m^2)						6.96	9.47	2.40	0.82	2.43	1.75	2.35		
Grout Take per m-run per Area of Segment. (kg/m^3)						940×10^{-3}	660×10^{-3}	170×10^{-3}	57×10^{-3}	190×10^{-3}	115×10^{-3}	135×10^{-3}		
Volume of grout used per m-run per Area of Segment. (m^3/m^2 unit)						1260×10^{-3}	880×10^{-3}	220×10^{-3}	76×10^{-3}	260×10^{-3}	150×10^{-3}	310×10^{-3}		

TABLE 4·12

Main Grout Curtain Results of Tertiary (T) Boreholes

Segment N°	1	2	3	4	5	6	7	8	9	10	11	12	13	14	
Grouted Surface Area (m^2)	660	3050	3050	2905	2894	2395	2825	2515	2218	4560	4100	3950	3500	1850	
Grout Take (kg cement)						33124	66459	3401	2297	7364	4433	2287			
Grouting B.H. Length (m)						320	356	311	266	324	317	235			
Density of B.H. (m/m^2)						0.133	0.126	0.124	0.112	0.071	0.077	0.059			
Grout Take per m-run of B.H. (kg/m)						103.51	186.68	10.93	8.64	22.73	13.98	9.73			
Grout Take per Area of Segment (kg/m^2)						13.83	23.52	1.35	1.03	1.61	1.08	0.58			
Grout Take per m-run per Area of Segment. (kg/m^3)						1840×10^{-3}	2960×10^{-3}	170×10^{-3}	116×10^{-3}	115×10^{-3}	80×10^{-3}	34×10^{-3}			
Volume of grout used per m-run per Area of Segment. (m^3/m^3 unit)						2450×10^{-3}	3950×10^{-3}	220×10^{-3}	150×10^{-3}	150×10^{-3}	110×10^{-3}	45×10^{-3}			

TABLE 4·13

Main Grout Curtain Results of Queternary (Q) Boreholes

Segment N°	1	2	3	4	5	6	7	8	9	10	11	12	13	14	
Grouted Surface Area (m^2)	660	3050	3050	2905	2894	2395	2825	2515	2218	4560	4100	3950	3500	1850	
Grout Take (kg cement)						5898	60094	Ø	Ø	11917	47	Ø			
Grouting B.H. Length (m)						127	377	Ø	Ø	138	8	Ø			
Density of B.H. (m/m^2)						0.053	0.133	Ø	Ø	0.03	0.0019	Ø			
Grout Take per m-run of B.H. (kg/m)						46.44	159.40	Ø	Ø	86.36	5.87	Ø			
Grout Take per Area of Segment (kg/m^2)						2.46	21.27	Ø	Ø	2.61	0.011	Ø			
Grout Take per m-run per Area of Segment. (kg/m^3)						130×10^{-3}	2830×10^{-3}	Ø	Ø	78×10^{-3}	0.022×10^{-3}	Ø			
Volume of grout used per m-run per Area of Segment. (m^3/m^2 = unit)						170×10^{-3}	3760×10^{-3}	Ø	Ø	104×10^{-3}	0.029×10^{-3}	Ø			

TABLE 4·14

Main Grout Curtain Results of Check (E and C) Boreholes

Segment N°	1	2	3	4	5	6	7	8	9	10	11	12	13	14	
Grouted Surface Area (m^2)	660	3050	3050	2905	2894	2395	2825	2515	2218	4560	4100	3950	3500	1850	
Grout Take (kg cement)						1237	13092	698	681	5031	290	857			
Grouting B.H. Length (m)						120	417	100	155	133	43	104			
Density of B.H. (m/m^2)						0.05	0.147	0.04	0.07	0.029	0.01	0.026			
Grout Take per m-run of B.H. (kg/m)						10.31	31.36	6.98	4.39	37.83	6.74	8.24			
Grout Take per Area of Segment (kg/m^2)						0.516	4.63	0.277	0.307	1.10	0.071	0.217			
Grout Take per m-run per Area of Segment. (kg/m^3)						25×10^{-3}	680×10^{-3}	11×10^{-3}	21×10^{-3}	32×10^{-3}	71×10^{-3}	5.64×10^{-3}			
Volume of grout used per m-run per Area of Segment. (m^3/m^3 = unit)						34×10^{-3}	910×10^{-3}	15×10^{-3}	28×10^{-3}	43×10^{-3}	0.942×10^{-3}	7.51×10^{-3}			

for drainage purposes (<u>see</u> Plate 4.12).

(b) Grouting results

Grouting results presented here concern the main grout curtain for the completed segments, while mention of the blanket (consolidation) grouting results will be made where necessary to denote important geological factors conditioning the evaluation of the results.

The gouting parameters considered are the same as those defined in section 4.2.4 (b).

The detailed grouting field results are presented in the "as built drawings" which are given in the supplementary volume.

For evaluation purposes, the above results are gathered together in Table 4.9, where the results in the succession of Primary, Secondary, Tertiary, Quaternary and Check holes are given in Tables 4.10 to 4.14. The results are grouped in basic segments as at Pournari for comparison purposes.

The averaged results of grout takes in kg of cement per m^2, boreholes drilled in m-run per m^2, and the volume of grout used per m-run per area of segment (dimensionless parameter), as well as the treated area in m^2 of each segment, are illustrated in Fig. 4.26. In this Figure, differences such as those exhibited in segments 7 and 9, or similarities such as those exhibited in segments 11 and 12, facilitate the evaluation of the predominant geological factors involved.

The effectiveness of grouting treatment performed in the succession of P,S,T,Q, E and C grouting stages, are presented in Figure 4.27.

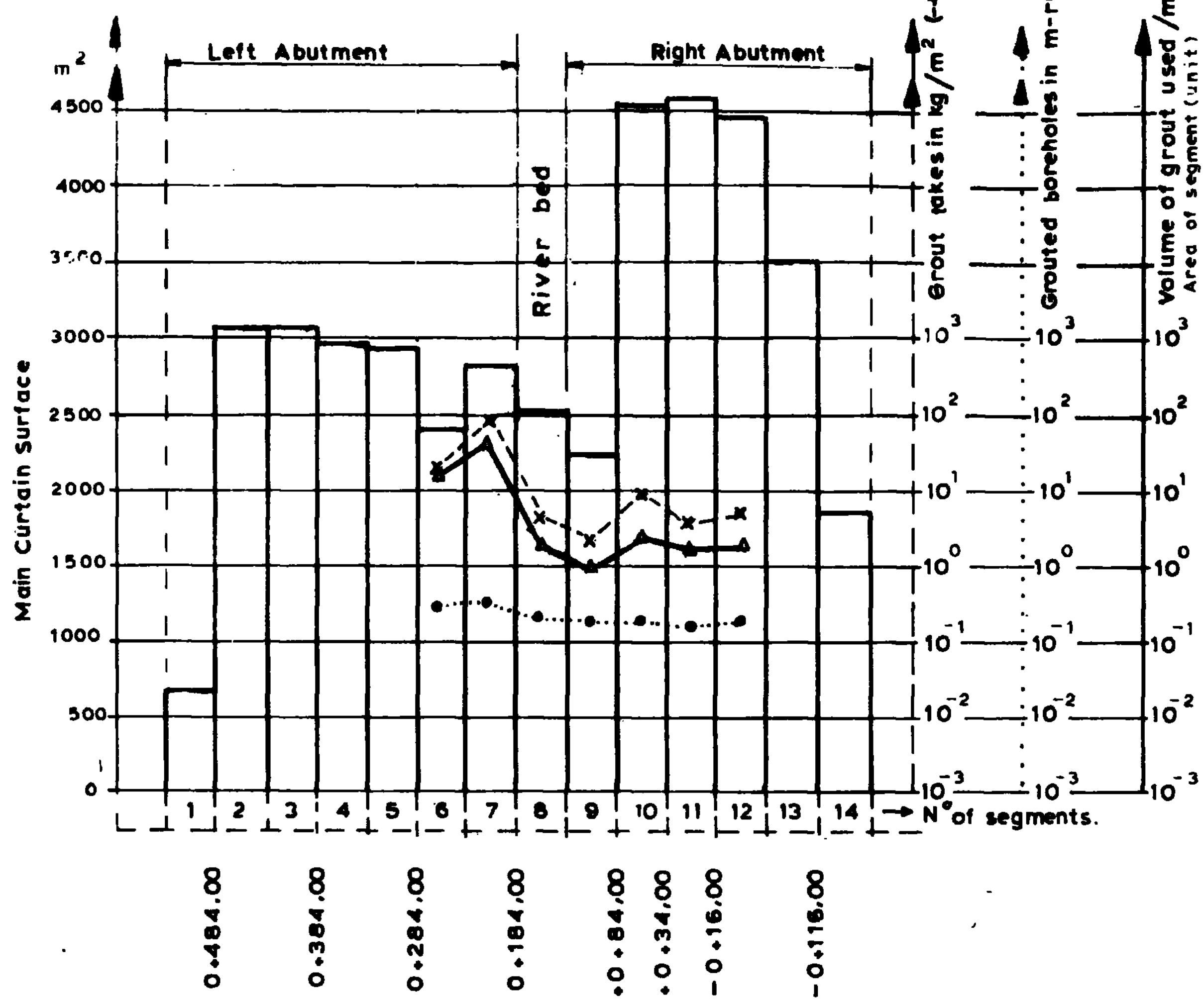


Fig : 4 . 26. *Foundation grouting treatment (Assomata dam) and behaviour in terms of:*

1. *Main grout curtain extension along dam axis (m^2)*
2. *Borehole density in m-run per m^2 of curtain.*
3. *Grout takes per m^2 of the curtain*
4. *Volume of grout / m-run of bh/area of segment (unit)*

Assomata HEP.

ASSOMATA HEP

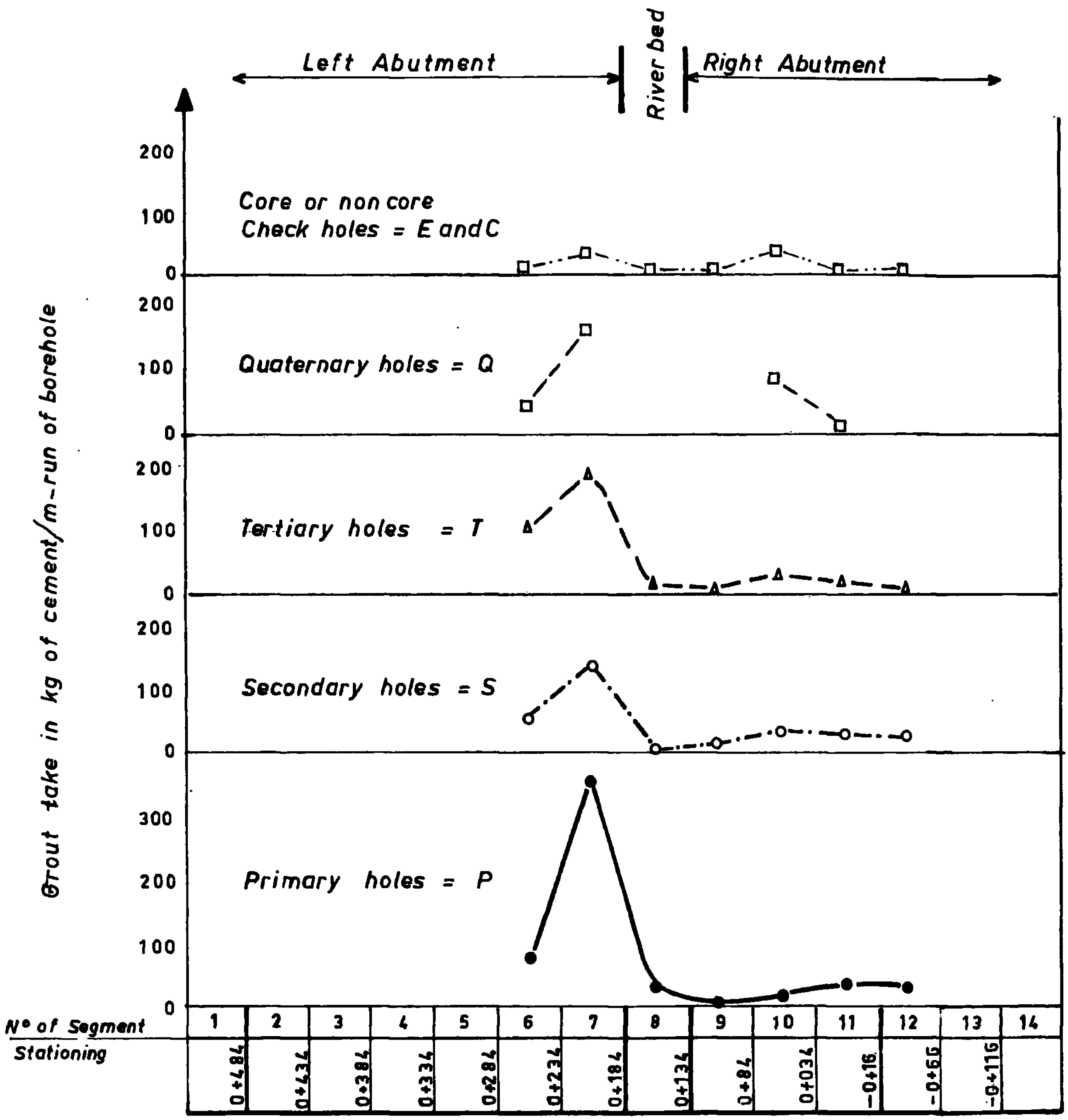
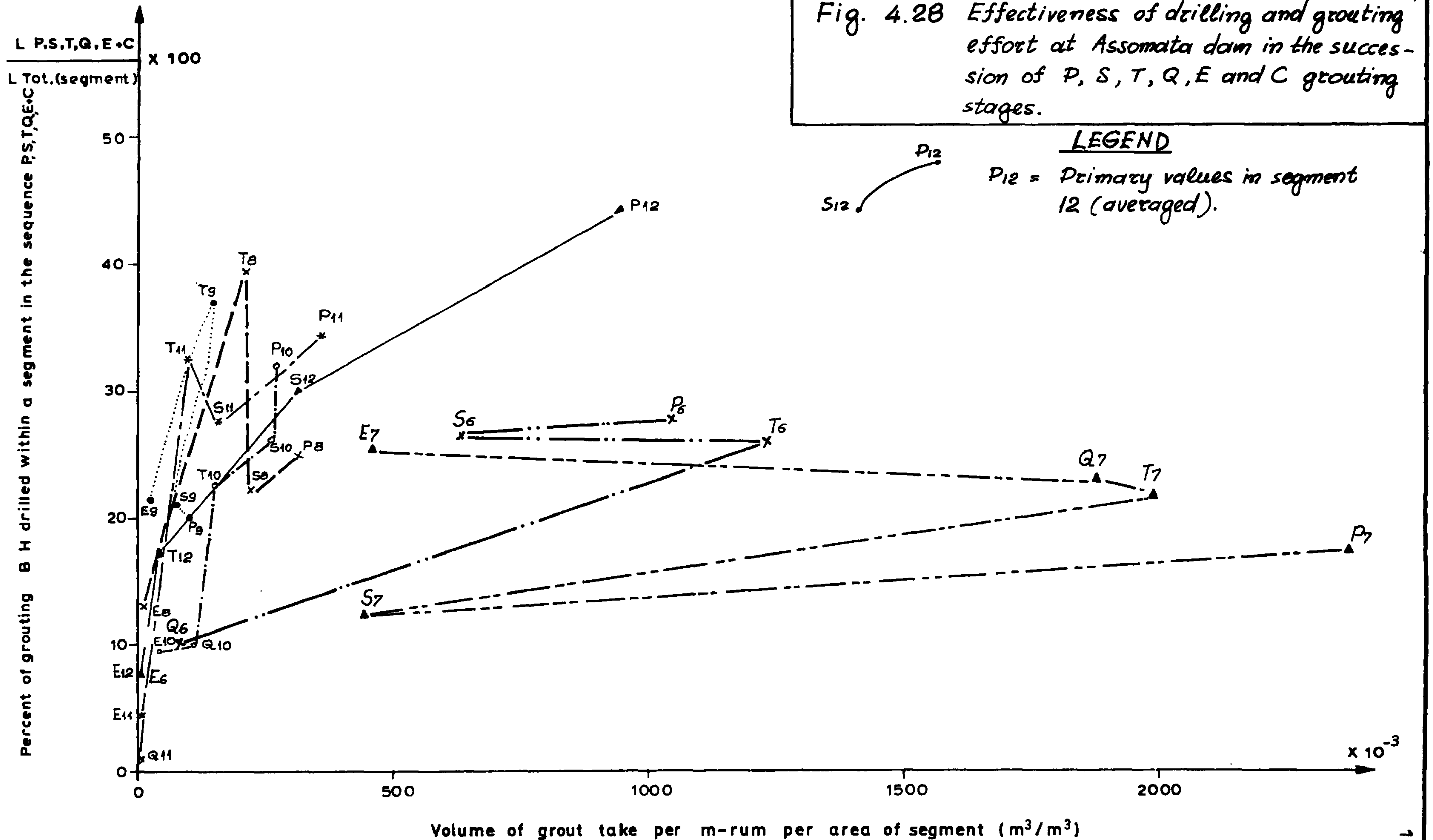


Fig. 4.27 *Comparative results of foundation bedrock behaviour in grouting treatment along the main Assomata dam grout curtain (Plots represent the average grout take, in Kg of cement per m-run of borehole for each segment considered).*

Fig. 4.28 Effectiveness of drilling and grouting effort at Assomata dam in the succession of P, S, T, Q, E and C grouting stages.



Assomata HEP

Differences, such as those of segments 7 and 10, are indicative of the depth and frequency of the most important groutable discontinuities and the lithological horizons cited in the foregoing paragraphs.

Finally, Figure 4.28 illustrates the effectiveness of the drilling and grouting effort in the succession of the P,S,T,Q stages and the checking stages E and C. The effectiveness achieved, as illustrated by the P,S,T,Q,E and C curves, acts as warnings and guidance towards the appropriate future measures needed to supplement the grouting works already performed. It is thus possible to isolate and specify the existing or assumed geological factors involved.

It is noted that it is often necessary to perform further grouting or drainage works after completion of the initial work.

4.3.5 Summary discussion and conclusions on Assomata dam

On the basis of the grouting results and the other field information available, as presented in this Section, a number of conclusions can be drawn, in spite of the complex nature of the variables considered, and the uncertainties which usually accompany the geological factors involved. In the evaluation of the grouting results, some correlations can be made between
a) the grouting parameters (quantities), and the foundation conditions (rock voidness), and b) the extent of the effectiveness achieved by the grouting works and the pre-set completion criteria (see Appendix C).

On the other hand, a number of ambiguities and the need for additional information from specific tests, geological records and observations are brought to light when an attempt is made a) to evaluate the foundation properties and their response to the methods of improvement applied, and b) to grade their interrelations. It was not possible to carry out these tests, but suggestions for further research will be detailed subsequently.

Curves (Fig. 4.26) of the average results of the grout used per unit surface (m^2) of the basic segmental divisions of the grout curtain indicate that the quantities of grout used are higher in segment 7 and lower in segment 9, while there are no considerable differences between segments 11 and 12. It can also be noted that the density of grout boreholes drilled per unit surface of the basic segments is about the same in segments 8, 9, 10, 11 and 12, while in segments 6 and 7 it is double that in segments 8, 9, 10, 11 and 12.

Consideration of the quantities of the grout used, and the density of the boreholes drilled in segments 6 and 7, shows that the effective openings of the treated foundation rock are higher in segment 7 and must be attributed to factors pertaining to this segment. These factors are:

a) The near-vertical faults recorded at the surface resulting in disturbance (voidness) of tuff agglomerates.

b) The different effect that those faults exert on the absorption properties of the different lithological units involved. Softer rocks exhibit more tolerance when stressed than do brittle rocks.

In segment 6, again, the higher grout absorptions can be attributed to similar discontinuities present at greater depth in a tuff agglomerate rock band. The major thrust fault zone and the thick (20-30m) character of the schistose serpentinites attached to this zone did not exhibit any substantial grout absorption (see also conclusions of Section 4.3.3) except for isolated intervals. But it can be argued that its presence resulted in higher relaxation of the adjacent stronger rocks.

It is worth noting also that in those two segments consolidation grouting was applied to the full width of the core trench by a net of blanket holes (6-20m deep). Voids present at the surface were filled by pouring in a thick cement grout. Some of these voids absorbed up to 6 tons of cement.

In segments 7 and 8, the horizon of the transitional zone (schistose serpentinitic agglomerates) proved to be of low permeability (see Section 4.3.3) and of low grout absorption. Its extension and depth were checked by core sampling in all Primary grout holes which were executed there. This assisted in locating an inclined horizon of tuff agglomerates which exhibited the higher grout absorptions within segment 6. Dislocations of the tuff rocks and other evidence suggests that there is an upward movement of a few metres) in the left abutment in relation to the riverbed.

The relatively high grout absorption noted in segment 10 is due to:

a) The presence of a fault and minor shears (several centimetres thick) and its relation to

b) The presence of the tuff agglomerates of the abrupt right abutment (elevation 70-80m ; see also Section 4.3.2A).

c) The proximity of the grout holes (above elevation 50m) to the surface of the abrupt cliff there, and

d) The proximity to the underground and surface constructions (power house, access tunnel, grouting gallery of the right abutment, access roads, and the intense blasting during construction).

Inherent jointing conditions have been aggravated by these construction works. Joints have opened (dilated) as a result of stress relaxation.

Higher grout absorptions recorded in Tertiary holes (see Fig. 4.27), as well as some high grout takes in check holes, are due to their proximity to the tuff agglomerates and the presence of a fault zone (see Plate 4.8 and "as built" drawings).

In Figure 4.28 it is evident that the effectiveness of the succession of P, S, T, Q, E and C holes in sealing the bedrock voids presents some anomalies until the last stages in the segments 6 and 7 compared to that of segment 12. This happened for two reasons:

a) the steepness (almost vertical and subparallel) of the faults occuring there and their frequency (1-2m apart), the grouting boreholes being also vertical, and

b) most of the injected grout was thick (1 : 1 water cement ratio). The thick grouts caused premature filling of the smaller cracks interconnected with the big voids (which resulted from the presence of the faults).

However, anomalies such as those exhibited in the curves of segments 6 and 7 (Fig. 4.28) and high grout takes in the last stages Q, E and C holes (Fig. 4.27) of the segments 7 and 10 do indicate that a certain number of open cracks still remain.

Although these are in isolated intervals within the segments they are potential leakage paths, and if it is not economical to proceed with further grouting, drainage must be provided, but with extreme caution because of the presence of the fine alteration products.

4.4 Sfikia dam

4.4.1 Description of project

Sfikia dam is located on the Aliakmon River, 10 km upstream from Assomata dam. Under construction at the time of writing, it is scheduled to start generating power next year (1984). It is a pump storage scheme and will operate in close relation with Assomata dam. The Sfikia project comprises a rockfill zoned dam with inclined core, 82 metres high, founded on a steeply-walled gorge of metamorphosed rocks (meta-andesites, gneiss and amphibolite schists). It has a riverbed width of 40m and its crest length is 220 metres (see Plates 4.14, 4.15 and Fig. 4.29). All its appurtenances lie within the left abutment. That is:

FIGURE 4.29

Sfikia Dam Layout

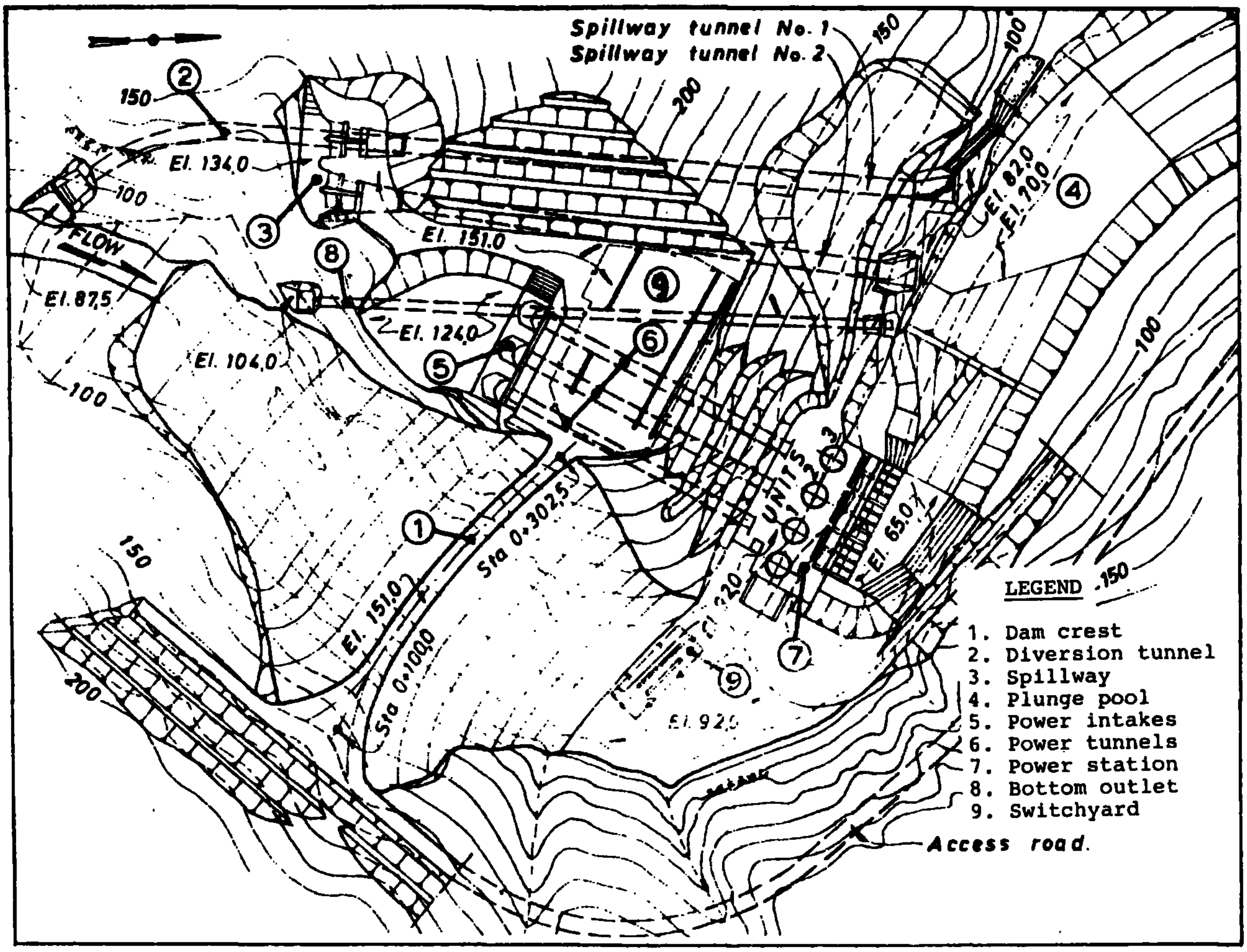


Plate 4.14
Sfikia dam

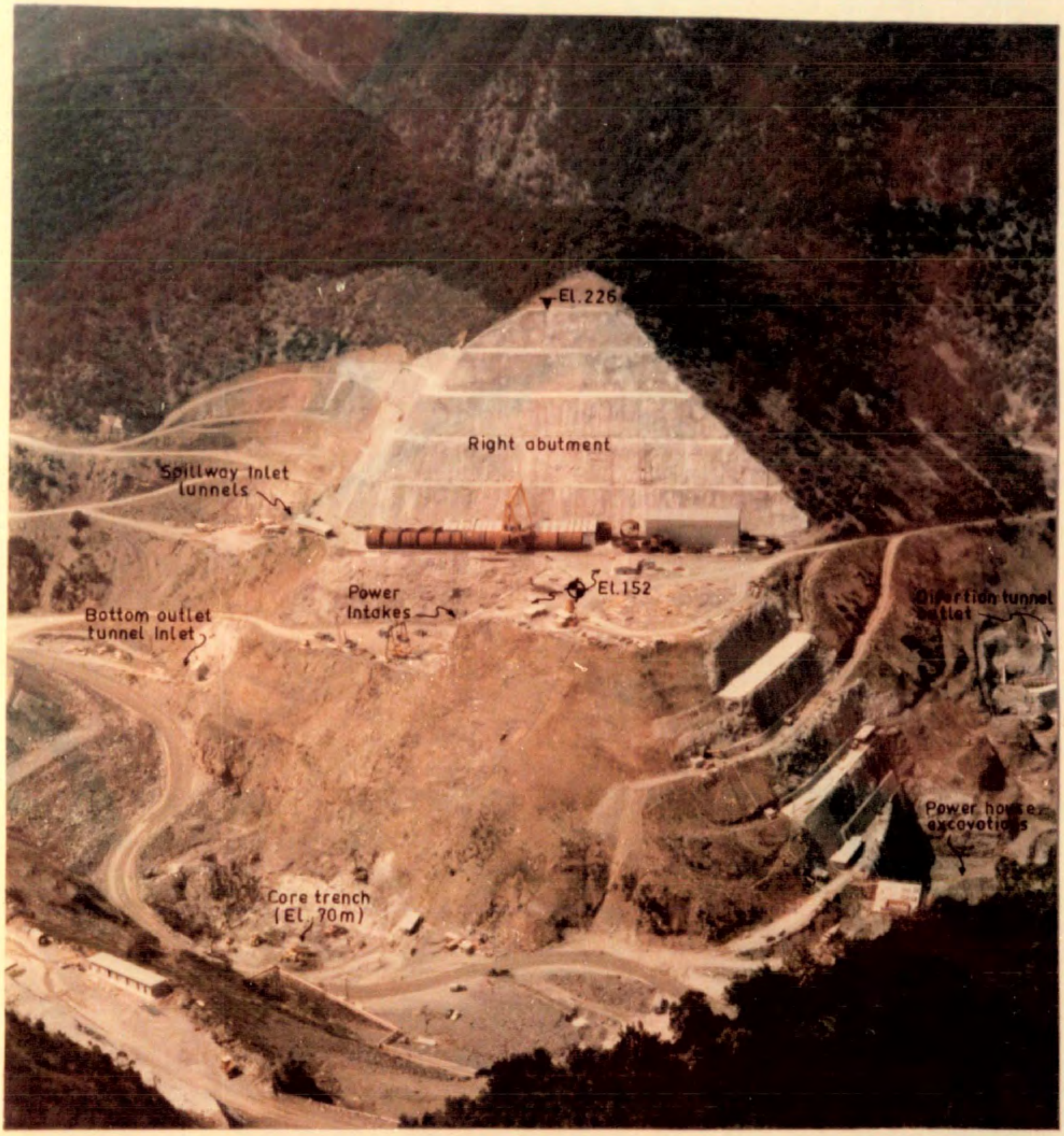


Plate 4.15 Sfikia dam: Excavations at riverbed and left abutment; general view

Plate 4.16
SFIKIA DAM
Panoramic view

Three concrete and steel lined power tunnels of 7 metres in diameter and 170 metres long. They lead to a surface powerhouse with installed capacity of 315 MW. Each generator (105 MW) is reversible and is able to pump water from Assomata reservoir.

The bottom outlet tunnel which is 312 metres long, and has its half front length (156m) concrete lined (3.5 metres in diameter) and its final half length (156m) concrete and steel lined (3 metres in diameter).

The diversion tunnel which is 490 metres long, concrete lined at 7.5 metres internal diameter.

The two spillway tunnels which consist of 50 metres long inclined shafts (about 45^{o}) and horizontal sections of 253 metres long. The horizontal section of spillway tunnel No.1 is the second half of the diversion tunnel.

The excavation products are used as rockfill materials, and graded in zones, the outer zones being the sound and oversized ones, the inner zones being the finer and weathered excavation materials.

4.4.2 Foundation bedrock conditions: an assessment prior to grouting

Sfikia dam foundation conditions and the anticipated corrective measures for strengthening and sealing of the foundation bedrock have been investigated in detail. The exploration programme conducted was similar to that of Pournari and Assomata damsites (see Section 4.2.2). From the bulk of the information collected as above, selective features will be used in this study to delineate foundation bedrock conditions as outlined below.

A. Geomorphological and lithological characteristics of the damsite

The Sfikia H.E. project and its reservoir lie in meta-andesite, gneiss and amphibolite schist (undifferentiated) formations at the eastern bound of the Pelagonian massif (geotectonic zone). The river trends SW to NE forming a sigmoidally-shaped canyon downstream of the dam axis (see Plate 4.15). At the damsite the river forms a very steep rocky gorge surrounded by steeply-incised creeks. In spite of the very abrupt cliffs along the river there are at the damsite only a few shallow screes (1-2m deep).

Exceptions are the one deeper rock slide (a few metres deep) in front of the spillway inlet structure and the one located immediately downstream of the outlet portal of the diversion tunnel. This latter one is of mixed nature; that is, it includes rock slide debris and river or torrential fan materials from the nearby creek.

From upstream to downstream of the dam axis and from the top of the core trench to the bottom (riverbed), the river has eroded undifferentiated formations of meta-andesites, gneiss and amphibolite schist formations, in which are included several aplitic, as well as quartz,veins of several centimetres thick.

An almost vertical fault line, exhibiting a mean dip direction of 300^{o} and a varying dip of 70^{o}-85^{o}, breaks the continuation of the formations mentioned above. It traces the left abutment in a straight line, marked with a sheared red clay band of several centimetres thick (up to 1 metre) from the left wall of the spillway approach channel to the left wall of the power house excavations. It traverses the diversion tunnel,the spillway and the bottom outlet tunnels, and also traverses the river and the right abutment. Along that fault there is a dyke of massive intrusive rocks such as gabbros, amphibole-and olivine-rich rocks (of basic origin). There are also dioritic rocks and large inclusions of meta-andesites and amphibolite gneiss schists. This denotes the complex geotectonic history of the area (see Chapter 3, Section 3.2.4). The upstream contact of the dyke is separated from the basement rocks with a 0.5 to 1m thick zone of red clays and a brecciated zone with clay up to 40 metres thick (Power house fault). The dyke rocks are interdigitated with the older rocks as far as the outlet portal of the diversion tunnel,but do not form a distinct shear plane in the manner of that of the red clays.

B. Bedding/schistocity and folds

At the site, upstream of the previously mentioned dyke, the foliation has an average dip and dip direction of $55^{o}/290^{o}$. Locally, dip directions vary between 270^{o} and 310^{o} and dips vary between 40^{o} and 70^{o}. The causes are minor displacements due to the existence of small faults and microfolding which have not substantially affected the averages referred to above (see Figs. 4.31, 4.32, 4.33 and 4.34). Folds at the site have not been recorded. Folded strata are in abundance upstream and downstream of the damsite. The major regional axes of the observed folds run SW to NE and NW to SW.

C. Joints, shears and faults

Joints, shears or minor faults are in abundance in the site (see Fig. 4.31 and Plate 4.17). Although they appeared to have a local provenance, together with the power house fault (which seems regional) they have caused a high degree of stress relaxation.

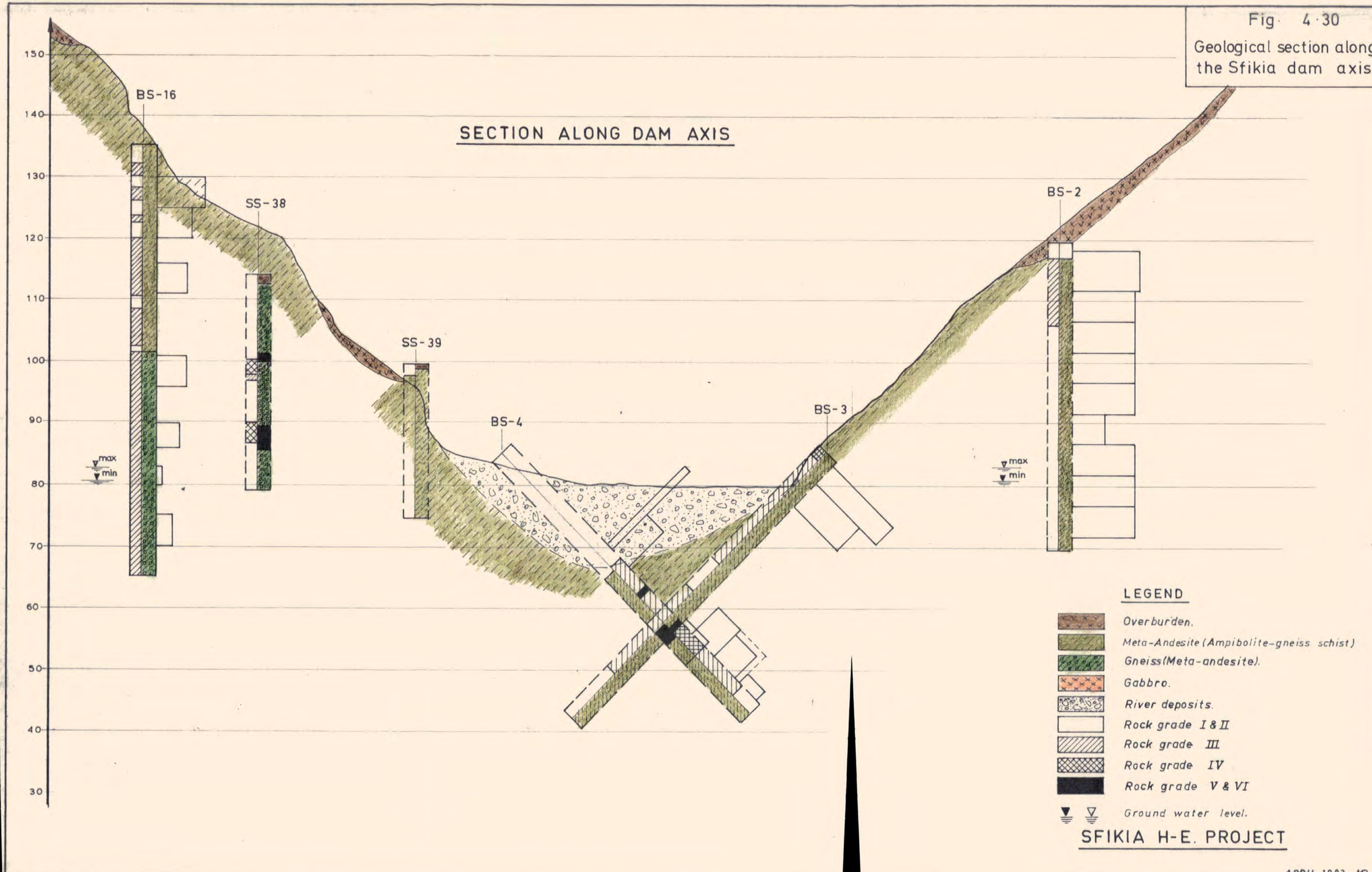


Fig. 4·30
Geological section along the Sfikia dam axis
SECTION ALONG DAM AXIS
BS-16
SS-38
SS-39
BS-4
BS-3
BS-2
150
140
130
120
110
100
90
80
70
60
50
40
30
max
min
LEGEND
Overburden.
Meta-Andesite (Ampibolite-gneiss schist)
Gneiss (Meta-andesite).
Gabbro.
River deposits.
Rock grade I & II
Rock grade III
Rock grade IV
Rock grade V & VI
Ground water level.
SFIKIA H-E. PROJECT
APRIL 1983, MD

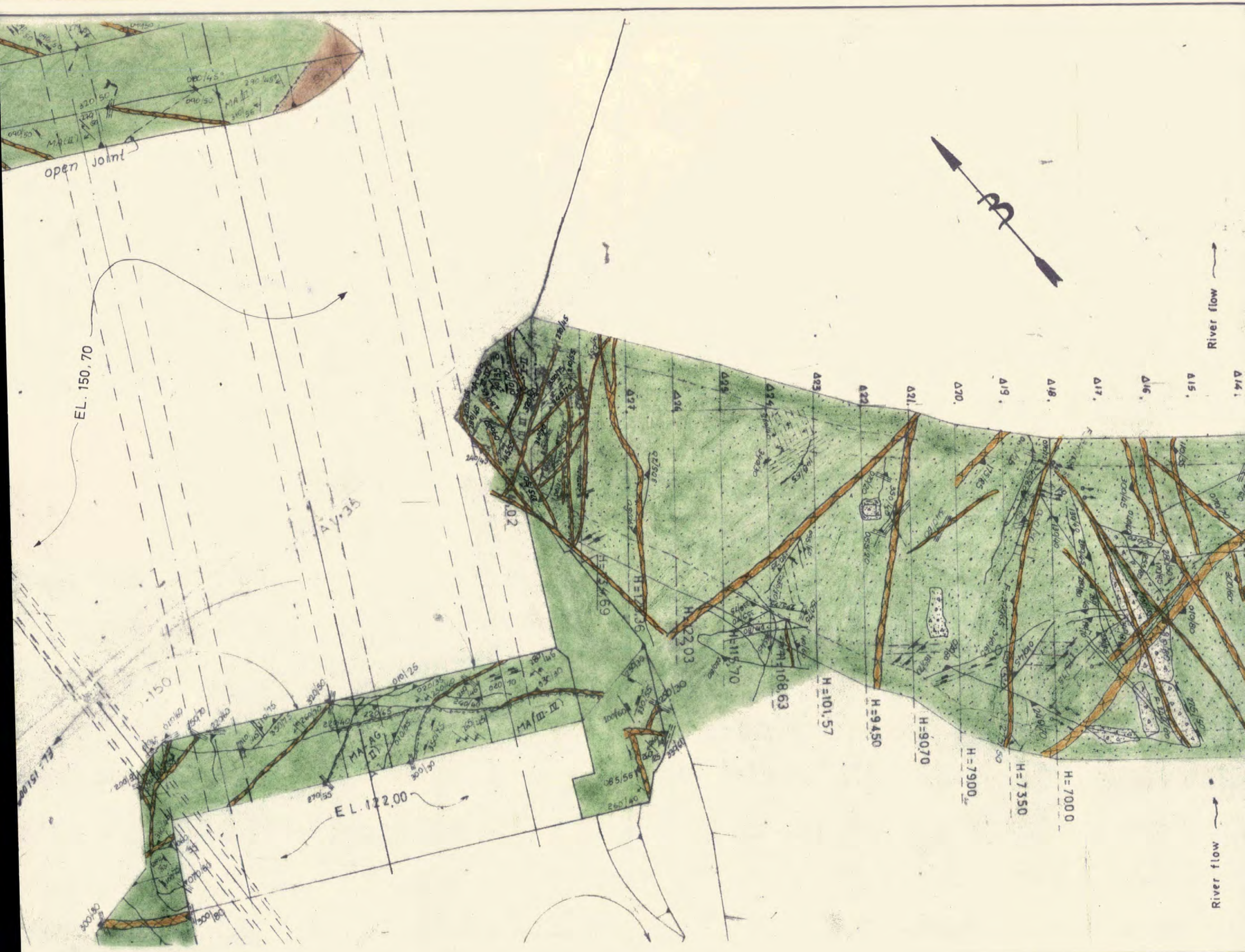
Fig 4. 31 SFIKIA DAM Core Trench Geology

LEGEND

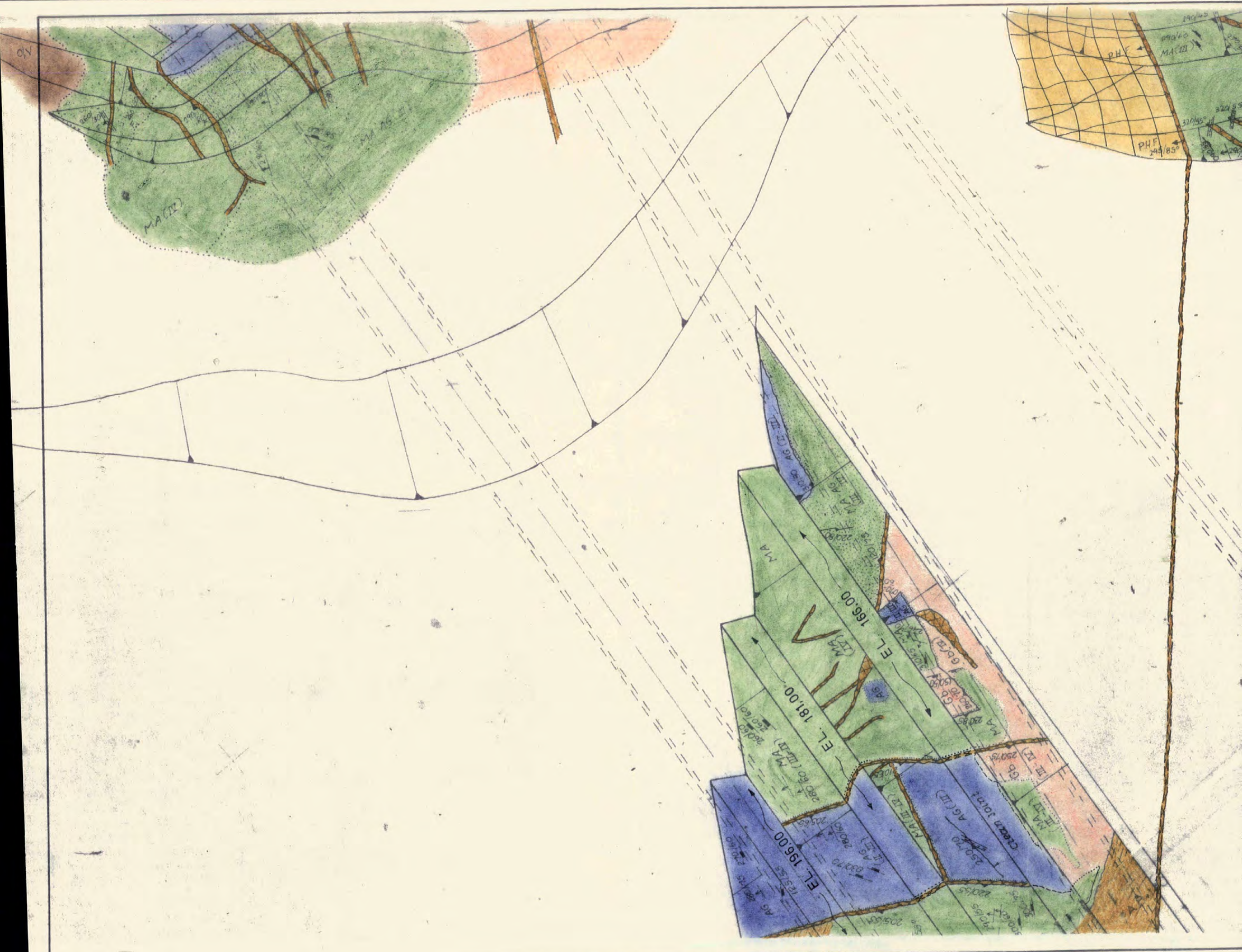
O/V	Overburden.
Gb	Gabbro
MA-AG	Meta-Andesite-schist (Amphibolite-gneiss-schist, undiferenciated)
MA	Meta-Andesite-schist.
	Concrete Slabs
	Rock / Rock boundary certain.
	Boundaries between rock weathering grades.
110/35	Major/shear joints (dip direction/dip)
360/25	Fault/shear (dip direction/dip)
080/85	Surface exposure of shears/faults (dip direction/dip.)
190/75	Joint (dip direction/dip)
290/50	Schistosity/foliation (dip direction/dip)
	Water inflow.

ROCK WEATHERING GRADES

(I)	Fresh	(F)
(II)	Slightly weathered	(SW)
(III)	Moderatery weathered	(MW)
(IV)	Highly weathered	(HW)
(V)	Completely weathered	(CW)
(VI)	Residual soil	(R)



open joint
EL. 150,70
EL. 122,00
River flow
River flow
H=101,57
H=94,50
H=90,70
H=79,00
H=73,50
H=70,00
H=108,63
H=115,70
H=122,03
MA(III-IV)



EL. 166.00
EL. 181.00
EL. 196.00
MA (II)
PHF
clean joint

POLAR NET
(Upper Hemisphere Projection)

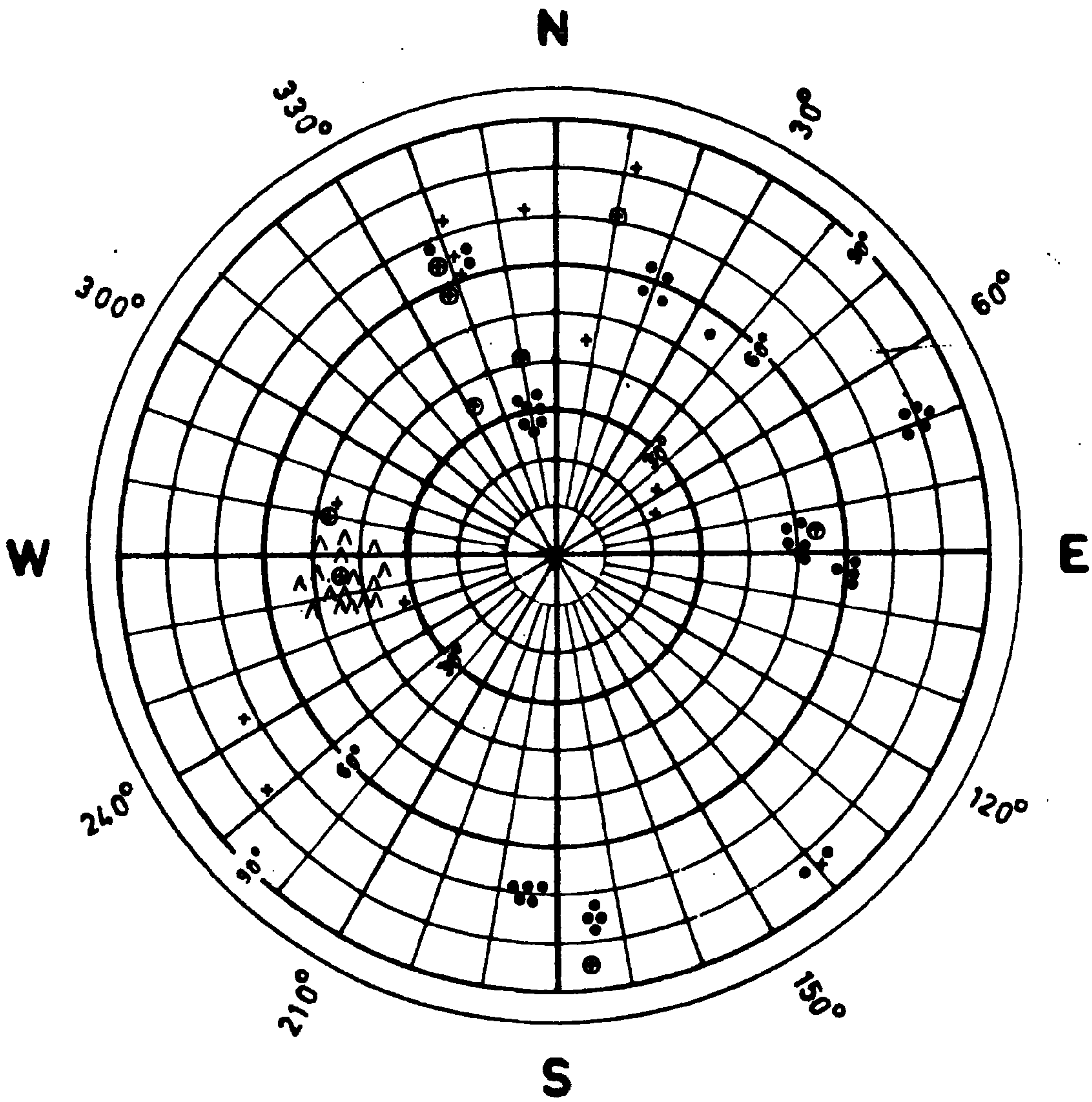


Fig. 4.32 *Discontinuity measurements along dam axis (Left abutment, Sfikia dam).*

LEGEND

- • Joints
- + Minor shears and faults
- ⊕ Faults traversing the core trench (and with clay)
- ʌ Foliation / schistocity

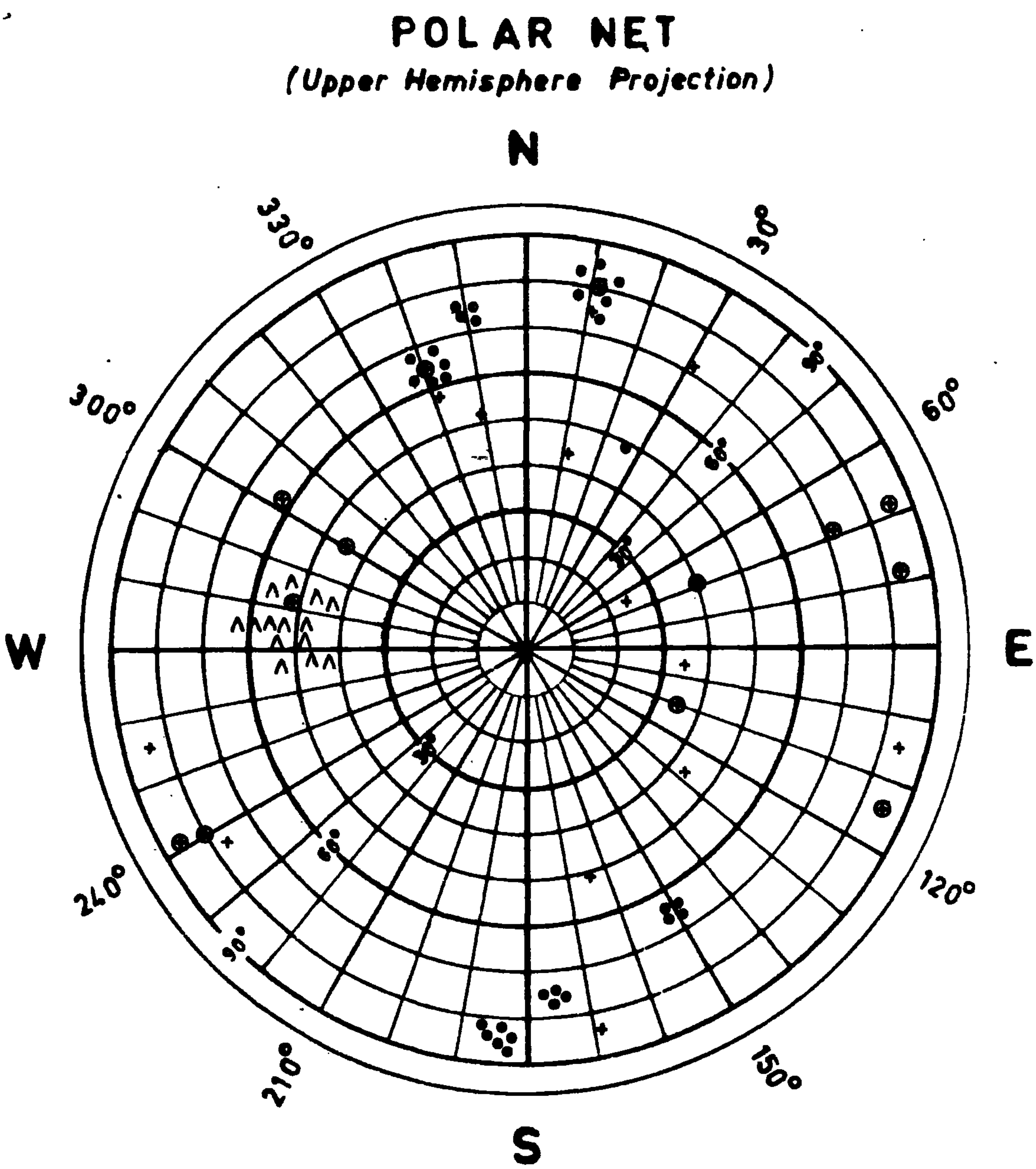


Fig. 4.33 Discontinuity measurements along dam axis (Riverbed, Sfikia dam).

LEGEND

- ● Joints
- + Minor shears and faults
- ⊕ Faults traversing the core trench (and with clay)
- ʌ Foliation / schistocity

POLAR NET
(Upper Hemisphere Projection)

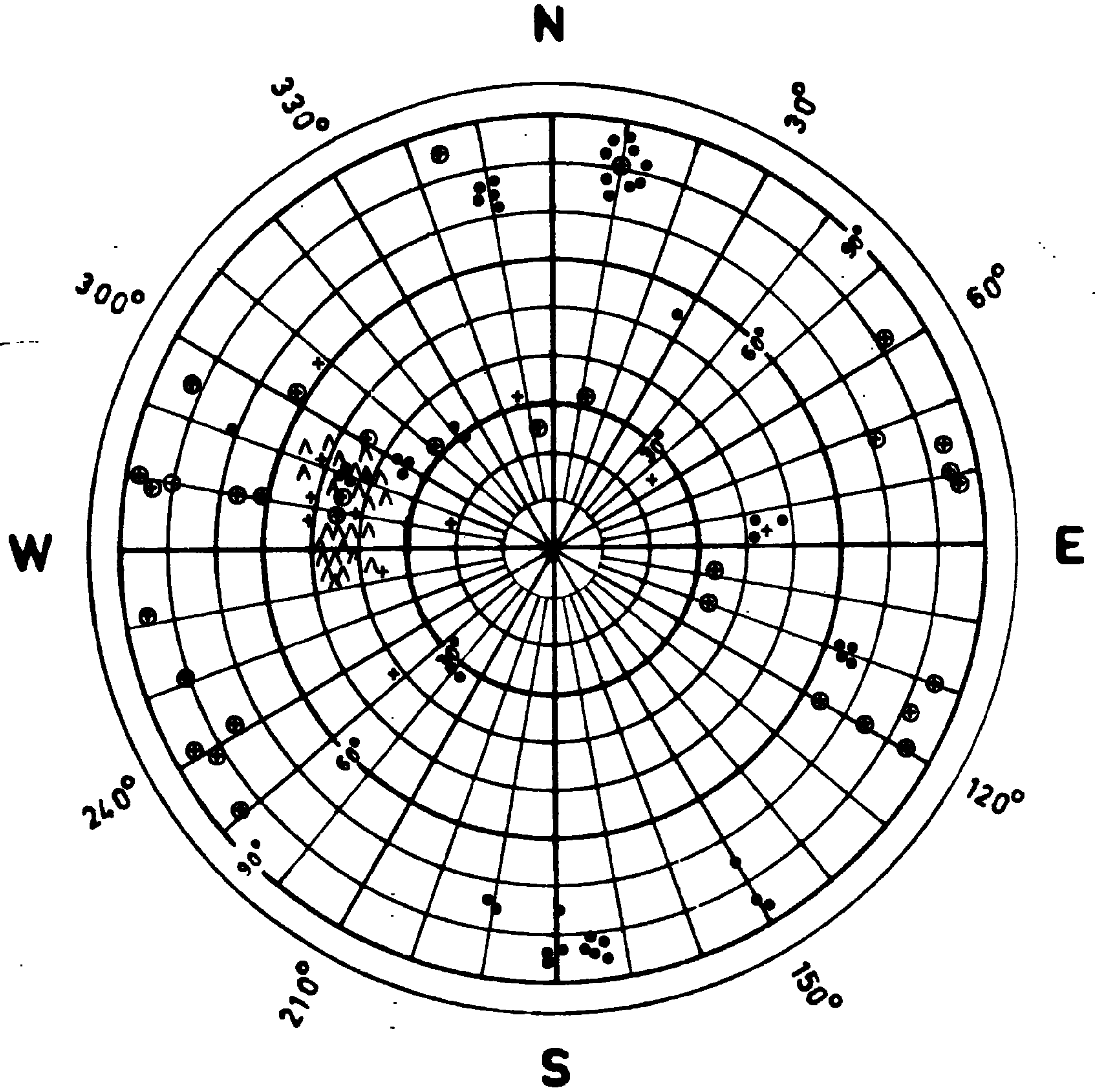


Fig. 4.34 *Discontinuity measurements along dam axis (Right abutment, Sfikia dam).*

<u>*LEGEND*</u>

- • *Joints*
- + *Minor shears and faults*
- ⊕ *Faults traversing the core trench (and with clay)*
- ∧ *Foliation/schistocity*

This is suggested by a) the weathering which is up to grade V, (see next Section D) in many shears and faults, either steep (almost vertical) or shallower ones, the observed open - or clay - filled joints and shears, and c) the low water table at river level in both abutments (see Fig. 4.30).

Shears or faults, other than the power house fault, have caused a small degree of brecciation or shattering of the damsite foundation bedrock into discrete units of several centimetres up to approximately 1 metre thick (see Plates 4.17 and 4.18). In the zone of the power house fault several metres of brecciated and clayey zones exist in addition to relatively sound extensive lenses (blocks) of older or younger rocks.

Along the minor shears and faults, displacements of a few centimeters up to 1 to 2 metres have been observed. At the power house, fault striations and stylolitic structures suggest that up-thrust movements have occurred and are probably continuing. A difference of 6 metres in the rock contours of the riverbed (detected by boreholes) between the two sides of the power house fault (the downstream rocks being higher) might suggest a similar upthrust as that indicated on Plate 3.2.

D. Weathering characteristics

Weathering at Sfikia is classified in six grades from a fresh to a completely-weathered rock (residual soil).In the power house fault zone, deep penetrative weathering is observed, even of grades V and VI (for narrow bands) in the horizontal section of the spillway and bottom outlet tunnels. In the rest of the site, weathering is also deep but restricted to narrow bands along

Plate 4.17 Fault treatment at core trench, Riverbed, at El. about 65 metres (Sfikia dam)

Plate 4.18 Core trench clearing, and grouting in progress. Right abutment at Sfikia dam.

faults, and particularly along the sheared zones sub-parallel to the foliation. Bands of reddish clay along faults are observed, even in the deeper rocks penetrated by exploratory boreholes.

E. Ground water characteristics

The ground water table profile (derived from piezometer readings) is about horizontal at river level in both abutments. The annual fluctuation of the recorded water level barely reaches 1 metre.

This suggests quite permeable rocks, even for very deep horizons. The water table in the left abutment was at the same elevation at either side of the power house fault.

4.4.3 Permeability characteristics

Foundation bedrock permeabilities were investigated with packer pressure tests using clean water and carried out from boreholes drilled along the dam axis in the river bed and the abutments. The results are presented in Figures 4.35 to 4.38. The main variable interactions that could be examined were:

Permeability versus rock types encountered

Permeability versus RQD

Permeability versus weathering

Permeability versus depth.

The tests were executed at five metre intervals at allowable pressures not exceeding the overburden pressure, and for very deep holes not exceeding a pressure of 15 kg/cm^2 (1 471 kN/m^2).

Each test is symbolized to represent the main lithological units encountered so as to provide an indication of the interrelations between rock types, weathering and fracture characteristics in terms of RQD, and position in relation to depth.

A first examination of the permeability tests presented in Figs. 4.35 to 4.38 indicates that foundation bedrock in the Sfikia damsite is permeable. All the tests carried out exhibited permeability values higher than 1 Lugeon = 1.3 x 10^{-7} m/sec and and thus an improvement (by grouting) is required (Houlsby, 1982).

If that is compared with the Pournari and the Assomata permeability tests (see Sections 4.2.3 and 4.3.3), it is obvious that Sfikia dam foundation is a special situation. The curtain has to be constructed to quite a depth in order adequately to increase the leakage path length, and to be quite tight. This is an essential requirement, since Sfikia is a pump storage scheme.

The main conclusions which can be drawn from examining the permeabilities recorded in relation to rock type, weathering, RQD and depth are as follows:

Permeabilities are almost equally high in all the rock types encountered (see Fig. 4.35).

There is no clear tendency towards reduction of permeability values as compared with RQD values (see Fig. 4.36).The exhibited RQD values suggest that jointing of the foundation rock is high. At least three quarters of the tests performed exhibited RQD values smaller than 50 per cent and this suggests that the fracture frequency is in the range of about 5 to 15/m and even more in very poor rocks (Farmer, 1978).

Permeability values appeared to be high, independent of the weathering condition of the rocks encountered(Fig.4.37). Very few tests in sound rock exhibit permeability values around one lugeon.

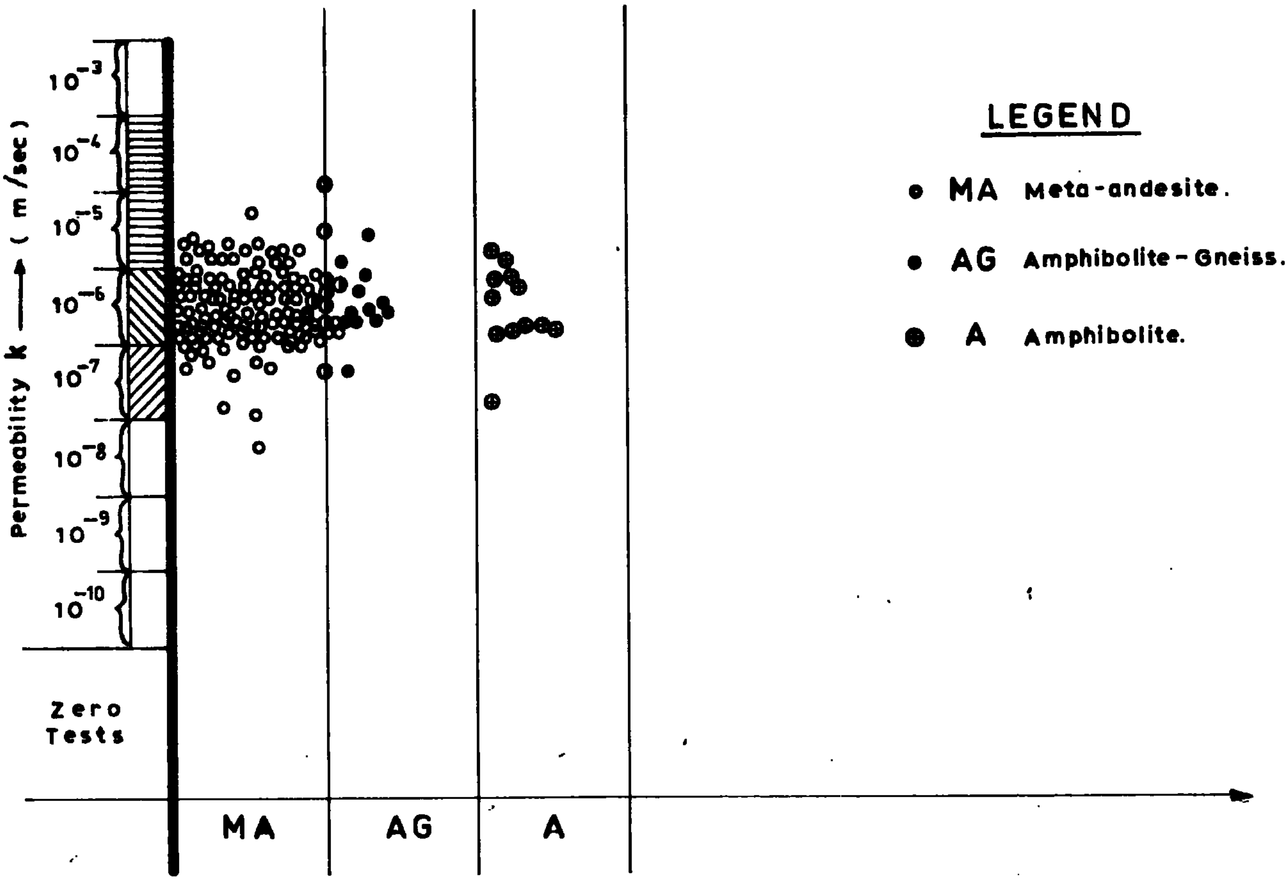


Fig. 4·35 Permeability vs rock types encountered

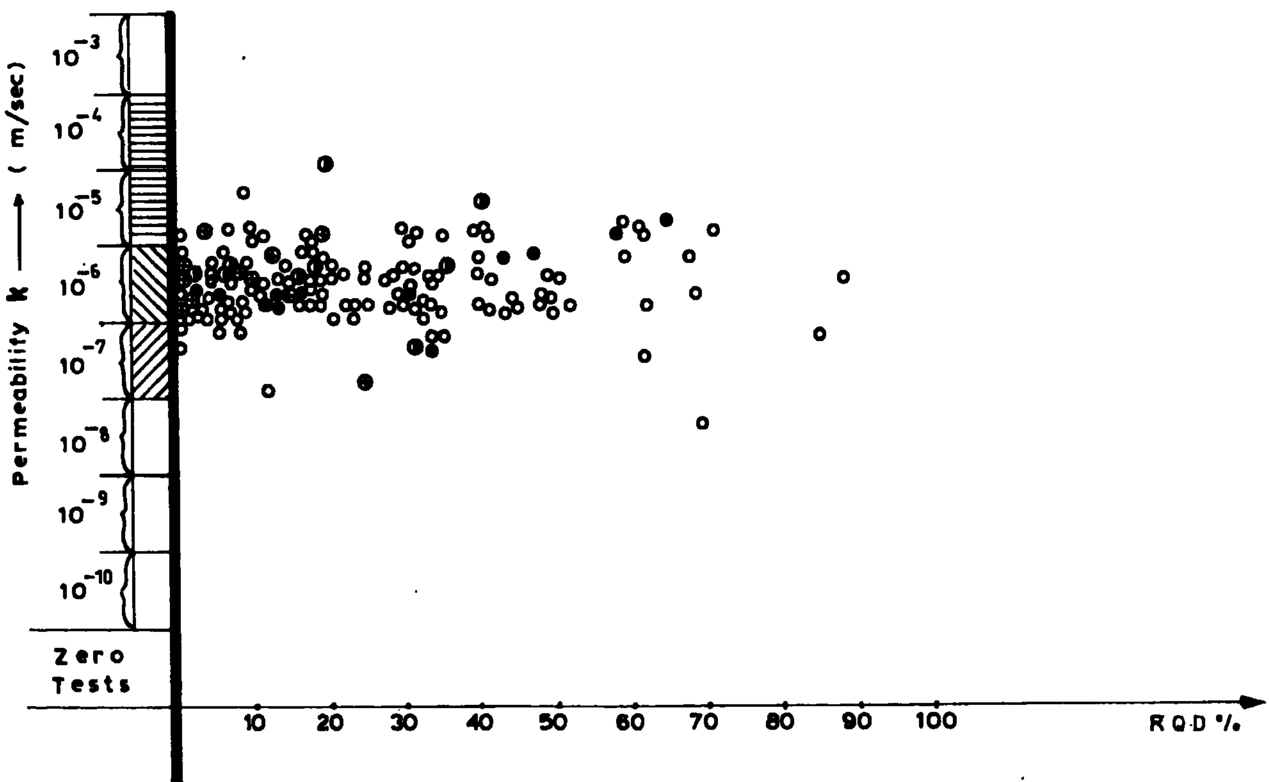


Fig. 4·36 Permeability vs R Q D

Sfikia HEP

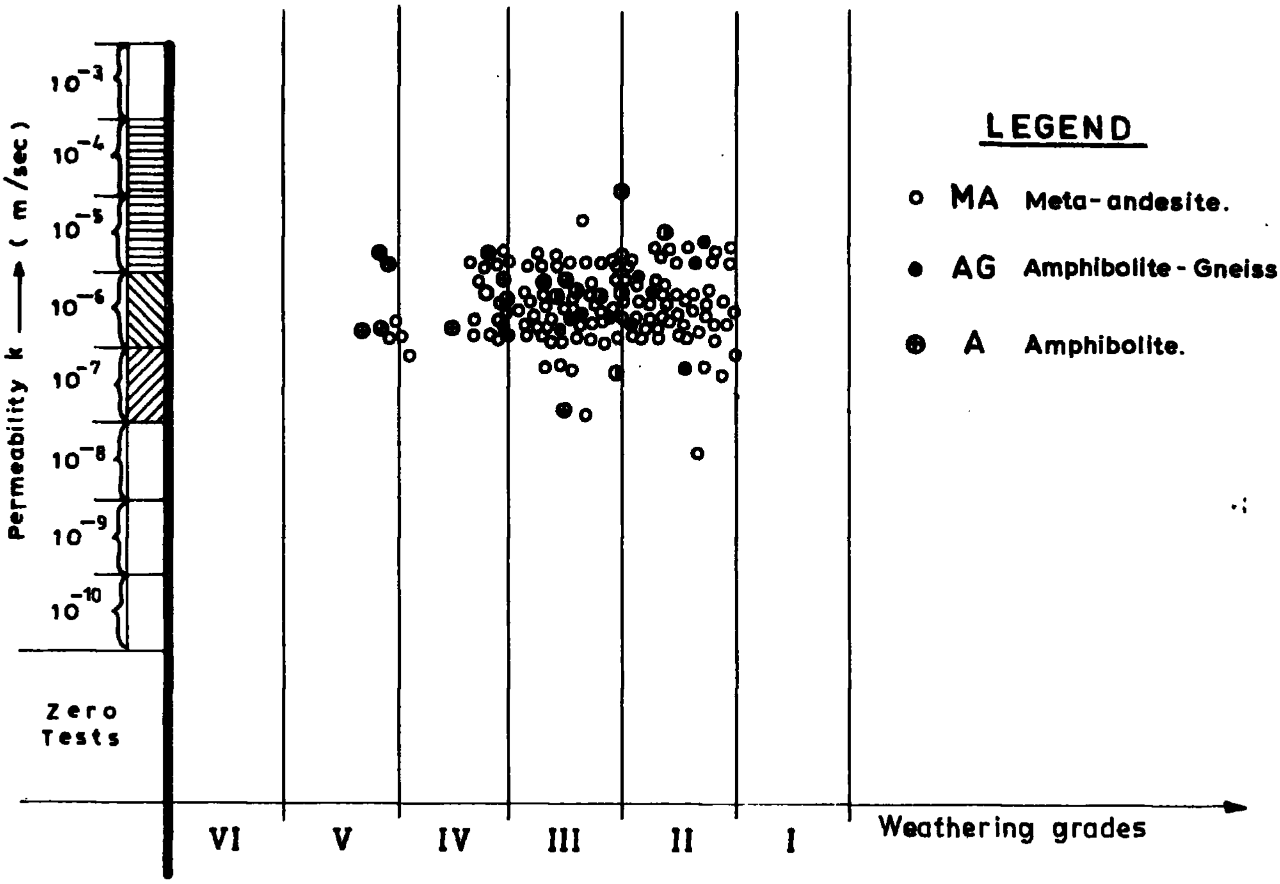


Fig. 4·37 Permeability vs weathering

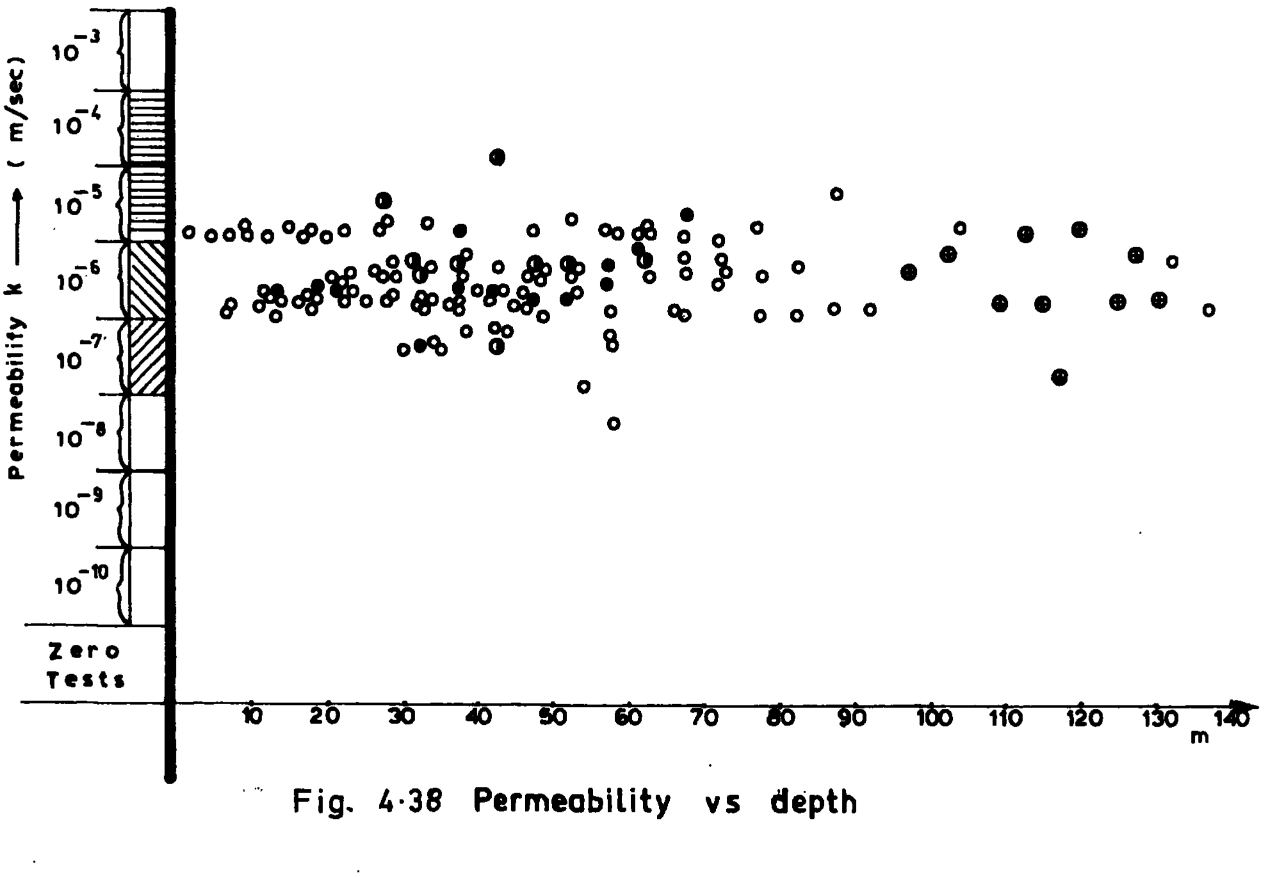


Fig. 4·38 Permeability vs depth

Sfikia HEP.

Permeability values remain equally high in shallow or deeply (down to 140m) seated rocks (<u>see</u> Fig. 4.38).

The above conclusions draw attention to three important questions:

a) Do large openings (wide fractures) exist deeper than the design limits of the grout curtain?

b) To what extent will the designed grouts penetrate the finer fractures which are in abundance in the core trench?

c) In particular, how effectively will the most weathered rock zones be sealed?

In the above question (b) there are two positions taken by various researchers. One is to initiate hydrofractures by applying higher pressures (<u>see</u> Attewell and Farmer, 1976) in a strictly controlled manner. The other is that the gravitational stresses superimposed by the dam body acts beneficially in reducing the effective aperture of the existing cracks (stress dependent permeability; Maini, <u>et al</u>, 1972).

Sharp (1972) reports that experiments by Bernaix (1969) on a laboratory scale have shown that confining pressures in a triaxial cell can change the permeability of fissured rock by three or four orders of magnitude. The Malpasset dam failure is an example (Bernaix, 1967) in which, due to the loads imposed by the dam, permeabilities were reduced approximately by the order of 10^2.

4.4.4 Grouting at Sfikia

(a) General

Grouting at Sfikia is needed for three main reasons:

1) To reduce leakages, under the dam and from the abutments.
2) To strengthen (by consolidation grouting; see Plate 4.19) the foundation rock under the core. Blanket grouting also assists the performance of the main curtain by sealing off all discontinuities traversing the core trench, and hence eliminating any danger of erosion of soft foundation rock or core material.
3) To reduce any uplift pressures in the abutments.

Grouts, drilling methods and grouting procedures are the same as those for Pournari and Assomata (see Sections 4.2.4 (a) and 4.2.4 (b)). Drilling of blanket grout holes is performed by rotary percussion drill rigs.

It is worth noting also that the pressures applied in the upper four intervals (20m below ground surface) are much lower than the overburden pressures. These gradually rise to 1 kg/cm^2 (98.067 kN/m^2) at a packer depth of 15m. Below that level (deeper grouting intervals) grouting pressures were increased up to overburden pressure (but not exceeding 1 471 kN/m^2 for very deep intervals).

(b) Grouting results

The grouting results presented here concern the main grout curtain for the completed segments. Mention of the blanket (consolidation) grouting results will be made where necessary to denote important geological factors involved.

Plate 4 19 Blanket (consolidation) grouting results and boreholes layout (Sfikia dam)

Note The complete plan is included in the supplementary volume

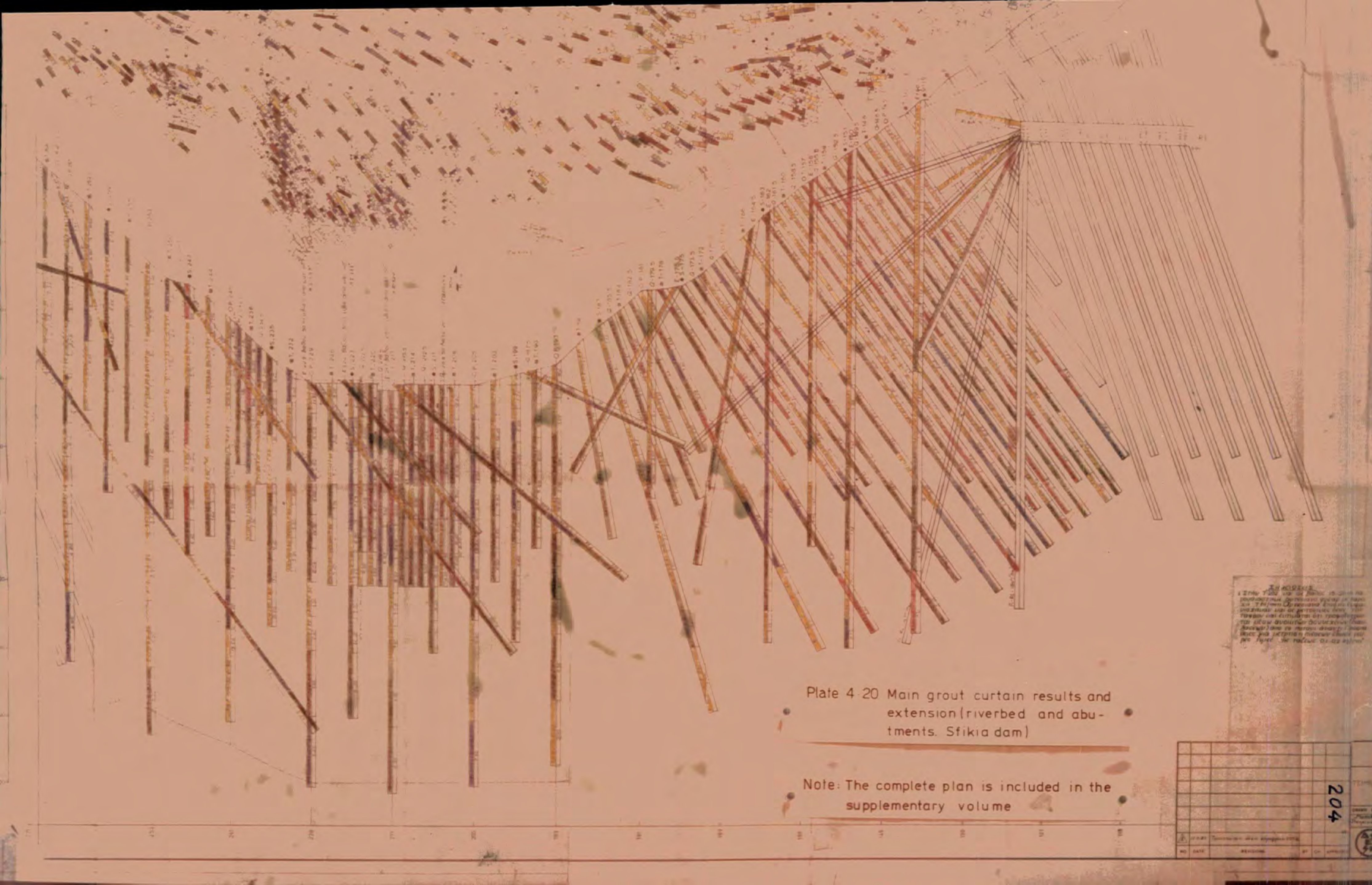


Plate 4.20 Main grout curtain results and extension (riverbed and abutments. Sfikia dam)

Note: The complete plan is included in the supplementary volume

Plate 4·21 Main grout curtan results (left abutment extension Sfikia dam)

Note: The complete plan is included in the supplementary volume

TABLE 4·15

Main Grout Curtain Results

Segment Nº / Grouted surface (m²)		1	2	3	4	5	6	7	8	9	10	11	12
Grouted surface (m²)		4750	5000	5000	5000	5000	4350	3375	3148	3750	4250	4500	3100
Grout Take per Segment (kg)								40785	35982	148120			
Grouting B.H. Length (m)								985	952	1427			
(m/m²) Density of B.H.								0.29	0.302	0.380			
Grout Take per m-run of B.H. (kg/m)								41.41	37.80	103.8			
Grout Take per Area of Segment (kg/m²)								12.084	11.43	39.5			
Grout Take per m-run per Area of Segment (kg/m³)								3500×10^{-3}	3450×10^{-3}	15010×10^{-3}			
Volume of grout used per m-run per Area of Segment. (m³/m³ x unit)								4660×10^{-3}	4590×10^{-3}	20000×10^{-3}			
P	Grout Take (kg)							16716	7540	60992			
	Length of B.H. (m)							385	180	217			
S	Grout Take (kg)							13274	9037	12877			
	Length of B.H. (m)							205	160	219			
T	Grout Take (kg)							7845	7225	28746			
	Length of B.H. (m)							290	220	386			
Q	Grout Take (kg)							2048	7728	31581			
	Length of B.H. (m)							75	237	358			
E and C	Grout Take (kg)							902	4452	13924			
	Length of B.H. (m)							30	155	247			

Sfikia HEP

TABLE 4·16

Main Grout Curtain Results of Primary (P).... Boreholes.

Segment N°	1	2	3	4	5	6	7	8	9	10	11	12	
Grouted Surface Area (m^2)	4750	5000	5000	5000	5000	4350	3375	3148	3750	4250	4500	3100	
Grout Take (kg cement)							16716	7540	60992				
Grouting B.H. Length (m)							385	180	217				
Density of B.H. (m/m^2)							0.114	0.057	0.058				
Grout Take per m-run of B.H. (kg/m)							43.42	41.89	281.07				
Grout Take per Area of Segment (kg/m^2)							4.95	2.40	16.26				
Grout Take per m-run per Area of Segment (kg/m^3)							564×10^{-3}	137×10^{-3}	943×10^{-3}				
Volume of grout used per m-run per Area of Segment. (m^3/m^3 unit)							751×10^{-3}	182×10^{-3}	1256×10^{-3}				

TABLE 4·17

Main Grout Curtain Results of Secondary (S) Boreholes.

Segment N°	1	2	3	4	5	6	7	8	9	10	11	12	
Grouted Surface Area (m^2)	4750	5000	5000	5000	5000	4350	3375	3148	3750	4250	4500	3100	
Grout Take (kg cement)							13274	9037	12877				
Grouting B.H. Length (m)							205	160	219				
Density of B.H. (m/m^2)							0.061	0.051	0.058				
Grout Take per m-run of B.H. (kg/m)							64.75	56.48	58.80				
Grout Take per Area of Segment (kg/m^2)							3.93	2.87	3.43				
Grout Take per m-run per Area of Segment (kg/m^3)							240×10^{-3}	150×10^{-3}	200×10^{-3}				
Volume of grout used per m-run per Area of Segment. (m^3/m^3 = unit)							320×10^{-3}	200×10^{-3}	270×10^{-3}				

TABLE 4·18

Main Grout Curtain Results of Tertiary (T) Boreholes.

Segment N°	1	2	3	4	5	6	7	8	9	10	11	12
Grouted Surface Area (m^2)	4750	5000	5000	5000	5000	4350	3375	3148	3750	4250	4500	3100
Grout Take (kg cement)							7845	7225	28746			
Grouting B.H. Length (m)							290	220	386			
Density of B.H. (m/m^2)							0.086	0.070	0.103			
Grout Take per m-run of B.H. (kg/m)							27.05	32.84	74.47			
Grout Take per Area of Segment (kg/m^2)							2,32	2.29	7.66			
Grout Take per m-run per Area of Segment (kg/m^3)							200×10^{-3}	160×10^{-3}	790×10^{-3}			
Volume of grout used per m-run per Area of Segment. (m^3/m^3= unit)							270×10^{-3}	213×10^{-3}	1050×10^{-3}			

TABLE 4 19

Main Grout Curtain Results of Quaternary..(Q) Boreholes.

Segment N°	1	2	3	4	5	6	7	8	9	10	11	12	
Grouted Surface Area (m^2)	4750	5000	5000	5000	5000	6350	3375	3148	3750	4250	4500	3100	
Grout Take (kg cement)							2048	7728	31581				
Grouting B.H. Length (m)							75	237	358				
Density of B.H. (m/m^2)							0.022	0.075	0.095				
Grout Take per m-run of B.H. (kg/m)							27.31	32.61	88.21				
Grout Take per Area of Segment (kg/m^2)							0.61	2.45	8.42				
Grout Take per m-run per Area of Segment (kg/m^3)							13×10^{3}	184×10^{3}	800×10^{3}				
Volume of grout used per m-run per Area of Segment (m^3/m^3= unit)							18×10^{-3}	245×10^{-3}	1065×10^{-3}				

TABLE 4·20

Main Grout Curtain Results of Chech. (E. and C.) Boreholes.

Segment N°	1	2	3	4	5	6	7	8	9	10	11	12	
Grouted Surface Area (m^2)	4750	5000	5000	5000	5000	4350	3375	3148	3750	4250	4500	3100	
Grout Take (kg cement)							902	4452	13924				
Grouting B.H. Length (m)							30	155	247				
Density of B.H. ([illegible]/m^2)							0.009	0.049	0.066				
Grout Take per m-run of B.H. (kg/m)							30.06	28.72	56.37				
Grout Take per Area of Segment (kg/m^2)							0.27	1.41	3.713				
Grout Take per m-run per Area of Segment (kg/m^2)							2×10^{-3}	70×10^{-3}	245×10^{-3}				
Volume of grout used per m-run per Area of Segment. (m^3/m^3 = unit)							3×10^{-3}	92×10^{-3}	330×10^{-3}				

The grouting parameters that are considered are the same as those defined in Section 4.2.3 (b).

The detailed grouting field results are presented in the "as built drawings" which are given in the Supplementary Volume 2.

For evaluation purposes the above results are gathered together in Table 4.15, while the results in the succession of Primary, Secondary, Tertiary, Quaternary and Check holes are given in Tables 4.16 to 4.20. The results are grouped in basic segments as at Pournari and Assomata for comparison purposes.

The averaged results of grout takes in kg of cement per m^2, boreholes drilled in m-run per m^2, and the volume of grout used per m-run per area of segment (dimensionless parameter),as well as the treated area in m^2 of each segment, are illustrated in Fig. 4.39. In this Figure, in which only three segments are presented (completed), some noticeable differences in the grout absorptions pertaining to the structural condition of the riverbed and the abutments will be discussed in the next Section (Section 4.4.5 below).

The foundation bedrock response (grout absorption) in the succession of P, S, T, Q, E and C grouting stages is presented in Fig. 4.40. In this Figure the difference in the amount of grout used for the various grouting stages delineates the importance of some significant faults (see Plate 4.18 and Fig. 4.31) in the different response to grouting of the segments.

Finally, Fig. 4.41 illustrates the effectiveness of the drilling and grouting effort applied to different segments.

The effectiveness achieved, as illustrated by the P, S, T, Q, and

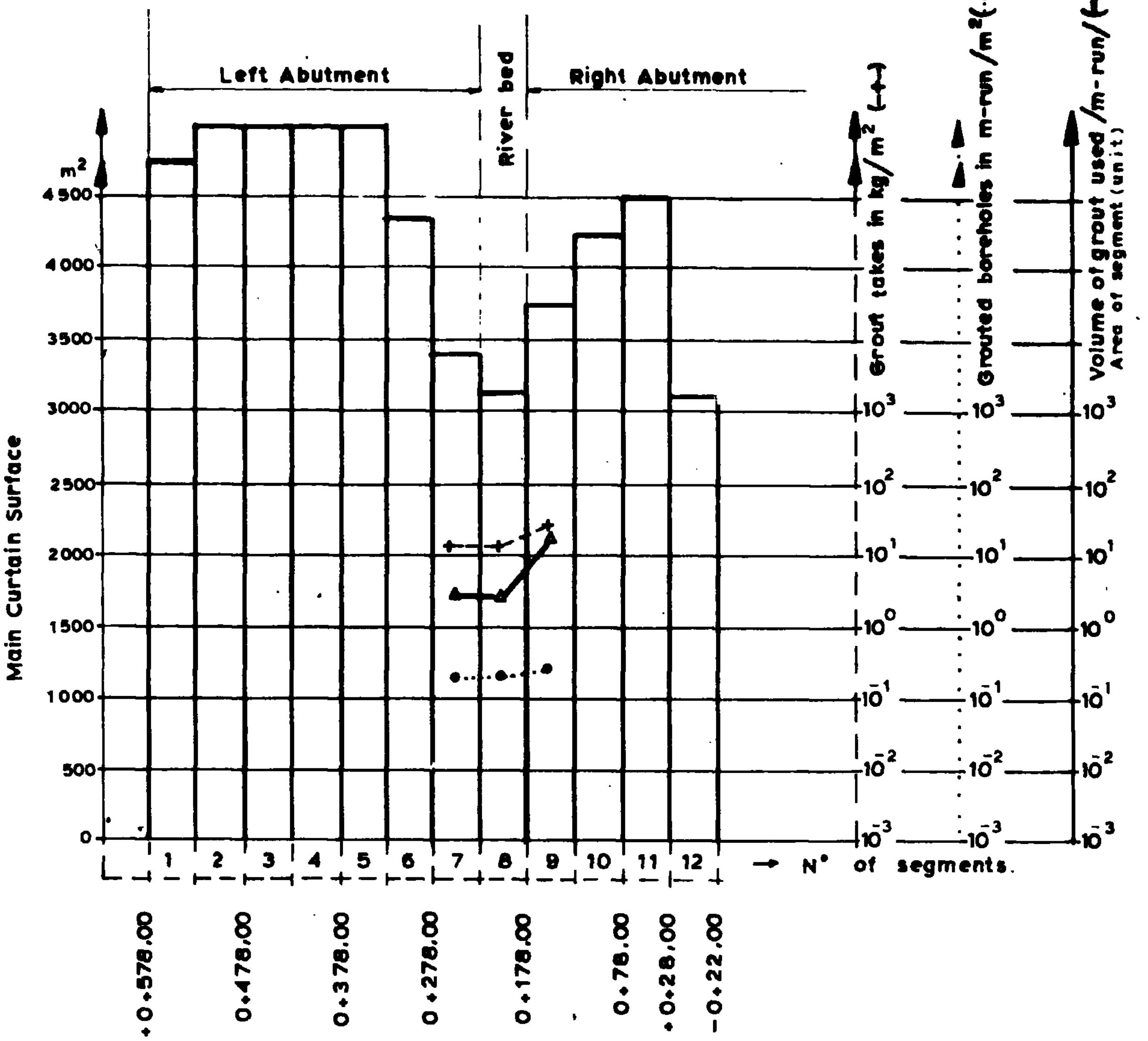


Fig : 4.39. *Foundation grouting treatment (Sfikia dam) and behaviour in terms of:*

1. *Main grout curtain extension along dam axis (m^2)*
2. *Borehole density in m-run per m^2 of curtain.*
3. *Grout takes per m^2 of the curtain.*
4. *Volum of grout/m-run of bh/area of segment (unit)*

SFIKIA HEP

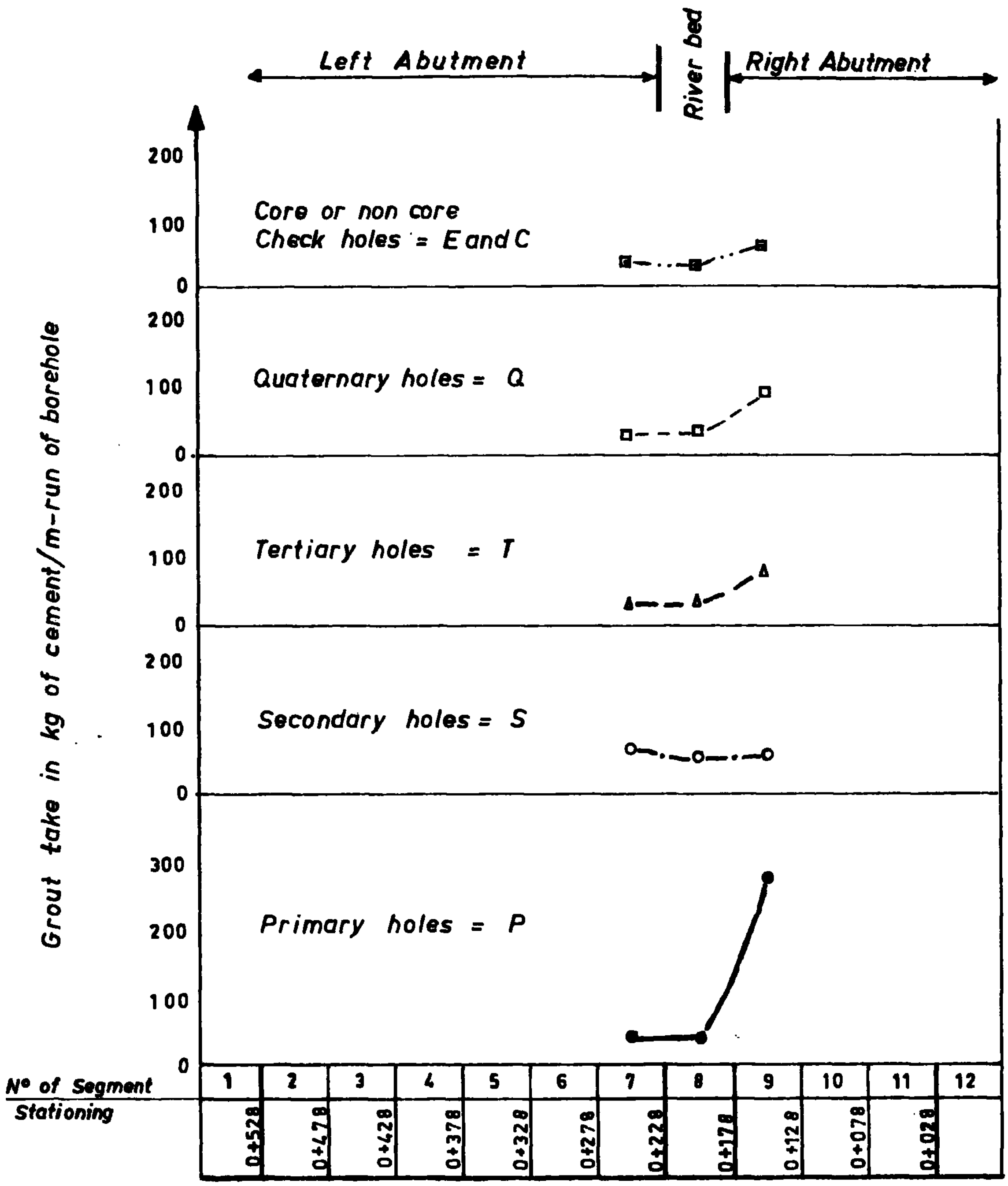


Fig. 4.40 *Comparative results of foundation bedrock behaviour in grouding treatment along the main Sfikia dam grout curtain. (Plots represent the average grout take in Kg of cement per m-run of borehole for each segment considered).*

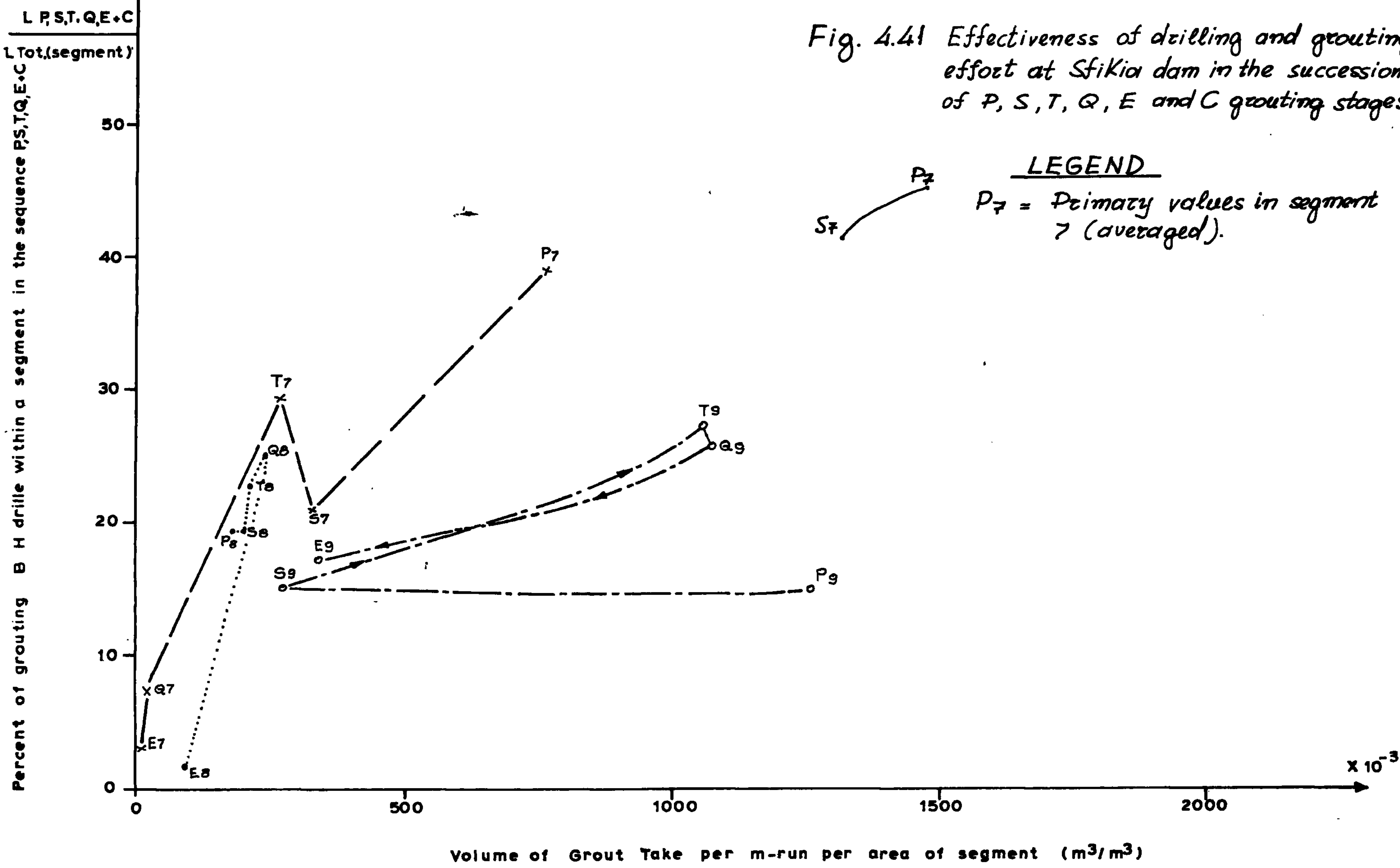


Fig. 4.41 Effectiveness of drilling and grouting effort at Sfikia dam in the succession of P, S, T, Q, E and C grouting stages.

LEGEND

P_7 = Primary values in segment 7 (averaged).

Sfikia HEP

E curves, assesses the interrelations between (a) the drilling performed, and (b) the grouts used and the nature of the geological factors involved. It also acts as a warning and offers quidance towards the appropriate future measures needed to supplement the grouting works already performed.

4.4.5 Summary discussion and conclusions on Sfikia dam

On the basis of the grouting results and the other field information available (although only three segments have been completed at the time of writing) as presented in this Section, a number of conclusions can be drawn, in spite of the complex nature of the variables considered, and the uncertainties which usually accompany the geological factors involved. In the evaluation of the grouting results, some correlations can be made between a) the grouting parameters (quantities), and the foundation conditions (rock voidness), and (b) the extent of the effectiveness of the grouting works performed, and the preset completion criteria.

On the other hand a number of ambiguities and the need for additional information from specific tests and geological records and observations are brought to light when an attempt is made a) to evaluate the foundation properties and their response to the methods of improvement applied, and (b) to grade their interrelations. These as well as suggestions for further research will be detailed in due course.

The curves (Fig. 4.39) of the averaged results of the grout used per unit surface (m^2) of the basic segmental divisions completed indicate that the quantities of grout used are much higher in segment 9 than in segments 7 and 8. The reasons for this may be attributed to the two systems of almost vertical faults and a system of shallow dip (dipping downstream) faults and the foliation which caused relaxation and probable small slips towards the river. The steepness also of the right abutment and the dip and dip direction of the observed foliation suggest that some buckling of the strata might have occurred.

The grouting results of segment 9, as illustrated in Fig.4.40 and the curves of Fig. 4.41, indicate that up to the Quaternary holes high grout takes were absorbed in this segment. Even the E and C holes put down to check the whole depth of the curtain (even below its limits) showed a high absorption zone (cement takes greater than 200 kg of cement per metre).

This led to a re-consideration of the depth of curtain for segment 9 and segment 10 and to extend it deeper (see E and C, as well as the fan holes from the lower grouting gallery of the right abutment). The blanket (consolidation) grouting results also denote that along the vertical fault line the lower part of the core trench in the right abutment caused unusual relaxation.

The above remarks warn about a potential zoning of unusually high grout takes in other segments too. Furthermore, relatively high takes in the check holes might mean that the foundation bedrock possesses some appreciable residual permeability which might have to be dealt with in future work.

4.5 Summary discussion and conclusions on Pournari, Assomata and Sfikia dams

In the preceding paragraphs of this Chapter the foundation conditions of the Pournari, Assomata and Sfikia dams were examined in detail. Certain comments were made and some conclusions drawn. Some of the predominant geological factors involved were identified and described in the course of the assessment of the water-tightness properties of the foundations. This was done by using as tools the records of geological observations and the permeability values up to the construction stage. Grouting results presented in the form of quantitative parameters do verify the importance of some of the above geological factors.

Grouting results also substantiate the importance of faults and joints as principal permeability conductors and locate some potential or certain leakage sources. Moreover, these results quantitatively clarify ambiguities concerning the influence of the geological factors on the permeabilities of the foundations. These ambiguities take the form of folds, fault lines, relaxations caused by faults, joints or bedding, weathering or construction. Thus, the grouting results indicate where bedrock voidness appears when caused by such factors, visible at exposure or invisible below ground. At the same time they assist in grading the effectiveness of the applied ground improvement, and in locating precisely the questionable areas, within the geometry of the curtain, which need further attention.

The grouting effectiveness in the succession of P, S, T, Q, E and C grouting holes determines whether additional grouting or other measures, such as drainage, are needed in order to safeguard the operational efficiency of the project.

On the basis of the above remarks and the information as cited in the previous sections, the following conclusions can be drawn concerning the Pournari, Assomata and Sfikia dams:

1) Pournari dam

The Pournari dam has been founded on stratified sedimentary rocks of flysch formations, and the dam axis alignment coincides with the strike of the strata which dip upstream. The river course at the dam axis was created by erosion of the strata which was facilitated by structural weaknesses (jointing) caused by secondary folds, the axes of which seem to coincide with the general bedding dip direction. No other major structural defects, such as faults, have been detected in the site. Jointing in the site has been developed in the stronger sandstone strata in sets of wide frequency and extension and in accordance with the regional fold of the Epirus-Akarnania syncline. This is due to the brittle strain behaviour of the thickly-bedded sandstone rocks on which geotectonic stresses have been exerted. Jointing in the thinly-bedded siltstone/sandstone series appears closely spaced and of small extension, while in the unstratified silty conglomerate local shearing and shattering of the rock is recognised. It seems that the strain behaviour of the less strong rocks (siltstone and silty conglomerates) is more ductile on application of geotectonic stresses.
Local variations in the jointing and bedding patterns in the site are due to the presence of secondary superimposed folds of the kind found on the riverbed.

No major discontinuities traverse the core trench.
A small thrust fault detected in the upper sandstone series has only a small displacement of 2.5m.

Weathering is more extensive on major joints and bedding planes, and appears to be deeper within the sandstone strata. Locally it is shallower in the intensely-jointed strata such as those at the apex of the anticlinal flexure, and in the rebound joints at the foot of the relatively abrupt abutment.

Grouting at Pournari suggests that:

i) Foundation bedrock openings, liable to promote substantial leakage, are restricted mainly within the upper sandstone strata which exhibit stronger jointing. These strata are relieved and exhibit a deeper weathering, sometimes as deep as the contact between the sandstone and the silty conglomerate series.

ii) The underlying rocks, although jointed, exhibit very low effective openings except in surface areas which are structurally weak, such as those at the apex of the anticlinal flexure or those at the foot of the left abutment. Weathering too, is found to be substantial in those weak areas.

iii) The higher directional propensity of grout takes is limited to the stronger sandstone-bearing series, so that the principal axes of permeabilities may be confined within the alignment of the sandstone formations.

iv) A properly designed drainage curtain was needed to control residual seepage and to eliminate its deleterious effect (pore water pressures) on the shear strength properties of rocks downstream of the curtain.

Thus, grouting results at Pournari reveal that only surface rocks have undergone a relatively high degree of relaxation which is expressed by the high values of the grouting parameters.

This relaxation, which has aggravated the pre-existing joint openings, has been caused mainly by the unloading of the upper strata due to erosion, weathering and rebound jointing, particularly where strong topographic anomalies exist. Deeper located rocks (a few metres from the surface) are relatively tight, and the grouting parameter values tend to be minimal. The above observations indicate that either compressional geotectonic stresses stored within the foundation rocks have not been fully released or that such stresses are at present in the process of being relieved.

It must be noted that highly-weathered rocks near the surface were excavated to reveal sounder rocks of at least grade III (in the Dearman et al., 1972 scale of weathering grades).

2) Assomata dam

The Assomata dam is founded on serpentinised ophiolitic lithological variations of rock, all of which traverse the core trench and are separated by faults. Other major local faults traverse the lithological units and the full width of the core trench. The above complexity of the lithological and structural evolution of the damsite strata has created strong differences between the two abutments and the riverbed. These differences affect both the watertightness properties of the foundation bedrock and its strength characteristics. The situation has been aggravated by deep weathering and hydrothermal alterations. A partial re-healing of open joints is also observed in the form of secondary deposition of calcitic or ophiolitic-calcitic veins, sometimes a few centimetres thick.

The permeability values, ranging up to 10^{-5} m/sec (only two out of a total of sixty six of them being 10^{-4} m/sec) appear to be in close relation with existing or assumed fault lines, and are higher in the left abutment. Transitional and sheared serpentinised agglomerates were proved to be relatively impermeable in comparison with tuff agglomerates. The confinement of an artesian water table of high pressure suggested the presence of an impermeable substratum the permeability values of which were in the range of 10^{-7} to 10^{-9} m/sec.

Before the excavation of the core trench no large voids were found, although minor dislocations had been assumed.

The above description of the rocks and their faults makes the discussion of recorded joints secondary. Such a discussion is required with respect to the segments where faults are absent.

Grouting results at Assomata suggest that:

i) The foundation bedrock voidness is mainly observed within the younger tuff agglomerate rocks, whenever they occur within the site.

ii) Although surface excavation of the core trench indicates a void width equal to or less than 4 centimeters, the size of the voids could be larger than half a metre.

iii) All tuff rocks within or adjacent to the low-angle thrusts exhibit a high voidness which is extended deeply into the left abutment. But the worst dislocation was that of the combination of the low angle thrust and the subvertical faults at the foot of the left abutment. All these faults are found to traverse the core trench.

iv) The tuff agglomerate rocks above the wide sheared thrust zone can be explained as an upthrust of those found at the foot of the left abutment. It is probable that this is a resultant movement of the low angle thrusts and the subvertical faults. The occurrence of this movement appears to have affected the whole abutment.

v) The depth of the highly disturbed zone terminates at the contact of the tuff agglomerates and the transitional agglomerates. As grouting progressed,sealing off the tuff rocks near the core trench surface (20m deep), check holes showed that the artesian horizon was leaking within the tuff agglomerates.

vi) The consolidation grouting, originally designed to reduce differential settlements within the core trench, was modified in the course of grouting works to reinforce (thicken) the grout curtain in all of the left abutment. This was done because of the extremely high grout takes absorbed by surface cracks in about a surface areaof 500 m^2 (57 257 litres of grout were poured into them). The high absorption of grout by the main curtain holes suggested the existence of such openings 20 metres below the core trench surface.

vii) Provision for further grouting works (in the left abutment) and well-designed drainage have been made as preventive measures against potential ineffectiveness of the grouting works.

viii) The river bed rock and the right abutment rocks proved to be quite tight. They were also found to absorb high quantities of grout only locally, that is, where the tuff agglomerates exist.

ix) An effective drainage curtain must be constructed, in spite of the intense grouting effort.
This is demanded by the poor shear strength characteristics of brecciated and altered rocks which are liable to be eroded by leakages.

It must be noted that deep excavation took place in the left abutment but could not be allowed to proceed any further in case it created instability in the spillway foundations.

3) Sfikia dam

The Sfikia dam is founded on metamorphic rock of meta-andesites and amphibolite-gneiss schists (undifferentiated) within the core trench excavations. Mixed facies of intrusive rocks of gabbros, amphibole or olivine-rich rocks, dioritic rocks form a dyke within the powerhouse fault. Within the magmatic rocks large erratic blocks of the metamorphic basement rocks have been found.

The main characteristics of the foundation bedrock are:

a) Relatively strong rocks with a consistent foliation (bedding/schistocity) found throughout the dam foundation, and dipping relatively steeply from the right to the left abutment so as to traverse the full width of the core trench.

b) Well-developed jointing in more or less orthogonal systems closely spaced but of small extension. The joints are usually open and clean, whereas locally they are partly infilled with clayey materials in moderately-to highly-weathered zones.

c) Three main local fault systems present at the riverbed and the abutments (apart from the power house fault and the small faults accompanying it which have been traced at the riverbed).

The one system is subparallel to the foliation and its discontinuities are spaced about 20-40 metres with persistent extension of several tens of metres. The other two systems are subvertical and oblique to the strike of the foliation of the formations. They are spaced 50-70 metres apart with an extension up to several hundred metres.All the three fault sets transverse the core trench.Weathering is extended deep along the fault lines, and faults usually appear to be filled with reddish or yellow-grey clay.

Surface formations appear to be more weathered.

This structural development of the site gives a water table, in both abutments, at river level. The permeability values are quite high (between 10^{-7} to 10^{-5} m/sec) and independent of the rock types, the depth, the RQD or the weathering.

The excavations for the core trench were shallow in comparison with the excavations of the other two dams.

Grouting at Sfikia suggests that:

i) The foundation bedrock voidness is mainly caused by the vertical faults.

ii) The highest voidness appears in the right abutment despite the impression that sounder rocks exist there. The high grout takes below the originally-designed limits of the grout curtain suggest that the foundation bedrock is subjected to tensional geotectonic stresses.

iii) High grout takes were registered in the riverbed and the left abutment (only at the formations near the surface and up to 20m deep) and at isolated intervals. This does not exclude the possibility of higher grout absorptions deeper down and near the power-house fault formations.

iv) The power house fault brecciated rocks in the areas traversed by the grout curtain exhibit low grout absorption mainly due to the clayey infillings present there and due to partial re-healing of the fault breccias (secondary re-cementation or diagenesis).

4.6. Relationship of the grouting behaviour and the foundation bedrock characteristics to the regional geological and structural setting.

The grouting behaviour of the foundation bedrock of the three damsites that were studied does seem to reflect the individual characteristics of each of the damsite locations. Further, it is suggested that the bedrock response to injection also reflects, to a degree, the characteristics of the general and regional geology of the zones to which these dams belong. Thus far, the grouting results have been discussed and placed in the context of the mean and extreme conditions encountered in each site. Hereafter they will be placed within the regional geological context which has been discussed in Chapter 3.

The main conclusions to be drawn here are as follows:

i) The Pournari grout curtain is built into the Ionian zone flysch formation and intersects three main lithological units, namely the thickly-bedded (upper) sandstone strata,the massive silty conglomerates (middle) and the thinly-bedded siltstone/sandstone sequence (lower).

It is aligned subparallel (NW to SE) to the strike of the beds, which generally dip (25^{o}-45^{o}) up-stream. No local or regional faults intersect the site. The structural characteristics of the foundation bedrock are controlled by the stratification of the beds and the fold systems existing there, which are also responsible for the structural and topographic genesis of the site.

The higher mean grout absorptions appear in the sandstone-bearing strata, and the extreme values are determined by local peculiarities, such as local major joints, slips, weathering, infillings.

The silty conglomerates and the siltstone/sandstone sequence are generally tight, but with local variations, such as at the apex of the river bed anticlinal flexure. Among the examples of grout curtains in flysch formations those of the River Moravia can be mentioned. They are considered to be quite permeable rocks

(Verfel, 1969).

In Greece (Chapter 2) in the leaking Kremasta grout curtain on Gavrovo flysch formations, the geological factors which resulted in those leakages are of interrelated local and regional character.

The existence of calcareous conglomerate horizons, within the grout curtain traced by local faults,relates to the emergence of sulphate hot springs.

The calcareous conglomerates are cavernous because of the solution caused by the sulphate springs.

Thermal springs exist in several areas of Greece along the separating lines of the major tectonic belts such as the one separating the Pindos and Gavrovon tectonic zones.

The above features are absent in Pournari and Kastraki dams which rest on Ionian flysch formations.But it is important to note the existence of small lenses of conglomerate within the sandstone strata of Pournari . and which have shown high weathering.

Finally, the predictions based on the local and the regional records and geotechnical considerations in the Pournari damsite concerning the watertightness of the foundation bedrock were proved to be satisfactory during the construction stage , and according to our expectations.

The grouting results coincide with the investigation conclusions on the watertightness of the site.

Small deviations proved to be due to the rock condition in the river bed. It was feared that there might be a fault in the river bed, and the over-conservative design of the curtain was caused by the fear that similar leakages as at Kremasta might occur here.

ii) At the Assomata damsite the controlling factors for determining the extreme values of the grouting results in some segments are the local and regional lithological and structural characteristics. The presence of tuff.   agglomerates marks the high voidness horizons of the foundation bedrock. These horizons occur in several locations within the horizontal and vertical dimensions of the curtain, as well as in other locations of the dam foundation.

Their genesis was argued (see Chapter 3) to have resulted from general and regional tectonic movement. In fact they appear to be less metamorphosed, as well as altered and weathered, than the surrounding ophiolites in which they have been embedded. Their disposition and voidness have resulted in high grout absorption. The nature and emptiness of both the surface and the deeper voids indicate that continuous movements take place there. This happens because the voids are only partially, or not at all, infilled, although they are surrounded by finer materials (clay and sheared-mylonitized rocks).

The above conclusion is confirmed by recent movements noticed in other areas surrounding the site. The nature of the movements are most probably tensional because of "graben" fault structures which are apparent in and around the damsite.

Efforts to establish evidence of recent geotectonic movements in the damsite area were conducted during the investigation stages, but failed to produce any results.

The ground investigations led to the discovery of the principal permeable formations in the site, but were proved to be incapable of providing the evidence of the extreme situation of wide voids which was discovered there afterwards. It is this extreme voidness of tuff agglomerate rocks and the hidden influence of the thrust faults that would suggest a probable alternative dam axis upstream of the adopted one.

Grouting results have so far indicated that the whole of the left abutment is located within two fault systems. The high grout takes of the quaternary holes and the check hole results indicate that the improvements which have been already been made were insufficient. Thus, additional measures, such as drainage, will have to be provided.

It is noted also that the evidence acquired for the Assomata damsite and the Sfikia damsite could be used for producing the geological map of the area which has not yet been undertaken. There are regional geological maps for areas north and south of these sites but no such map exists for the lower Aliakmon valley and the surrounding areas.

iii) Although the grouting works have not progressed as much as those for the Assomata curtain, some conclusions can be drawn concerning local conditions and their interrelation with the regional geology.

As stated earlier, high grout absorptions in the Sfikia damsite are concentrated along the vertical fault lines.
In the case of the right abutment they have been assisted by the shallow deep thrusts (dipping upstream in a north-east direction and related to the main tectonic thrust between the Pelagonian and the Vardar zones) in further relaxing the upper strata. These fault lines are oblique to the riverbed erosion line at the dam axis, the riverbed erosion line having been created by local faults parallel to the powerhouse fault. As other researchers have stated (Harper, 1972; Snow, 1965) such oblique discontinuties, which exhibit little or no displacement and very little accompanying gouge and clayey infillings, constitute principal conductors of permeability. This phenomenon contrasts with the main faults, such as those parallel to the powerhouse fault, which are the main tectonic elements and indicate clayey infillings and highly brecciated rock gouge, and thus considerable displacements.

These oblique local fault systems have to be considered as post-orogenic events created passively by secondarily-released strain energy stored within the rocks during folding of the basement rocks. Primarily, the area was relieved by the vertical and horizontal breaks of the basement rocks. These breaks were accompanied by thrust movements during the paroxysm phase of the successive orogenic stages (Alpine and post-Alpine) and thus gave rise to the complexity of the tectonic history of the region. The gouge and reddish clay observed along the powerhouse fault, as well as the intrusive rocks, indicate the considerable displacements which have taken place along such faults.

The relationship of the cement-absorbing structural features to the strike and dip of the schistocity of the formations, and the subparallel shears to the foliation, suggest that these local faults have resulted from deformations of flexural flow and flexural slip (Price, 1966) on the hinge and the limb of a regional fold hidden below the Sfikia village terrace. The axis of the fold is trending SSW to NNE and is subparallel to the strike of the powerhouse fault.

4.6.1 Predictions of ground improvement in future damsites in similar regional geological settings to those of the Pournari, Assomata and Sfikia damsites

In the preceding study of the grout curtains of the Pournari, Assomata and Sfikia dams, the voidness of the formations caused by several geological factors was examined. It became apparent that most of these factors were recognized in the investigation programmes conducted up to the design stage. It was found, however, that anticipation of real bedrock voidness cannot be specified even on the basis of evidence from the permeability test results. The nearest approximation could probably be achieved by evaluating similar foundation conditions within the range of the averaged grouting parameters observed in the three dams.The grouting parameters presented in the previous sections expose the particular responses to grouting of the three different foundation bedrocks in which similar grouting methods have been adopted with the aim of reducing the different permeabilities of each foundation down to a common value of 10^{-7}m/sec or less.

It is worth noting that the extreme values of grout absorption in isolated grouted intervals differ substantially and cannot be used for specification of voidness. This is clearly indicated by the fact that in the Pournari dam the extreme values of permeability in isolated intervals were in the range of 10^{-5} m/sec to 10^{-4} m/sec and gave several hundred kilograms of cement absorption, while the same range of permeabilities in isolated intervals at Assomata and Sfikia gave several thousand kilograms cement absorption.

CHAPTER 5

Conclusions

The studies described in the preceding chapter were conducted to investigate those geological factors which influence the watertightness of a given damsite in a given geological environment.The geological factors and their importance in creating the foundation bedrock voidness, the grouting data and the geometry of the grout curtain of three damsites in dissimilar geological environments, the regional and local geological settings were studied and the grouting result variations within the same curtain and between the three curtains were related to the significance of the major geological factors present at each of the damsites.

The writer has fully participated in the design studies and the preparation of the contract documents for the three dams under study (Pournari, Assomata and Sfikia) and he is in charge of the construction supervision of the Assomata and Sfikia grout curtains. Grouting is, at the present time, very much an art and depends upon the intuitive perception of the team in charge.

To avoid misinterpretations of any foundation conditions and bias on a decision making level, judgement in preparing grouting specifications must be flexible and be guided by an informed appraisal of the geological conditions requiring the foundation improvement.

A series of geological records, geotechnical test results and the curtain grouting results have been examined. The main variables considered were the lithology, the structural features, the degree of weathering and alteration, the depth, and the construction process. It was hoped, by examining this rather limited number of variables, to isolate the main geological factors governing the foundation bedrock voidness which has resulted in high grout absorptions and which hence affects the watertightness (as expressed by the permeability) characteristics of the damsites.

Similar grouting methods have been applied to the three grout curtains which rest on different foundation rocks in three different geotectonic zones. Thus, some interpretation of the grouting results in the context of the foundation rock structure is possible, and may allow some extrapolations for use in comparable prospective damsites in other areas of Greece. Some comparisons with previous similar cases have also been made in order to determine the degree of correlation.

The following major conclusions were reached, some of which have been summarized in some detail in Sections 4.5 and 4.6 as well as in the summary sections of the preceding chapters.

1. Effect of different lithological units (stratigraphy) within a site or different sites upon the grout takes

By examining the variations in grout absorption of the curtain segments of a particular dam, or the grout absorption between curtains of different dams, the effect of the various lithological units (sandstones, siltstones, tuff agglomerates, serpentinitic rocks, meta-ande sites, gneissic schists and the others) on the ability of the foundation bedrock to absorb grout was clearly demonstrated.

Well-stratified and stronger rocks exhibited higher grout absorption. The individual voids in them were wider than the voids in weaker rocks. Geological factors of a regional character, such as regional folds or faults, have caused the creation of grout-absorbing features in the rock mass. However, local factors, such as relative depths of the individual strata, topographic anomalies, small flexural structures and weathering, influence their intensity.

The pre-construction strain behaviour of the rocks depends on the in situ geotectonic stresses which, in relation to other features such as the extent and thickness of the strata, determine the formation of the principal permeability conductors of the foundation bedrock. These are the main grout-absorbing features within the geometry of the curtain.

Thus, at Pournari, the sandstone bearing series contained the main grout-absorbing elements of the foundation.

At Assomata, the tuff agglomerates and the stronger serpentinitic rocks contained the main elements.

At Sfikia (with undifferentiated rocks) the most absorbent strata were those which exhibited strong structural defects without considerable displacements, weathering and fines (gouge).

2. Effect of the different structural defects upon the grout takes

The main sources of high grout absorption in the three cases studied in detail were found to be the structural features such as faults and joints and the contact zones between different lithological units. Their form and nature, as expressed by an ability to absorb grout, is mainly dependent upon: a) the regional lithological and geotectonic characteristics which control the site, b) the local details of the lithology and structure and, c) the "site-creation" characteristics, such as weathering, erosion and relaxation, which in turn determine the geomorphology and topography.

The Pournari dam grouting results showed that, because of the absence of faults, the principal grout-absorbing features were the joints of the sandstone bearing strata, and particularly the dilated joints in the upper strata which had undergone higher relaxation and weathering. Sub-vertical rebound joints caused by unloading of the strata in association with strong topographic profiles exhibited some relatively high grout absorptions and suggest that horizontal stresses stored within the site rocks are not yet fully relieved.

The anticlinal flexure showed some concentration of dilated and grout-absorbing joints.

Bedding planes and contact zones have also acted as stress relief elements but were grout absorbing only where they exhibited weathering. The parallel alignment of the grout curtain to the strike of the strata, which strata dip upstream, eliminated from the curtain many of the principal permeability conductors.

The Assomata dam grouting results showed that the key factor controlling the acceptance of grout was the presence of regional faults. The combination of sub-vertical faults and the tuff agglomerates gave extremely high grout absorptions and suggested the existence of tensional geotectonic stresses which dilated the sub-vertical faults in a small dome-like anticlinal flexure within the tuff agglomerate at the base of the left abutment.

The subvertical faults seem to be subparallel to a regional graben fault structure, which occurs 500 metres upstream, and which appears to have affected the site. The thick sheared zone did not exhibit any high grout takes, but it allowed the stronger strata to dilate.

Any small leakages might create erosion and perhaps piping of the extremely fine alteration products (talc, asbestos, chlorite) present as bands in the rock.

The Sfikia dam grouting results showed that although large regional faults trace the site, the main grout absorbing features are secondary faults and joint sets which show minimal or no displacement.These faults and joint sets seem to have resulted from flexural flow and flexural slip deformations on the hinge and the limb of a regional fold concealed below the Sfikia village terrace. The main fault and joint systems act as stress dissipators, allowing these minor local faults to dilate. It is likely that the clay gouge occurring within the main faults acts as a water barrier.

The very deep high grout takes indicate a totally relaxed terrain in which tensional geotectonic stresses are still relaxing. A tensional geotectonic stress regime is also suggested from the earthquake analyses of the region (Papazachos,1979).

The faults subparallel to the foliation of the strata, as well as the foliation itself, did not show any significant openings along their development. However, some shearing along them reinforces suspicions of the existence of a fold deeper in the right abutment.

Along the foliation and shear zones, certain weathered areas seem to have inhibited effective grouting.

3. Influence of weathering

Weathering influences the groutability of the foundation bedrock in two ways. It can stress-relieve the rocks, thus adding to the effective openings of joints, or destroying the texture and reducing the strength of the rocks by producing fines which finally fill existing openings.

In both cases the existence of weathered materials acts against the effectiveness of the grouting. As noted above, the slightest seepages from poorly grouted voids could lead to erosion of the weathering fines, so creating piping. In the Assomata and Sfikia sites weathering posed difficulties in drilling and sealing-off surface rocks in particular.

The effectiveness of the grouting of weathered and sheared zones was considered doubtful and required intensive and close-spaced drilling. This added to the costs disproportionately to the effectiveness achieved. In Pournari most of the weathered surface rocks, particularly the weathered silty conglomerates and siltstones, were excavated. Karstification, observed in the limestone series in the hills upstream of the Assomata site, created fears of potentially high leakages and affected the decision to select the adopted dam axis. Karstification is not dealt with in this study, since it was absent from the three damsites,but it does exist in other regions where dams may be built and is a matter of great engineering concern.

4. Effect of bedrock relaxation produced by tectonic phenomena and by engineering construction upon grout takes

The most grout absorbing features at the sites studied were the joints and local faults produced by tectonic phenomena.

In Assomata, and particularly in Sfikia, joints and faults created by post-tectonic energy release of the deformed strata (stress relief) do not exhibit substantial displacements and appear free of infillings. Weathering has not yet proceeded to a sufficiently advanced stage to decompose discontinuity walls. The great depth (over 70m from ground surface) at which high grout takes are observed, even below the originally designed depth limit of the grout curtain, imply that the geotectonic stress regime of the Assomata and Sfikia sites is tensional, and total relaxation of the strata has promoted bedrock openings and their interconnections. Any build up of stresses seems to be dissipated by the regional fault movements.

In Pournari only surface rocks appeared to be highly relaxed, as the grouting results have suggested. The rebound joints observed in abrupt cliffs and in strong topographic anomalies imply that high horizontal stresses are still important in the site. Shearing and small displacements within the deeper rock of the abutments suggest that stress relief processes are taking place.

High grout takes in the treated rock caused by construction action (tunnels, excavation) were found to be of very limited importance, and were observed in very rare instances.

5. <u>Interrelations of the lithology, structural defects, weathering and relaxation of the foundation bedrock and the grouting results</u>

The grouting results examined earlier referred to non-karstic foundation bedrocks.

The results exemplify the interrelation of the above factors and show that lithologically stronger rocks are deformed in a brittle manner forming well-defined wide and high grout-absorbing joint or fault sets. The lithologically weaker rocks are deformed plastically in a ductile manner and the joints and faults are less discrete, less persistent, more closely spaced, and absorb less grout.

The degree of relaxation induced in the stronger rocks reflects wider effective openings amenable to grouting; the weathering is mainly concentrated in those faults and joints. In the weaker rocks, joints and faults appear less open, absorbing less grout, but the grouting effectiveness must be considered with great caution.

The high grout absorptions are usually concentrated along major joints or faults, and in particular on the steeper ones. High absorptions at depths such as those observed at Sfikia indicate that the geotectonic stress regime is in tension, and faults and joints which exhibit little or no displacement due to an absence of weathering products, absorb high quantities of grout. In Sfikia, secondary faults such as those described in earlier paragraphs have created greater openings than the main ones, but the main ones, which are regional, are major stress dissipation features, exhibiting considerable displacement and clayey products. Strong fault zones with considerable clay gouge may act as water barriers; an example is the power house fault at Sfikia and the main left bank fault at Assomata.

Relaxed formations, which are closely spaced and have joints or faults of characteristically high grout absorption, may indicate the existence of folds, such as that observed in Pournari or those in Assomata and Sfikia.

6. The deviation of grouting results from the specified expectations of sealing efficiency

Deviation of the grouting results from what was expected can arise because of two reasons. One reason could be mistakes in carrying out accurately the schedule of mechanical work to seal the foundation bedrock. The other reason is the unexpected foundation response to the treatment because of the abnormal foundation geology.

The first reason falls within the satisfaction of the contract conditions and is not the matter of this study.

The second reason is the subject-matter of this work. Examples of unforeseen defective foundations have been reported in Chapter 2. In the cases studied, Pournari dam did not show any particular deviations. The moderately high grout takes in the apex of the river bed anticlinal flexure and the rebound joints of the left abutment were even less than the design expectations. Strong interconnections between rock openings were considered to be the weathered parting surfaces, which had caused relaxation of the strata. But grouting results showed that major grout takes were observed only in isolated wide joints which resulted from the regional and local folds and the rebound joints, and that weathered infilling of discontinuity spacings has prevented effective grouting in some cases. It can be said conclusively that, although grouting was a major operation due to the high water head above river bed (100 metres), grouting results showed that a proper drainage curtain is crucial for the safeguard of the stability of the abutments. After the filling of the reservoir some water leakages (a few litres per minute) were intercepted by drainage holes mainly in the sandstone series of the left abutment.

In the Assomata dam, extreme grout acceptance conditions (worse than predicted by design assumptions) were revealed and it seems that these conditions extend to a great depth within the left abutment.

As a result, the designers were compelled to modify the depth limits of the grout curtain. In order to increase the number of blanket (consolidation) holes and to convert two rows (one upstream and one downstream) of the blanket holes into auxiliary curtain holes, they were compelled to increase the core trench width of the left abutment.

The grouting results showed that, apart from some local lithological and structural features, the regional ones assume the greatest importance. These regional features have created the adverse conditions and any future defective response of the site will be controlled by such features. It is very important to understand their role in creating the site conditions and to arrange suitable measures for combatting any future catastrophe resulting from their behaviour. Some defensive measures have already been adopted during construction, as well as arrangements for immediate action in the event of any emergency conditions. Long-term monitoring instrumentation has been installed in suspect areas.

In Sfikia dam, sets of secondary faults and joints proved to be the highest grout-absorbing features. The investigations up to the design stage showed, that higher relaxation had occurred in the left abutment, but grouting results to the present time have shown that the right abutment exhibits greater grout absorption overall and at depth than does the left abutment. The limits of the curtain extended deeper than those originally designed for both abutments because of these results. Moreover, the results show that the most permeable curtain will probably be that at Sfikia rather than at the other two dams.

The adoption of a rockfill type of dam rather than a competitively-costed concrete arch dam would seem to have been justified in view of the degree of voidness and both the intensity and depth of weathering in the abutment rock.

7. <u>Relationship of the grouting results, the previous information gathered on the site, the regional geology, and their implication in the planning of future damsites</u>

The effects of the regional geological characteristics of the tectonic zones and their influence on the foundation bedrock of the sites are, in general, reflected in the grouting results examined in the Thesis. The range of disturbance of each individual foundation bedrock is given by the range of quantities of the grouting treatment applied. The information gathered at the site during investigations leads to the same conclusion, but has failed to establish the peculiarities of each site. This is proved by the evaluation of grouting results and the modifications employed during construction and discussed in the previous sections.

These modifications consisted of "trial and error" experimentation in an effort to define the margins of safety against probable leakages through or beyond the curtain limits. These tolerances are finally tested after the filling of the reservoir.

It is known among grouting specialists that permeability test results cannot establish any satisfactory relationship between permeability and grout absorption. This deficiency is mainly attributable to the heterogeneity of the bedrock jointing and the different fluid properties of the water and the grout (Bruce, 1982).Research to establish relationships between the fracture porosities of a given ground and the permeability values (Snow, 1968; Koening and Heitfeld, 1964) by modelling the ground defects (fracture aperture networks and bedrock volume, see Reiss,1980), although found useful for reservoir engineering, is not so in grouting. The diversities observed in the permeability values and the grout absorptions in the three damsites do not offer any grounds for comparison. Only the averaged grout takes, when the structural heterogeneities of the ground have been improved after grouting and the fracture and weathering regimes do not differ substantially, can be related, within a reasonable range (one order of magnitude), with the permeabilities observed subsequently.

The above remarks could form a useful basis for an engineering geological zoning in comparable future damsites. The regional and local structural, lithological, weathering and geotectonic stress regimes of the new damsites could be compared with those of the previous ones. Future location and design of Greek dams should draw on the experience, outlined in this thesis, of interpreting the interactions between the geology, ground improvement, construction progress and subsequent dam performance.

5.1 Recommendation for further research

i) This study has been concerned with void volumes in the rock foundations of earth and rockfill dams. Water permeability and grout volume assessments have not expressly addressed the distinction between a large number of small open interconnected discontinuities and much smaller number of larger open discontinuities of equivalent total volume. It is the larger voids that pose the most serious problems and which therefore need to be identified. This study could form the basis of a new research programme.

ii) Although simple discontinuity models may not be validated by field experience, nevertheless they do help in a perception of the problem. In Appendix A a simple appraisal is made for the application of a cementitious grout. Appendix B considers the assessment of discontinuity permeability and the specification of packer spacings for a required density of discontinuity intersections within the packer test zone. This quasi-statistical technique is suitable for development with further research into discontinuity distributions,underpinned by the assumption that discontinuity occurrence in rock is a Poisson process.

iii) Concern for discontinuity presence may mask an understanding of the effects (both beneficial and adverse) of weathering. Clay gouge-either host rock or imported - acts to decrease secondary permeability but inhibits discontinuity intake of particulate grout.

More needs to be known about the hydraulic implications of rock weathering. Current classification schemes involving only geological descriptions need to incorporate geotechnical design implications.

iv) The dimensionless parameter "grout take per metre run per area of segment", as used in this study, has particular attractions for assessing the pre-impounding water tightness of the dam foundations. It is also applicable to local water permeability check assessments. There is a practical weakness in that grout expansion out of section (the segment area) is not taken into account. This shortcoming is of no consequence in the (unlikely) event of the discontinuity density distribution being homogeneous and isotropic throughout the segment. In practice this will not be so,and thus the results from this parameter will be distorted. An alternative grouting parameter, which may not then be dimensionless, is required to more accurately reflect the structural inhomogeneity and anisotropy of the rock foundation mass. This could prove to be a fruitful research area, with considerable concomitant practical benefits.

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APPENDIX A

Penetration of a cementitious grout

Let λ = discontinuity frequency (so-many per unit distance)

Where λ^{-1} = discontinuity spacing (say metres)

When $\lambda = 0$, fluid flow is through the intrinsic pore structure (infinite distance between discontinuities)

Let us assume that flow is along a bundle of capillaries (pipes) having radius r.

Also assume a Newtonian-flow fluid and assume laminar-flow.

Poiseuille equation:

$$r = \sqrt{\frac{8\eta k}{n\rho g}}$$

Where η = viscosity of fluid ($N.sec/m^2$)

k = permeability (m/sec)

n = porosity

ρ = density (kg/m^3)

g = gravitational acceleration (m/sec^2)

Assume that we are conducting a water permeability test.

Then, for rock,

$\eta = 10^{-3}$ $N.sec/m^2$ (water viscocity)

$k = 10^{-6}$ m/sec (say, permeable of rock mass)

$\rho = 1000$ kg/m^3 (water density)

$g = 9.81$ m/sec^2

$n = 0.3$ (rock porosity)

So, $$r = \sqrt{\frac{8 \times 10^{-3} \times 10^{-6}}{0.3 \times 1000 \times 9.81}} = \underline{1.6\ \text{microns}}\ (1.6 \times 10^{-6} m)$$

This test -at zero discontinuity frequency - shows that the rock is virtually impermeable.

Assume discontinuities are normal to borehole axis

" flow from a borehole is radial - laminar

" discontinuities have parallel-walls and are initially saturated

" that the above result is correct - that the rock material is rigid, inert, effectively impermeable to water.

Let flow along discontinuities be equivalent to laminar flow between parallel plates of opening width δ

$$\delta = \left(\frac{12\eta k}{\rho g \lambda}\right)^{1/3}$$

Suppose, now that we have a fracture frequency λ of 5 per metre and an apparent rock mass permeability of 7 x 10^{-4} m/sec.

Then

$$\delta = \sqrt[3]{\frac{12 \times 10^{-3} \times 7 \times 10^{-4}}{1000 \times 9.81 \times 5}} = \underline{555 \text{ microns}}$$

Let $\lambda = 7$ /metre, $k = 10^{-3}$ m/sec

$$\delta = \sqrt[3]{\frac{12 \times 10^{-3} \times 10^{-3}}{1000 \times 9.81 \times 7}} = \underline{559 \text{ microns}}$$

Let $\lambda = 10$/metre, $k = 2 \times 10^{-3}$m/sec

$$\delta = \sqrt[3]{\frac{12 \times 10^{-3} \times 2 \times 10^{-3}}{1000 \times 9.81 \times 10}} = \underline{625 \text{ microns}}$$

Thus, these three data sets (λ,k) suggest an aperture of $\simeq$ 0.6 mm. Thus, a cementitious grout (Opc) could be used at primary stage (dilute suspension for penetration ; Could use a soluble resin grout for secondary grouting).

APPENDIX B

Permeability testing

Before grouting-and after grouting-in order to assess the permeability of the rock mass, one conducts packer test using a double-packer system.

How does one decide what packer spacing to adopt? If the distance between packers is too small, then one risks not intercepting discontinuities and will so get an anomalously low permeability (k) value.

For the packer permeability test programme it is necessary to decide that it's required at least n% of the packer test lengths to contain m or more discontinuities. The geologist in charge will have to decide upon the values of n and m.

(That is, one fixes a packer spacing, say 3/4 metre. With this spacing one specifies that 95% of permeability test lengths should contain $\geqslant$ m (say 3) discontinuities. It would be unrealistic to say that one requires 100% of the tests to pump water through 4 or more discontinuities).

Let us say the following:

1) The RQD value for the rock mass is known (say 90%)
2) One assumes the discontinuity characteristics to be statistically homogeneous throughout the mass ie. a distribution function in one small area is replicated elsewhere.
3) The discontinuities occur along a given borehole in a random manner.

Given point (3), then after Priest and Hudson (see Attewell and Farmer 1976, pp 361-364) the occurrence is a Poisson process:

$$P\begin{pmatrix}\text{m discontinuities in}\\ \text{a length x}\end{pmatrix} = \frac{e^{-\lambda x}\ (\lambda x)^m}{m\ !} \qquad (1)$$

Where

P = Probability, m = 0, 1, 2, 3, . . ., λ = discontinuity frequency, as before, and ! denotes factorial

$$RQD = 100 \exp(-0.1\lambda)\ (1+0.1\lambda) \qquad (2)$$

(<u>see</u> equations 6.23, p 363 in Attewell and Farmer, 1976)

Where

$\lambda=1/\bar{d}$ in Attewell and Farmer.

Using RQD of 90%, and equation (2) above,

$$90=100\ e^{-0.1\lambda}\ (1 + 0.1\lambda)$$

One must now iterate (using trial values) to discover λ.

Trial values of λ		Resultant value of RQD from eqn (2) above
say 20	gives	40
10	"	73.6
5	"	90.97
4	"	93.8

So, one approximates well to an RQD of 90% by having λ=5 per metre

From equation (1) :

$$P\ (0 \text{ in } x) = e^{-\lambda x} \ \ldots\ldots\ldots\ldots \quad (3)$$

$$P\ (1 \text{ in } x) = \lambda x e^{-\lambda x} \ \ldots\ldots\ldots\ldots \quad (4)$$

$$P\ (2 \text{ in } x) = \left[(\lambda x)^2\ e^{-\lambda x}\right]/2 \ \ldots\ldots\ldots \quad (5)$$

$$P\ (3 \text{ or more in } x) = 1 - \left[P(0 \text{ in } x) + P\ (1 \text{ in } x) + P(2 \text{ in } x)\right] =$$

$$= 1 - \left[e^{-\lambda x} + \lambda x e^{-\lambda x} + \frac{\lambda x^2 e^{-\lambda x}}{2}\right] =$$

$$= 1 - \left[e^{-\lambda x}\ (1+\lambda x + \frac{\lambda^2 x^2}{2}\right] \qquad (6)$$

(a) We need P (3 or more in x) $\geqslant$ 95%

$$\text{or} \quad 1 - \left[e^{-\lambda x} \left(1 + \lambda x + \frac{\lambda^2 x^2}{2}\right) \right] \geqslant 0.95$$

We iterate to find critical value of x for λ=5 per metre:

Trial x	P (3 or more in x)
0.5 m	0.3236
0.75 m	0.7229
1.00 m	0.8753
1.20 m	0.9380
1.40 m	0.9704
1.30 m	0.9570
1.25 m	0.9480

So, we need a packer spacing of 1.25 metres (very nearly) minimum.

(b) Suppose that before performing all these calculations, one had performed some permeability tests. Say that 60 borehole packer permeability tests had been done with a 3/4 metre packer spacing. Of these 60 tests, 3 showed no discharge, indicating that the packer length contained no discontinuities.

Then,

Probability (0 in 0.75) = $e^{-0.75\lambda} = 3/60$

So, taking logs of both sides

$$\lambda = \frac{-\ln(3/60)}{0.75} = \underline{4 \text{ per metre}}$$

Iterate to find a critical value for x (with 90% RQD)

Trial x	P (3 or more in x)
0.5 m	0.3233
1.0 m	0.9267

1.2 m	0.8575
1.1 m	0.8149
0.9 m	0.6972

The nearest approach one can get is 1.0m packer spacing at probability of just over 90% that the test length will contain 3 or more discontinuities.

Calculation (a) above on a large sample of discontinuites gave a frequency of 5 per metre based on an RQD of 90%.

Calculation (b) above based on 3 zero discharge values out of 60 gave a frequency of 4 per metre.

This may imply that fractures have been introduced during the drilling and coring process (hence care is needed to "reject" new clean fractures from cores when assessing for RQD).

At low values of fracture frequency (λ), RQD is very insensitive to λ. Thus the usefulness of RQD as an estimator would seem to decline as the rock is less prone to fracture.

The lower apparent λ for the earlier tests (4/metre) may have been caused by closed discontinuities which, although intersected by the boreholes, did not transmit water.

APPENDIX C

Drilling method, grout injection method, pressures applied and completion criteria.

Three factors influenced the determination of the curtain depth and extension (into the abutments) of the Pournari, the Assomata and the Sfikia dams.

These factors were:the appreciation of the interaction between the local structural geology of the foundation rocks; the groundwater levels in relation to the topography; and the assessment of the packer pressure tests as well as the designed operation level of the reservoirs.

Thus the depth (see the "as built drawings" in the Supplementary Volume) of the Primary (P) holes in the Pournari dam was 50 metres. The Tertiary (T) holes were drilled 20 metres deep, and the Secondary (S) holes 30 metres deep.

In the Assomata and Sfikia dams again the depth of the Primary (P) holes were initially designed to be 50 metres but during the construction were extended 10 metres further locally. The Secondary and Tertiary holes were designed to be between 40 metres and 25 metres.

The above designed depth of the curtains it is planned according to the criterion (Simmonds et al.,1951):

S = 1/3 H+C where,

H = the height of the backwater (reservoir)

C = a constant varying between 8-25m as given by various authors (see Thomas, 1976; Verfel, 1969).

However, this sequence was not adopted strictly for the shallower holes (Quaternary holes also executed, particularly in Assomata and Sfikia dams), and they were sometimes increased even up to 50 or 60 metres, according to the conditions encountered (for details and particularities see, USBR Technical Memorandum 646, 1957; Houlsby, 1977 and 1982).

Diamond rotary drilling was employed for Pournari, Assomata and Sfikia dams, but at Sfikia dam the "down-the-hole" hammer drilling (see Gruse, 1979), was adopted later in order to speed up the grouting works.

The split spacing (or closure) method, in arranging the horizontal distance between the holes, was employed. This allowed the shortest horizontal distance between the two neighbouring holes to be 1,5 metres apart. This method also allows the increase of the depth of intermediate boreholes, in the sequence of Primary, Secondary, Tertiary and Quaternary holes to the depth required, if considered necessary, from the results of the neighbouring holes.

The grouts were mixes of Portland cement in varying proportions with water (see Table AC-1), with an addition of bentonite in a proportion of 2 per cent by cement weight. In rare cases, and where surface leakages of thick pumped-in grout were observed, sodium silicate and calcium chloride accelerator were added to shorten the gelling time of the mixes. Setting and flow properties of the grouts are shown in Figs. AC-1, AC-2 and AC-3.

Grout injection was carried out from the bottom of the holes upwards by setting a single packer at specified depths, the

TABLE AC-1

Grout mixes and weight of cement per unit volume of slurry

Water : cement ratio	Cement weight per litre of slurry
0.5 : 1 : 2*	1 216 kg/lit
1 : 1 : 2*	0.751 kg/lit
2 : 1 : 2*	0.430 kg/lit
3 : 1 : 2*	0.301 kg/lit
4 : 1 : 2*	0.231 kg/lit
5 : 1 : 2*	0.188 kg/lit

* Bentonite is included, mixed as standard 2 per cent by cement weight pre-activated (pre-hydrated).

Fig. AC-1 Grout mixes properties: Percent bleeding vs Elapsed time

At 23 °C

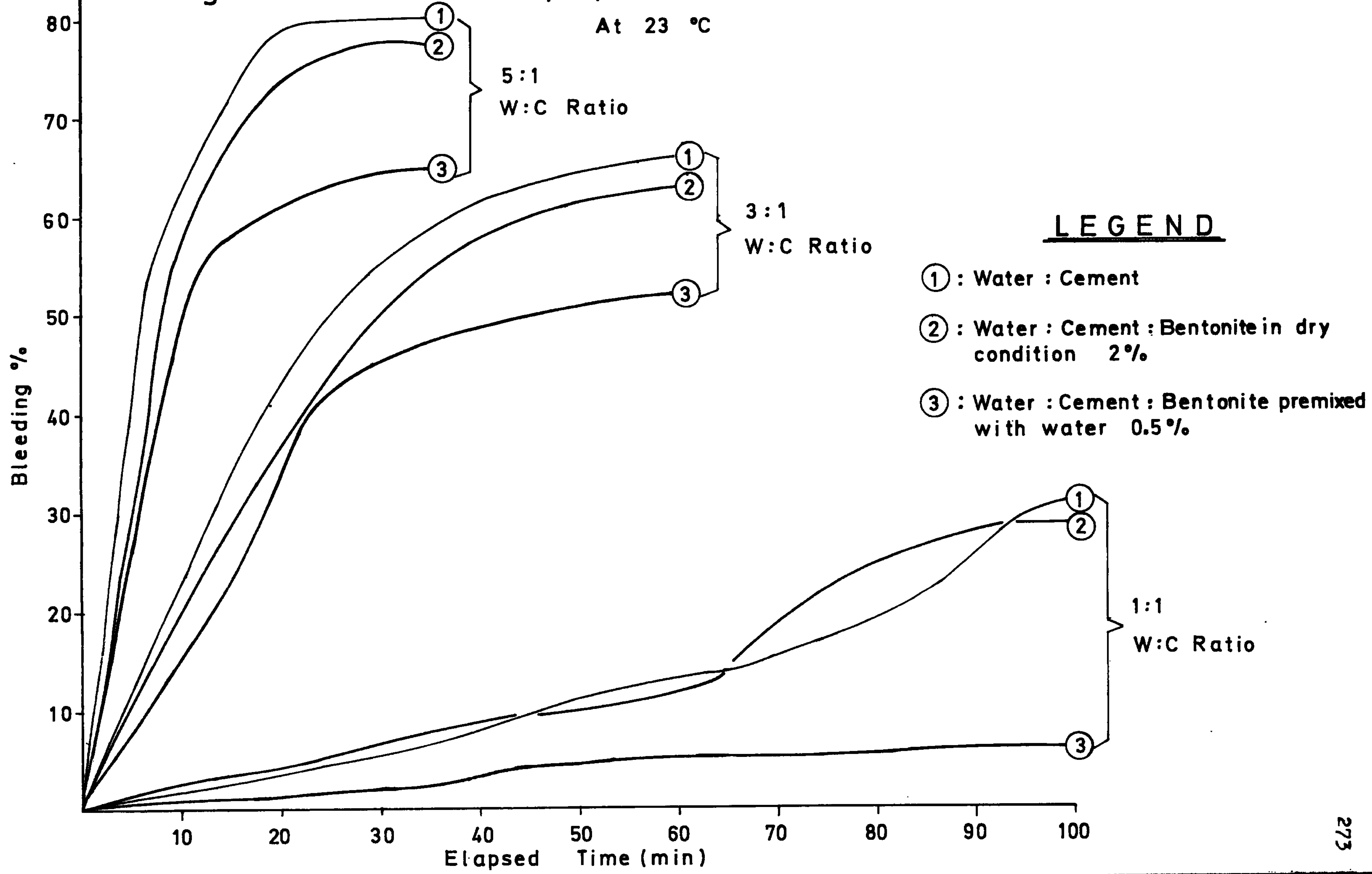
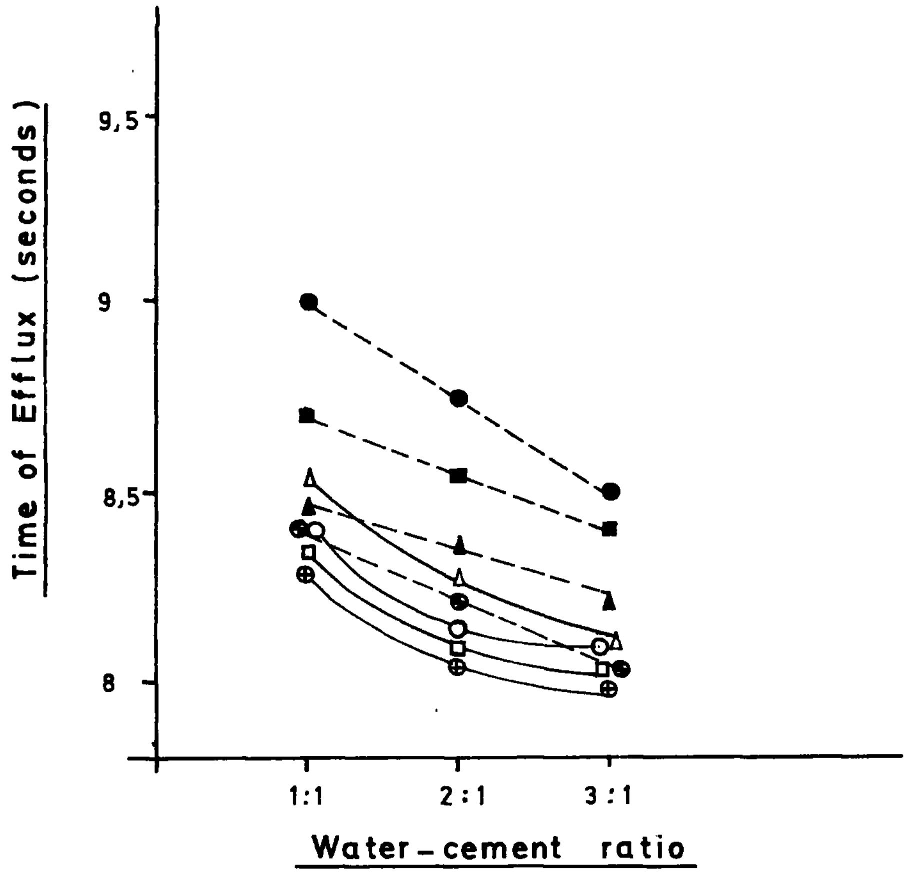


Fig. AC-2 Grout mixes properties: Time of Efflux vs water-cement ratio

Flow cone at 15 ± 1 °C



LEGEND

Open symbols : Bentonite premixed with water

Close ./. : ./. in dry condition

○ ● Cement 10% Poz. and 3% bentonite

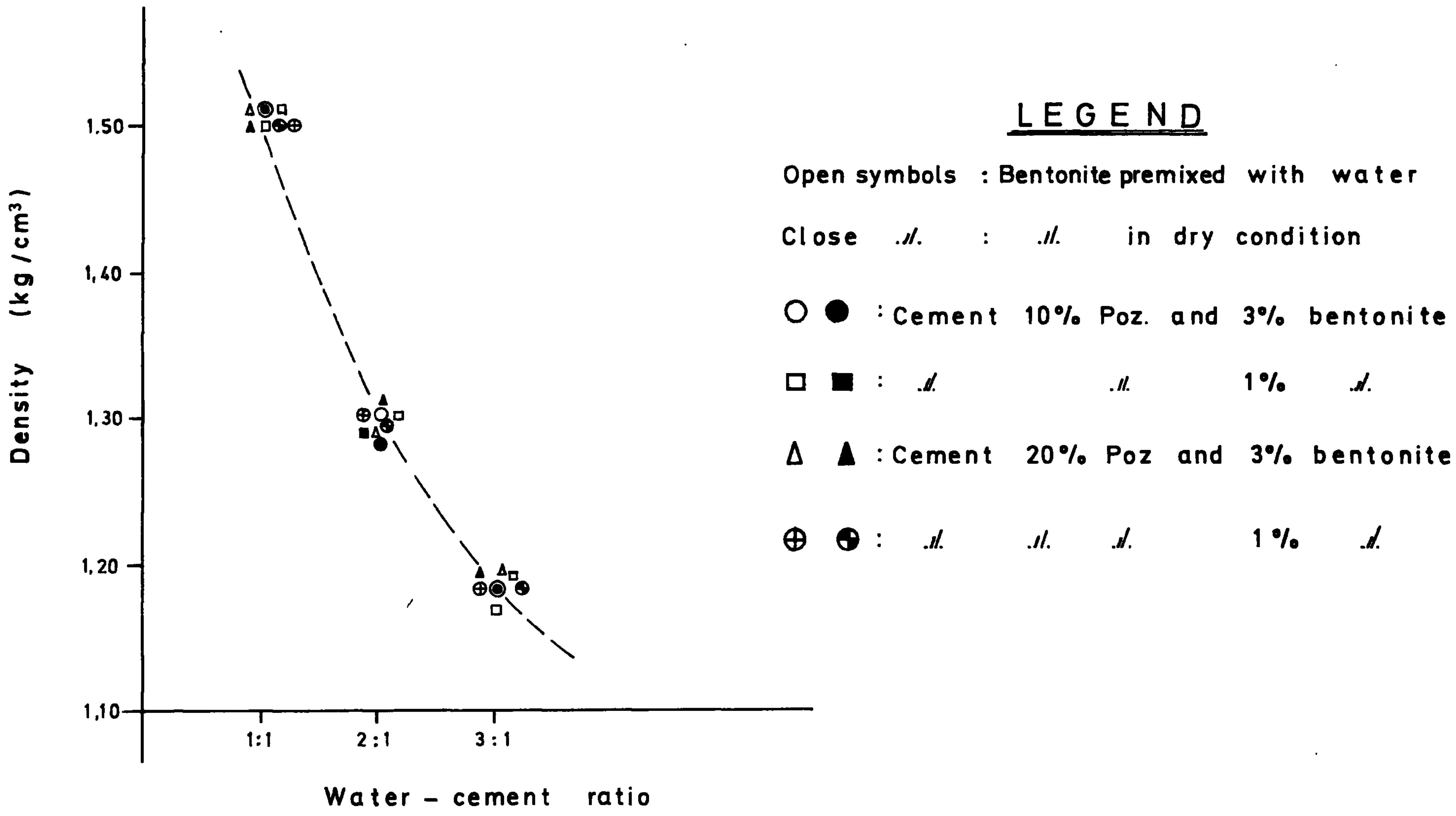
□ ■ ./. ./. 1% ./.

△ ▲ Cement 20% Poz. and 3% bentonite

⊕ ◕ ./. ./. ./. 1% ./.

Fig. AC-3 Grout mixes properties : Density vs water cement ratio

Density at 15 ± 1 °C



usual interval between two packer locations being five metres. The grouting holes were tested with clean water during the drilling of the borehole (packer permeability tests) for the same depths as for the grouting, with similar pressures as for the grouting. The down stage and zone grouting method (see Attewell and Farmer, 1976; Houlsby, 1977), although a possibility accomplished under the specifications, was not adopted (except rarely) because it was proved costly and time-consuming.

The pressures applied, as determined by the specification were 24.5 KN/m^2 (0,25 kg/cm^2) per meter of hole depth, did not in any way exceed the geostatic (overburden) pressures. At the near surface intervals (0,20 metres depth), the applied pressure for the Sfikia right abutment, did not exceed 39.22 KN/m^2 (0.4 kg/cm^2) gauge pressure, because of the unfavorable foliation of the strata and the near vertical minor faults joints existing there.

The design viscosity of one mix at the beginning of grouting depended on the water takes from the permeability test results, prior to grouting.(On this, see also Chapter 4).

Starting grout mixes (see also Scott, 1963, Cambefort, 1977; Littlejohn, 1982; Kutzner, 1982) were generally 5:1, or 3:1 (see Table AC-1). The standard procedure was to thicken the grout from 5:1, to 4:1, to 3:1, to 2:1 and 1:1 as grout takes became high. The grout injection was terminated when grout absorption was steady, 2 litres per minute, for ten minutes at the applied pressures.

The completion criteria used were as follows. Grout curtain closure was considered to be effected when grout takes were

50 kg/m for any stage grouted. If this criterion was not met for any stage the adjacent holes added later would be drilled and grouted, as required, until this criterion was satisfied.

At places, check holes were angled across the grout curtain and water pressure tested to determine permeability and evaluate the curtain's effectiveness. The check holes were grouted in a manner similar to other grout holes.

The drilling methods, the grout mixes and their flow properties, the pressures applied and the completion criteria have a bearing upon the groutability of the treated bedrock foundation. They provide the background knowledge for evaluating the grouting results achieved in the same or different grout curtains and offer a common base (if they are similar) upon which to compare the grouting parameters, as has been done in the preceeding chapters.

Their influence on the groutability of the foundation under treatment has been recognized through continuous laboratory and field research (Bowen, 1981; Houlsby, 1982; Albritton, 1982; among others).

The Pournari dam grout curtain covers an area of 56 264 m^2; 152 486 kg of cement was used, pumped-in through 12 659 metres of grouting holes. No blanket (consolidation) grouting was necessary for the core trench foundation bedrock.

The Assomata dam grout curtain, 90% completed (end of August, 1983), covers an area of 40 472 m^2; about 850 000 kg of cement was used pumped-in through 14 000 metres of grouting holes. Blanket (consolidation) grouting was used for the core trench foundation bedrock (see "as built drawings", in Supplementary

Volume).

The Sfikia dam grout curtain, only half completed (end of August, 1983), covers an area of 51 223 m^2 with about 807 000 kg of cement pumped in through 11 070 metres of grouting holes. Blanket consolidation) grouting was also used here for the core trench foundation bedrock (see "as built drawings" in Supplementary Volume).

In Assomata and Sfikia dams, about 6 000 m of blanket holes were drilled and 450 000 kg of cement pumped in to consolidate the trench foundation bedrock in each damsite.

The distribution of the absorbed grout quantities along the Sfikia curtain for the riverbed section was not substantially different from that at the abutment sections.

However for the Assomata dam the absorbed grout quantities observed along the curtain were uneven, with the left abutment, exhibiting the highest absorbed quantities than the riverbed and the right abutment. The riverbed and the right abutment absorbed minimal grout quantities.