IMPERIAL COLLEGE OF SCIENCE AND TECHNOLOGY

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SITE INVESTIGATION, ITS COST AND BENEFIT IN PORTUGAL

— 2 volumes —

volume II

by

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INTRODUCTION

In this volume, three different sets of information connected with the 26 projects studied are presented, as Appendices A, B, and C.

The Appendix A comprises 26 collections of data, one for each project. The first page of each contains information about the location, date of the design and design period, relevant characteristics of the project, and the names of both the owner and the designer of the project. The included construction costs are the budgets assessed at prices when the main design reports were released. The costs of the design, where quoted, comprise only the designs themselves ie., they neither include the costs of site investigations nor the costs of any other special works performed. But, they include the costs related to the engineering geological staff who worked either for the consultant who did the design or for the owner, where the designs were carried out directly by the owners. The task of the engineering geological staff (or geotechnical staff) is, as explained in Chapter V Vol.I, to perform the first site investigation (gathering relevant existing information and reconnaissance); to establish the programmes for the subsequent works; to supervise those works which, as a rule, are contracted out and, finally, to assess the results and produce the appropriate geotechnical assessment reports.

The costs for the designs represent the money actually spent by the owners to either pay the consultants who did the designs, or their own expenditure during the time that the designs were done by their own staff. In all cases the owners are Public Authorities.

Since the period of the designs varied very much and because when the designs were made by the owners they accounted the expenditures as the design progressed, the costs attached to them are not significant as a basis for comparisons due to the error introduced by costs varying

with time. The costs of the designs are therefore not used in any calculations, nevertheless it seemed they could constitute an interesting piece of information.

The introductory page of each project in Appendix A, ends with an estimated construction period, and the status of the project.

The information on each project in Appendix A, continues with a history of the site investigations performed and some general remarks which include a brief layout of the undertaking and other information considered useful (for example the mechanical parameters for the rock mass used in design). After that, there is a description of the general engineering geological conditions of the site which includes the rock type, structure, state of weathering and permeability. To complete the information, there is a detailed report of the site investigations performed during the different phases. This includes the date and period when the works were performed, the type of works and their specifications as well as their respective costs at the time they were undertaken. With regard to the mechanical characterization tests, these results are also quoted.

The author is convinced that the comprehensive information presented in Appendix A can, by itself, give a good picture of the Portuguese practice concerning rock mass site investigation for dams and tunnels. In addition, this information is a useful reference for future site investigations. As it is well known, when a given site is under investigation, the existing information about sites with similar geological conditions can be of great value and can even help save time and money. Although the results of the exploration works are not quoted in a detailed form (this would not be feasible for so great a number of projects), each project records what was done, which enables one to ask for further information when this is thought to be helpful.

The mechanical exploration test results had to be quoted in a consistent and easy to understand manner. For this reason, in some cases it was necessary to carry out calculations based on the individual results.

Appendix A is concluded with a sketch map of the geology of Portugal, a seismic intensity map of Portugal, a map of the average precipitation in the Portuguese hydrographic basins for the period 1954-55/ /1977-78, and a map showing the location of 24 of the 26 projects. Two of the projects are from the People's Republic of Angola, in Africa.

Presented in Appendix B (for each of the projects) are the marks

awarded for the benefit of each type of work performed during the site investigations, in terms of amount of knowledge of the geological and geotechnical conditions of the site that the investigation provided at the time it was undertaken. The significance of these marks and how they were arrived at, is explained in chapter IV, Vol.I.

The Appendix C comprises the cost of each type of work performed during the site investigations for each project at the time they were carried out, and the construction budget assessed at the time the design was released. In order to be able to make a comparison between the costs of the different works for a project and between different projects, it was necessary to convert all the costs to a common basis. It was decided to convert them at 1980 prices using the Bank of Portugal's price index, as explained in chapter V, Vol.I. Based on such converted prices, the total costs of site investigations and the relationship between them and the construction costs were worked out, for each project.

Although at the start of this research the author was awere that the task of gathering, treating and presenting all this information would be a very difficult one, the difficulties and the time required soon proved to be much greater than previously expected. The difficulties of this kind of work have also been mentioned by other authors -for example ATTEWELL and CLARK (1980).

First, it was necessary to gain access to both owners and consultants to assess the available information that could be of interest. After that it was necessary to conceive a plan for gathering and presenting the information in a consistent way for all the projects. This plan became more difficult to deal with as subsequent careful and detailed studies of the design reports were undertaken. Therefore, for obtaining some of the data, a subsequent step was undertaken, that of studying the reports of the contractores. Even after this, a greater number of gaps remained. These were gradually filled by patient personal contact with the owners, the contractors and the consultants. In some cases there was the need to contact personal colleagues actually involved in the site investigations to obtain data not readily available from the previously mentioned sources.

The difficulties increased when the assessment of the costs of each type of work performed during the site investigations was considered for each project. Only for a small number of projects was it possible to obtain those costs directly from the owners. In some cases it happened that a contract comprised either more than one type of works which had been accounted together or the contract dealt with more than one project.

In these cases it was necessary to seek help from the contractor.

For a number of site investigations, some of the exploration works (boreholes, trenches) were carried out directly by the owners. Obtaining the costs of these works, mainly for the boreholes, constituted a problem because they were accounted by the owners in a special way which gave rise to unreliable figures. In these cases, the author used prices in practice at the time the works were performed to assess the costs of the investigation.

Still regarding costs, the author wanted to know the cost of the engineering geological staff for each site investigation. This was not a very difficult task in the case of private consultants because they knew exactly how much it cost them and so the costs to the client could easily be assessed. The same did not happened when the owner's geological staff were involved, because in many of the situations the related costs were not accounted as a separate item. In such cases, help was obtained from the people who worked on those particular site investigations in order to evaluate the respective costs.

The costs for the design were also more difficult to obtain in cases where the owners themselves produced the design, and this was again due to the way the accounting system was organised. In some cases, more than one project had been accounted together and so, at the end, it was not possible to cost them individually. It also happened that sometimes the costs of the designs themselves were accounted together with other studies related to the designs. In the cases where the designs were performed by private consultants, it was possible to obtain their true costs, either through the owners who knew how much they payed out or through the consultants who knew how much they charged.

The author thinks that the owners should be encouraged to establish a type of detailed accounting that would allow the assessment of the cost of each item related to each project, in a way similar to the one used by the private consultants. This would also allow a comparison to be made of their own costs of identical works carried out by private contracting firms.

APPENDIX A

THE 26 CASE HISTORIES

DAM NAME — ALTO LINDOSO

— GENERAL DATA —

LOCATION	—	LIMA RIVER; 29 KM NE OF PONTE DE BARCA; PORTUGAL
TYPE	—	CONCRETE ARCH DAM
TOTAL CREST LENGTH	—	296.8 m
CREST CHORD LENGTH	—	252 m
MAX. HEIGHT ABOVE FOUNDATIONS	—	110 m
CREST CHORD / HEIGHT RATIO	—	2.3
MAX. HEIGHT ABOVE THE RIVER BED	—	105 m
WIDTH OF THE VALLEY (bottom)	—	≈50 m
RESERVOIR CAPACITY (total)	—	378.9 x $10^6 m^3$
FLOODED AREA	—	1 072 ha
DAM VOLUME	—	308.5 x $10^3 m^3$
DATE OF THE PROJECT DESIGN	—	JUL 1977
DESIGN PERIOD (Includes a prel. study)	—	1971-1977
OWNER	—	ELECTRICIDADE DE PORTUGAL
DESIGNED BY	—	ELECTRICIDADE DE PORTUGAL
CONSTRUCTION COST (project estimate)	—	1370 x 10^6 ESC
COST OF THE DESIGN	—	N.A.
CONSTRUCTION PERIOD (project estimate)	—	4 YEARS
STATUS	—	TO BE CONSTRUCTED

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigations were performed in 4 phases: phase 1 1968, phase 2 1968 - 1969, phase 3 1971 - 1972, phase 4 1976 - 1977.

1968	- Basic geological survey by field mapping
1968 - 1969	- A second phase of site investigations was undertaken, by means of boreholes and galleries. A preliminary geotechnical assessment report was produced in 1968.
1971 - 1972	- A new phase of site investigations by means of boreholes. A geotechnical assessment report was prepared and included in a preliminary study report of June 1973.
1976 - 1977	- Last phase of site investigations dealing with boreholes, trenches and laboratory tests. A final geotechnical assessment report was issued together with the design in July 1977.

A pre - design for a concrete arch dam located 650 m downstream of this one had been released in October 1962. Afterwards, the concept of the undertaking changed and topographic and economic reasons lead to investigate a new location. Geological reasons also played a part in the choice of the new site. In fact at the previous site the state of weathering was more intense, reaching greater depths (25 m in the right bank) and some structural features like unfavourable faults and joint sets were also present. The preliminary geotechnical assessment report made a comparison of both sites.

"In - situ" tests for mechanical characterization were not performed. The reason given was that the site studied in 1962 (B), which concerns the same geological formation, was tested both by loading and shear tests which enabled extrapolation of the results. Three tests of each of these types had been carried out at locations dealing with different sta-

tes of weathering.

The approach adopted to assess the deformability modulus for the actual site (A) was as follows: the relationship between the laboratory deformability modulus for the rock of both sites ($E_{lab\ A} / E_{lab\ B}$) was determined and it was assumed that the relationship between the "in - - situ" deformability moduli would be about the same. The average $E_{lab\ A}$ and $E_{lab\ B}$ values were 40 000 MPa and 30 000 MPa respectively; as $E_{field\ B}$ was 60 000 MPa, the value of 80 000 MPa was obtained for $E_{field\ A}$. Nevertheless the rock mass deformability modulus used in the design was 20 000 MPa, this is, equal to that of the concrete. The shear parameters adopted were C = 4 MPa and tan ϕ = 1.5, based also on the tests carried out at site B.

The dam is for irrigation and energy purposes. As safety controls there are two bottom discharges (armoured concrete conduits 2.55 m in diameter). An underground power house (73 m x 18 m x 17.5 m) driving 2x x 375 Mw and a 5.8 km long tunnel, which are not included in this study, complete the undertaking.

FOUNDATION CONDITIONS

1. Rock Type

Two differents rock types are present at the site - medium to coarse two mica granite, and micaschists (see Fig. Appendix A1). Special care was taken so that the dam foundations will concern only the granite.

2. Structure

Crossing the rock mass are aplite - pegmatite, quartz and dolerite veins, with the characteristics presented in the table below.

TYPE	STRIKE (o)	DIP (o)	WIDTH (m)
APLITE-PEGMATITE	N(15-36)W	SW or NE ALMOST 20°	0.05-4.0
QUARTZ	N(25-60)E	(50-80)NW	0.6 (max)
	N(25-45)W	66 NE to VERT.	
	N-S to N20E	SUBVERT.	
DOLERITE	N(41-56)E	(70-75)NW	1.0-1.8(max)

The main sets of joints, assessed in both banks, are:

STRIKE (o)	DIP (o)
N(25-50)E	60 NW TO VERT.
N(25-55)W	70NE TO 70 SW
N80E TO S80E	30N TO 30 S
N10W TO N10E	70W TO VERT
N(25-54)W	(10-38) SW

The assessed faults were grouped into four major sets:

STRIKE (o)	DIP (o)
N(26-60)E	(46-78) NW, SELDOM SE
N-S to N20E	62W to VERT.
N(16-40)W	(74-80) WSW
N(60-80)W	77 NNE

The fault spaces varied between 0.01 m and 0.5 m and the infillings are mylonitic in a clay matrix.

2. State of weathering

The rock mass shows, in general, a low degree of weathering near the surface and the more weathered zones are localised and related to faults, or due to water circulation through joints. It is expected that the depth needed for the excavation will not exceed 7 - 8 m in the left bank and 8 - 11 m in the right one. It is interesting to note that before the last phase of site investigations, excavation depths between 3 m and 6 m had been anticipated.

3. Permeability

According to the results of the Lugeon tests performed together with the boreholes, it was found that the impervious rock mass lies, sometimes at considerable depths. Some of those depths, in relation to particular zones, were refered to in the geotechnical report: 30 m under the right bank abutments; 71 m in a particular zone in the left bank; 30 m under the bottom of the valley and 30 - 50 m under the left bank abutment. Apart from these zones, the minimum depth expected to be grouted was 30 m.

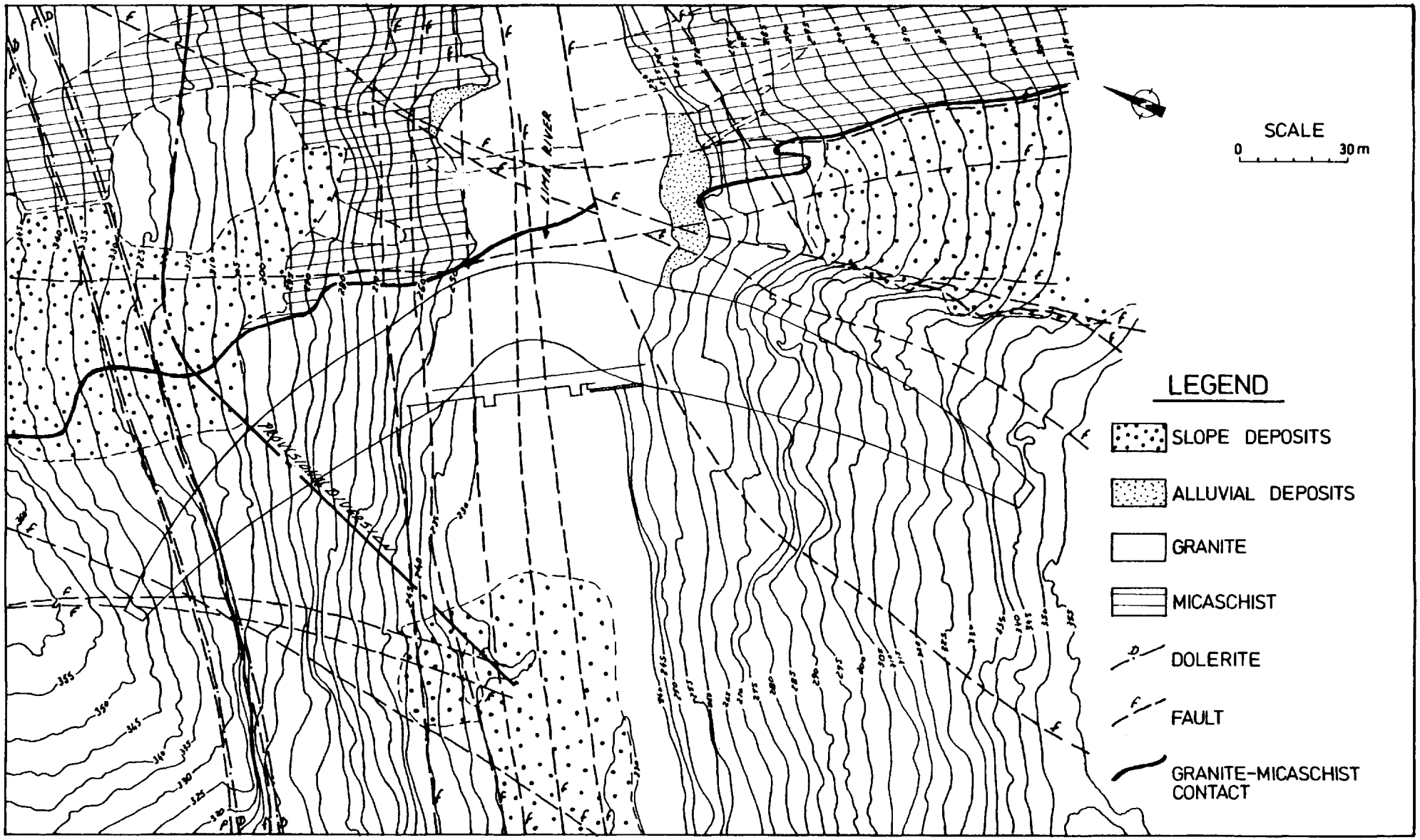


FIG. APPENDIX A1 - ALTO LINDOSO DAM SITE

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (1968)

1. Reconnaissance

PHASE 2 (1968 - 1969)

1. Boreholes (Jun - Oct 1968)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{(o)}$	NO OF LUGEON TESTS
RIGTH BANK	S 1 D	81.00	60	16
	S 2 D	9.39	45	2
	S 2 DA	50.30	45	10
	S 3 D	50.25	45	10
	S 4 D	55.50	45	11
LEFT BANK	S 1 E	70.46	60	14
	S 2 E	30.58	45	6
	S 3 E	40.20	45	8
	S 4 E	40.17	45	8

Total no of boreholes	9
Total length	427.85 m
Average length	47.54 m
Total no of Lugeon tests	85
Cost	300 x 10^3 ESC (1968)

2. Galleries

	DESIGNATION	LENGTH (m)
RIGHT BANK	G-1D	45.5
	G-2D	42.5
	G-3D	72.0
LEFT BANK	G-1E	47.5
	G-2E	31.5
	G-3E	55.0

Total no of galleries	6
Total length	294 m
Average length	49 m
Cost (assessed)	830×10^3 ESC (1969)

PHASE 3 (1971 - 1972)

1. Boreholes (Oct 1971 - Feb 1972)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{(o)}$	NO OF LUGEON TESTS
RIGHT BANK	S D 1	30.07	30	6
	S D 2	70.12	60	13
LEFT BANK	S E 1	30.00	45	5
	S E 2	25.00	45	3
	S E 3	25.00	45	5
	S E 4	70.06	60	14

Total no of boreholes	6
Total length	250.25 m
Average length	41.71 m
Total no of Lugeon tests	46
Cost (assessed)	190×10^3 ESC (1971)

PHASE 4 (1976 - 1977)

1. Boreholes (Aug 1976 ⟶)

DESIGNATION		LENGTH (m)	INCLINATION TO THE HORIZ$_{(0)}$	NO OF LUGEON TESTS
RIGHT BANK	S H 5	50.0	45	-
	S H 6	49.6	90	9
	S H 7	60.1	60	11
LEFT BANK	S H 1	50.0	45	4
	S D-2	30.0	60	5

Total no of boreholes	5
Total length	239.7 m
Average length	47.94 m
Total no of Lugeon tests	29
Cost (assessed)	400 x 10^3 ESC (1977)

2. Trenches (Feb - Mar 1977)

DESIGNATION	LENGTH (m)
SJ 1	19
SJ 2	13
SJ 3	9
SJ 4	14
SJ 5	5.5
SJ 6	4.5

DESIGNATION	LENGTH (m)
SJ 7	10
SJ 8	10.5
SJ 9	5
SJ 10	30
SJ 11	?
SJ 12	?

Total no of trenches	12
Total length (approx.)	145 m
Average length (approx.)	12 m
Cost	120 x 10^3 ESC (1977)

3. Laboratory tests (Apr - May 1977)

3.1 Uniaxial tests

Total no of tests 36

Type of samples: cut from rock cores

Diameter 52 mm or 42 mm

Aspect ratio 2.3 to 3

ASSESSMENT OF E

No of loading cycles 3

Maximum pressure applied 25 MPa

Results:

ROCK TYPE	E (MPa)			NO OF TESTS
	MAX	MIN	AVER	
GRANITE CLASSIFIED AS GOOD	56 300	25 100	42 800	28
GRANITE CLASSIFIED AS REASONABLE	40 600	19 800	28 400	7
DOLERITE CLASSIFIED AS GOOD	-	-	72 700	1

ASSESSMENT OF σ_{uc}

Tests performed on the same samples, increasing pressure up to failure.

Results:

ROCK TYPE	σ_{uc} (MPa)			NO OF TESTS
	MAX	MIN	AVER	
GRANITE CLASSIFIED AS GOOD	160	69	124	28
GRANITE CLASSIFIED AS REASONABLE	130	57	116	7
DOLERITE CLASSIFIED AS GOOD	-	-	222	1

3.2 Joint shear tests

Total no of samples 33

Type of samples: cut from rock cores

Area of shear 17.5 cm^2 to 42. 6 cm^2

No of tests per sample 4

Results:

c (MPa)			∅ (o)		
MAX	MIN	AVER	MAX	MIN	AVER
0.38	0.04	0.24	47	30	37

3.3 Rock shear tests

Total no of samples 12

Type of samples: cut from rock cores

Area of shear 21 cm^2

Results:

ROCK TYPE	c (MPa)	∅ (o)	NO OF SAMPLES
GRANITE CLASSIFIED AS GOOD	5.4	66	6
GRANITE CLASSIFIED AS REASONABLE	2.5	67	6

Cost of laboratory tests - 250 x 10^3 ESC (1977)

DAM NAME	—	ALVIDE
		— GENERAL DATA —
LOCATION	—	VINHAS STREAM; CASCAIS; PORTUGAL
TYPE	—	CONCRETE ARCH DAM
TOTAL CREST LENGTH	—	100 m
CREST CHORD LENGTH	—	66 m
MAX. HEIGHT ABOVE FOUNDATIONS	—	25 m
CREST CHORD / HEIGHT RATIO	—	3
MAX. HEIGHT ABOVE THE RIVER BED	—	22 m
WIDTH OF THE VALLEY (bottom)	—	20 m
RESERVOIR CAPACITY (total)	—	$0.8 \times 10^6 m^3$
FLOODED AREA	—	12 ha
DAM VOLUME	—	$5.2 \times 10^3 m^3$
DATE OF THE PROJECT DESIGN	—	OCT 1972
DESIGN PERIOD (pre-design)	—	1971-1972
OWNER	—	DIRECÇÃO GERAL DOS RECURSOS E APPROVEITAMENTOS HIDRAÚLICOS - LISBOA
DESIGNED BY	—	COBA, SARL- LISBOA
CONSTRUCTION COST (project estimate)	—	10×10^6 ESC
COST OF THE DESIGN (pre-design)	—	1.05×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	—	N.A.
STATUS	—	TO BE CONSTRUCTED

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigations for this dam were performed in four phases: phase 1-1971, phase 2-1971 - 1972, phase 3-1972 - 1973 and phase 4-1976 - - 1977.

1971 - Basic geological survey by field mapping , carried out in October.

1971 - 1972 - A second phase of S.I. was performed by means of one trench and some boreholes. A geotechnical assessment report was prepared which formed part of the pre - design issued in May 1972.

1972 - 1973 - A new phase consisting of one borehole and one gallery.

1976 - 1977 - A final phase of S.I. by performing geophysical exploration, boreholes, "in - situ" and laboratory tests. A final geotechnical assessment report was issued in June 1978.

Although the owner has still not ordered the project, no further site investigations are expected.

The dam will be built for flood regularization purpose and, according to the pre - design it will comprise a crest spillway and a bottom discharge.

The deformability modulus adopted in the 1972 pre - design was E=10 000 MPa with a Poisson coefficient of 0.22. "In - situ" and laboratory mechanical characterization tests were carried out in 1977, during the 4th phase of site investigations.

FOUNDATION CONDITIONS

1. Rock Type

The lithology at the site consists of limestones and interbedded marls of Cretaceous (Urgonian) age (see Fig. Appendix A2). The rock mass is overlain by a heterogeneous deposit of limestone boulders, quarry detritus and clay weathering materials. The stream bed is cut into a platform containing slope deposits and fine alluvium, 3 to 4 m thick.

2. Structure

The overall bedding attitute is subhorizontal.

The rock mass is highly jointed, the majority of the joints being subvertical ; there is a wide scatter in strike, which lies mainly between N 70° W and E - W and between N 10° E and N 30° E. No important tectonic failures which can interfer with the construction were found.

3. State of weathering

The rock mass is, mainly, slightly weathered to fresh. Some,but generally not very important dissolution features were noted at the surface, while exploratory works showed that some clay filled caverns occur at depth.

4. Permeability

All boreholes were carried out together with Lugeon tests, mainly in sections of 5 m.

At the bottom of the valley, the considered impervious rock mass was reached at depths varying between 15 m and 20 m, while within the banks that depth was assessed to be between 25 and 30 m.

A 20 to 30 m long grout curtain was suggested in the geotechnical assessment report.

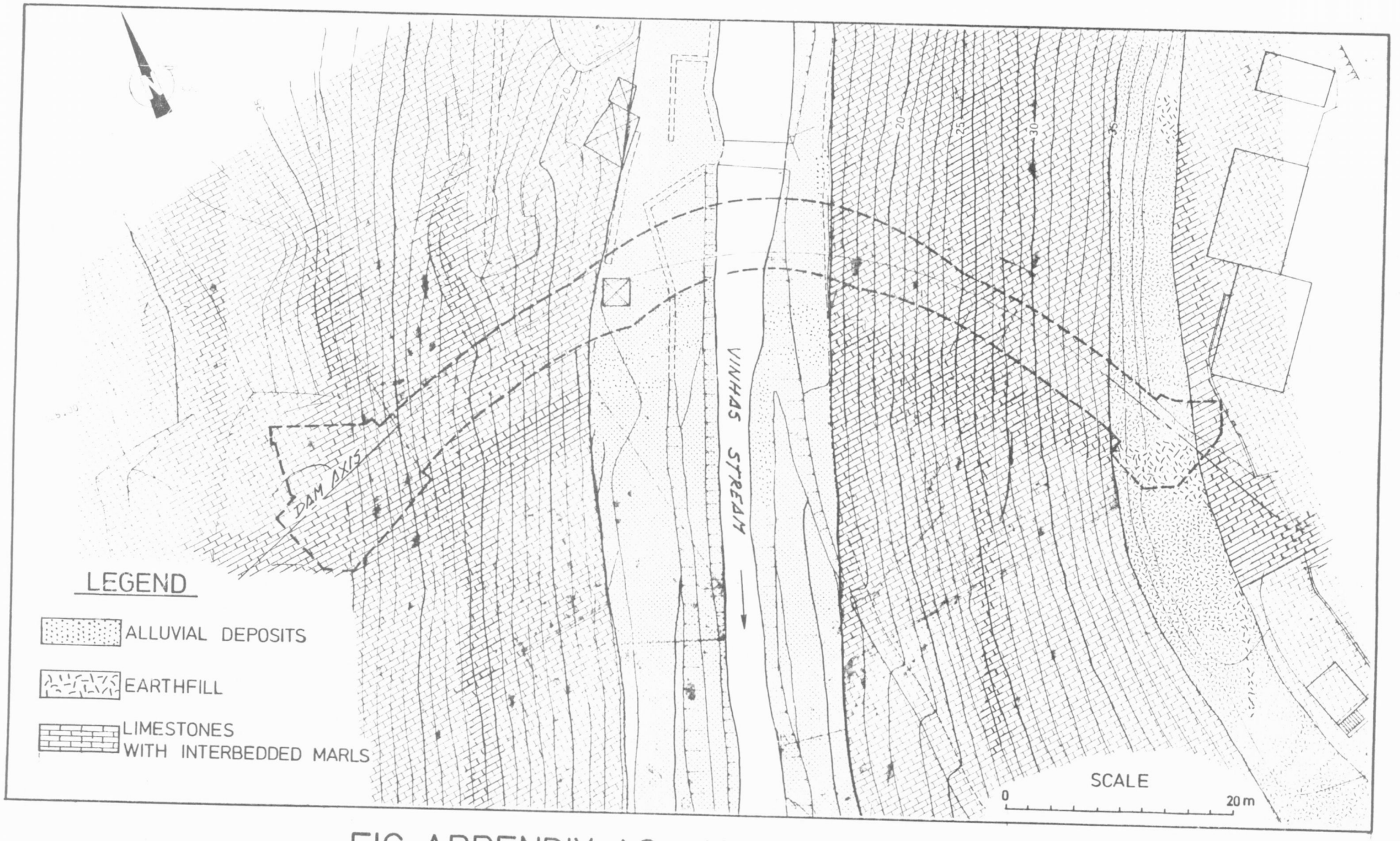


FIG. APPENDIX A2 - ALVIDE DAM SITE

SITE INVESTIGATIONS - DETAILS AND COSTS OF PERFORMED WORKS

PHASE 1 (1971)

1. Reconnaissance (Oct 1971)

PHASE 2 (1971 - 1972)

1. Trench

One trench along the dam axis, 95 m in length
Cost - 40×10^3 ESC (1971)

2. Boreholes (Nov 1971 - Jan 1972)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{(o)}$	NO OF LUGEON TESTS
RIGHT BANK	SAL-2	35.0	60	6
	SAL-4	30.5	75	5
BOTTOM OF THE VALLEY	SAL-6	35.0	60	6
	SAL-5	35.0	60	6
LEFT BANK	SAL-3	30.0	75	5
	SAL-1	35.0	60	6

Total no of boreholes	6
Total length	200.5 m
Average length	33.4 m
Total no of Lugeon tests	34
Cost	150×10^3 ESC (1971)

PHASE 3 (1972 - 1973)

1. Boreholes (Apr 1972)

DESIGNATION		LENGTH (m)	INCLINATION TO THE HORIZ.(o)	NO OF LUGEON TESTS
RIGHT BANK	SAL-8	20	90	3

Cost - 30×10^3 ESC (1972)

2. Gallery (Aug 1972 - Aug 1973)

DESIGNATION		LENGTH (m)
LEFT BANK	GAL	40

Cost - 380×10^3 ESC (1972)

PHASE 4 (1976 - 1977)

1. Boreholes (Oct 1976 - Mar 1977)

DESIGNATION		LENGTH (m)	INCLINATION TO THE HORIZ.(o)	NO OF LUGEON TESTS
LEFT BANK	SAL-7	25.0	60	6
RIGHT BANK	SAL-10	25.0	60	7
	SAL-12	30.2	60	8

Total no of boreholes	3
Total length	80.2 m
Average length	26.7 m
Total no of Lugeon tests	21
Cost	140×10^3 ESC (1976)

2. Geophysical exploration (Feb 1977)

2.1 Seismic refraction profiles

(direct and reverse shots, using a 12 geophone array).

DESIGNATION		LENGTH (m)
LEFT BANK	PS 1	81
	PS 2	75
	PS 3	52
RIGHT BANK	PS 4	72
	PS 5	59
INSIDE GALLERY	PS 6	55
	PS 7	61

Total no of profiles	7
Total length	455 m
Cost	(see below)

2.2 Seismic fans

Two seismic fans were carried out. Longitudinal wave velocities were measured in 12 directions, with shots inside the gallery and a 12 geophone array at the surface of the terrain.

Cost of the geophysical exploration - 30×10^3 ESC (1977)

3. "In - situ" tests (Feb - May 1977)

3.1 Dilatometer tests

Device used	BHD - LNEC
No of boreholes tested	3
Horizons measured in each borehole	4 - 5
Directions measured in each horizon	4
No of loading cycles	3 - 5
Maximum pressure applied	80 MPa (60 MPa in one test)
Total no of horizons measured	14

Results:

DILATOMETER MODULUS (MPa)		
MAX	MIN	AVER
46 200	22 800	21 600

Cost of the "in - situ" tests - 210×10^3 ESC (1977)

4. Laboratory tests (same period as above)

4.1 Uniaxial tests

Total no of samples 18

Type of samples: cut from rock cores

Diameter 54 mm

Aspect ratio 2.5 to 3

ASSESSMENT OF E

No of loading cycles 3

Maximum pressure applied 25 MPa

Results:

E (MPa)		
MAX.	MIN.	AVER.
88 800	11 000	66 000

ASSESSMENT OF σ_{uc}

Tests performed on the same samples, increasing pressure up to failure.

Results:

σ_{uc} (MPa)		
MAX.	MIN.	AVERAGE
164	37	116

4.2 Joint shear tests

Total no of samples 15

Type of samples : cut from rock cores

Area of shear 19.1 cm^2 to 50.4 cm^2

No of tests per sample 4

Results:

c (MPa)		Ø (°)	
MAX	MIN	MAX	MIN
0.36	0.07	41	22

The next lowest values for c and Ø were 0.13 MPa and 30° respectively.

Cost of the laboratory tests - 40 x 10^3 ESC (1977).

DAM NAME — ATROZELA

— GENERAL DATA —

LOCATION	—	PENHA LONGA STREAM; ATROZELA; CASCAIS; PORTUGAL
TYPE	—	CONCRETE ARCH DAM
TOTAL CREST LENGTH	—	95 m
CREST CHORD LENGTH	—	70 m
MAX. HEIGHT ABOVE FOUNDATIONS	—	23.75 m
CREST CHORD / HEIGHT RATIO	—	2.9
MAX. HEIGHT ABOVE THE RIVER BED	—	17.75 m
WIDTH OF THE VALLEY (bottom)	—	25 m
RESERVOIR CAPACITY (total)	—	$2.2 \times 10^6 m^3$
FLOODED AREA	—	12 ha
DAM VOLUME	—	$5.5 \times 10^3 m^3$
DATE OF THE PROJECT DESIGN	—	AUG 1977
DESIGN PERIOD	—	1975-1977
OWNER	—	EMPRESA PÚBLICA DAS ÁGUAS DE LISBOA (EPAL) - LISBOA
DESIGNED BY	—	HIDROPROJECTO, SARL - LISBOA COBA, SARL - LISBOA
CONSTRUCTION COST (project estimate)	—	29×10^6 ESC
COST OF THE DESIGN	—	2×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	—	8 MONTHS
STATUS	—	TO BE CONSTRUCTED

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigations were performed in three phases: phase 1 1972 , phase 2 1975, phase 3 1975.

1972 - Only a quick reconnaissance of the site and 4 boreholes to assess the behaviour of the rock mass at depth were carried out. After that, the site investigations stopped and were resumed in 1975.

1975 - This second phase comprised accurate field mapping.

1975 - Third phase of site investigations by means of boreholes and trenches. A preliminary study report was issued in December 1975, which included a geotechnical assessment , comparing different solutions: a concrete arch dam, a rock fill dam and an earth dam. A concrete dam was chosen for economic reasons and the project design was produced in August 1977, which included a final geotechnical report.

The undertaking will comprise the dam with a crest spillway, a bottom discharge under the dam body in the right bank, and a water intake, the purpose of the dam being for water supply.

Neither laboratory nor "in - situ" tests were performed. The reason for this was that local conditions were found to be similar to those of the Alvide dam site, 3 km away, for which both kinds of tests were carried out It is interesting to notice that at the time when the "pre - design" was issued in 1975, those tests had not yet been performed and then a deformability modulus of E = 20 000 MPa was adopted for the rock mass at A - trozela. In the project design, in 1977, and based on the tests results, that value was correct to E = 10 000 MPa; a Poisson coefficient of 0.22 was used. Stability calculations based on the rock mass shear strength were not made.

FOUNDATION CONDITIONS

1. Rock Type

The rock mass at the site, (see Fig. Appendix A3) deals with a very thick (500 - 600m) Cretaceous formation of limestones, at times marly , and with interbedded marls. The thickness of marl and limestone layers varies from a few centimeters to 4 m and between 0.5 and 2 m respectively. Some evidence of dissolution was found, which did not seem important. The stream flows along a heterogeneous alluvial platform of clay matrix, 3 to 4 m thick. The slope deposits are generally thin at the site and the rocks crop out over extensive areas.

2. Structure

The strata attitude differs slightly from one bank to another, being around N 60^{o} E; 85^{o} NW to 70^{o} SE in the right bank and about N 80^{o} E; 75^{o} S to subvertical in the left bank. This difference may be due to a fracture zone which is suggested, together with some others, from aerial photographs; no major defects were confirmed during the studies. Schmidt's polar diagrams for the joints in each bank are presented in Fig. Appendix A3.

3. State of weathering

Although the rock mass is quite jointed in the first 10 m below the surface, it shows a low degree of weathering. Percentage of core recovery was almost 100%. A depth of about 2 - 3 m was estimated for the dam foundations except below the stream bed were it should reach 5 m, but increased to 6 m for geometrical reasons.

4. Permeability

The rock mass exhibits high permeability evidenced both by the tests performed and by the variation in the water table depth between the dry and wet seasons. A grout curtain was thought indispensable down to depths where the permeability coefficient assessed by Lugeon tests be lower than 3.8×10^{-5} cm/sec.. This will need to reach depths of about 20 m and 40 m under the dam, in the right and left bank respectively.

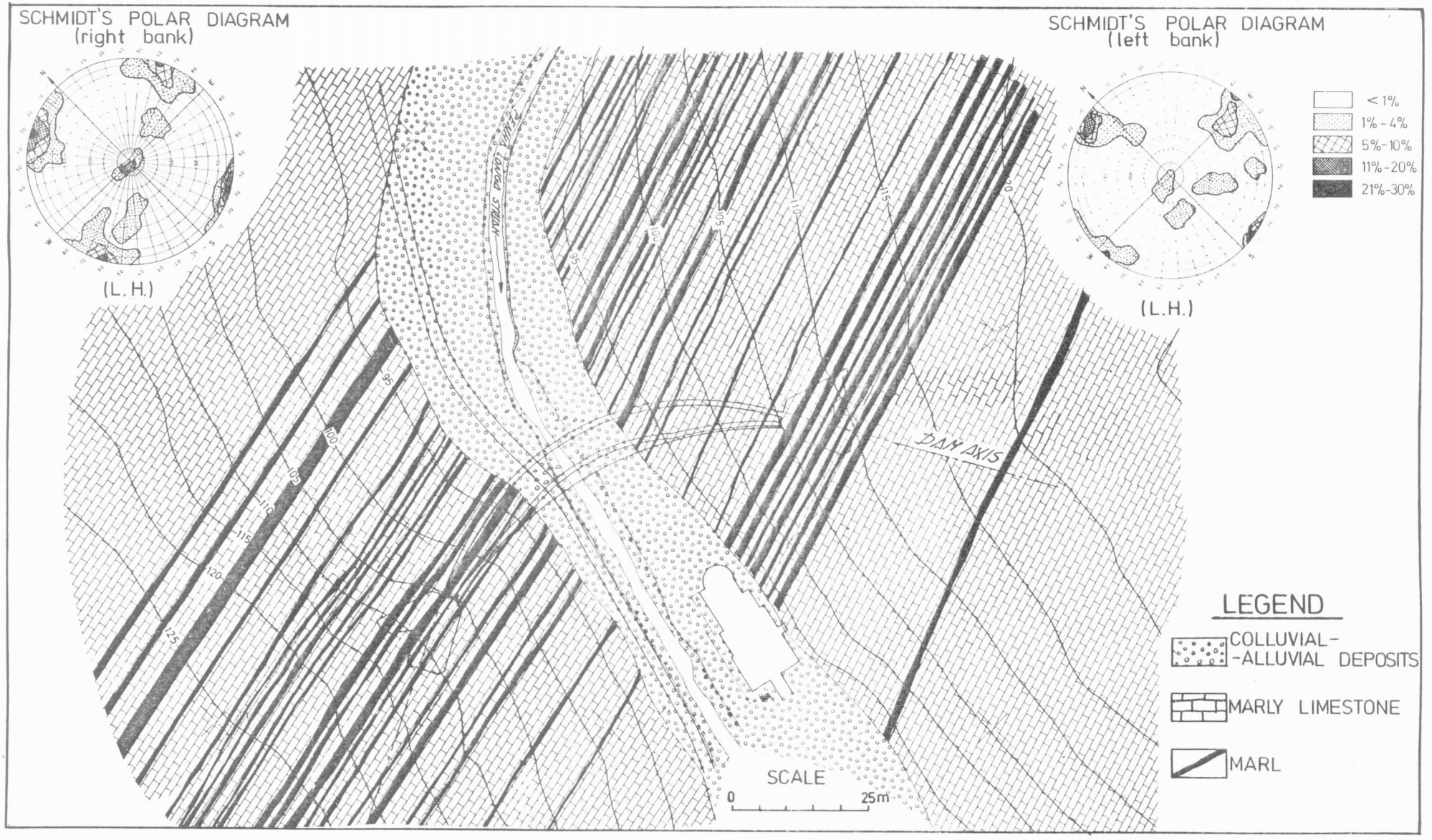


FIG. APPENDIX A3 – ATROZELA DAM SITE

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (1972)

1. Boreholes (Feb - Apr 1972)

DESIGNATION		LENGTH (m)	INCLINATION TO THE HORIZ.(o)	NO OF LUGEON TESTS
RIGHT BANK	SAT- 2	40	60	7
	SAT- 4	35	60	6
LEFT BANK	SAT- 1	55	60	10
	SAT- 3	55	75	10

Total no of boreholes	4
Total length	185 m
Average length	46.25 m
Total no of Lugeon tests	33
Cost	150×10^3 ESC (1972)

PHASE 2 (1975)

1. Reconnaissance

PHASE 3

1. Boreholes (Mar-Aug 1975)

DESIGNATION		LENGTH (m)	INCLINATION TO THE HORIZ.(o)	NO OF LUGEON TESTS
RIGHT BANK	SAT- 6	45.6	60	8
	SAT- 8	34.7	60	4
LEFT BANK	SAT- 5	47.0	70	8
	SAT- 7	51.4	60	7
	SAT- 9	48.4	60	6

Total no of boreholes	5
Total length	227.1 m
Average length	45.4 m
Total no of Lugeon tests	33
Cost	190 x 10^3 ESC (1975)

2. Percussion drills (Jun - Aug 1975)

DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ.(o)	NO OF LUGEON TESTS
SAT 10	40	60	5
SAT 11	45	60	7
SAT 12	50	60	4
SAT 13	50	60	6
SAT 14	45	60	4
SAT 15	45	60	5
SAT 16	50	60	4
SAT 18	20	90	3

Total no of drills	8
Total length	345 m
Average length	43.1 m
Total no of Lugeon tests	38
Cost	200 x 10^3 ESC (1975)

3. Trenches (Set 1975)

DESIGNATION	LENGTH (m)
V 1	50
V 2	24

Total length	74 m
Cost	8 x 10^3 ESC (1975)

DAM NAME — FAGILDE

— GENERAL DATA —

LOCATION	—	DÃO RIVER; VILA CORÇA; VISEU; PORTUGAL
TYPE	—	CONCRETE ARCH DAM
TOTAL CREST LENGTH	—	63.3 m
CREST CHORD LENGTH	—	61 m
MAX. HEIGHT ABOVE FOUNDATIONS	—	27 m
CREST CHORD / HEIGHT RATIO	—	2.3
MAX. HEIGHT ABOVE THE RIVER BED	—	18.5 m
WIDTH OF THE VALLEY (bottom)	—	12 m
RESERVOIR CAPACITY (total)	—	$4 \times 10^6 m^3$
FLOODED AREA	—	9 ha
DAM VOLUME	—	$8.6 \times 10^3 m^3$
DATE OF THE PROJECT DESIGN	—	MAY 1979
DESIGN PERIOD (includes the pre-design)	—	1973-1979
OWNER	—	DIRECÇÃO GERAL DE SANEAMENTO BÁSICO
DESIGNED BY	—	HIDROPROJECTO, SARL - LISBOA
CONSTRUCTION COST (project estimate)	—	42×10^6 ESC
COST OF THE DESIGN	—	2.2×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	—	2 YEARS
STATUS	—	TO BE CONSTRUCTED

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigations for this dam were performed in two phases, both in 1973.

1973 - The site investigations commenced with a geological survey by field mapping.

1973 - A second phase by means of a small number of boreholes.

A geotechnical assessment report was issued in March 1974 after the site investigations had been completed, which considered the site suitable for a 20 m high concrete dam. The pre - design presented in September 1976, still considered different conceptions for the type of dam to be built. A concrete arch dam was then selected, but the geotechnical conditions did not influence the choice.

The project design was issued in May 1979 which also included a final geotechnical assessment report. In this report, the following values for the mechanical parameters of the rock mass were suggested:
E = 10 000 MPa to 15 000 MPa
c = 2 MPa to 5 MPa
$\phi = 45^{o}$

A rock mass deformability modulus equal to 10 000 MPa and a Poisson coefficient equal to 0.25 were used for the design. Stability studies concerning sliding along the contact between the rock mass and the foundations showed that, not considering the cohesion, a value of $\phi = 33^{o}$ would be enough to guarantee the stability of the dam. Then, taking into account the geotechnical report suggestions, the assessed safety coefficient against sliding was 1.55.

It was intended, in 1963, to begin the site investigations but the studies were stopped just after a short (5 m) trench had been excavated. Nevertheless information was gathered from this trench during the 1973 site investigations and so it will be quoted when dealing with the works performed.

The scheme comprises a water intake located in the right bank, and the safety controls are two rectangular (6 x 3.4 m^2) vents set at the lower part of the dam body.

FOUNDATION CONDITIONS

1. Rock Type

The rock type present at the site is a medium to coarse, sometimes pseudoporphyritic, two micas granite.

Some coarse, sandy sediments and some granitic boulders of little significance fill the bottom of the valley.

2. Structure

The granite is crossed by some aplite or pegmatite veins, with attitude sometimes coincident with that of the more frequent joints.

No major structural defects were assessed, and the major joint set were reported as follows:

DIRECTION (o)	INCLINATION
N(50 - 80)W	SUBVERTICAl
N(40 - 50)E	SUBVERTICAL
SUBHORIZ. WITH SLIGHT DIP TO UPSTREAM	

The direction of the first of these sets is the same as that of the river alignment.

3. The rock mass presents great variations in its state of weathering. The outcrops at the right bank are fresh while in the left bank residual soils predominate, especially at the top. An average depth of 5 m was anticipated for the excavations, which will exceed 8 m in the top of the left bank.

4. Permeability

The depth to the water table increases from the bottom of the valley to the top of the banks where it reaches about 7 m. Water absorption tests were carried out together with the boreholes at pressure steps of 0.25; 0.5; 1; 0.5; 0.25; MPa, from the depth of 5m downwards. Of the 4 boreholes, two gave nil absorption from the depth of 10m and the other two gave nil absorption in all the sections tested. It was considered that the rock

mass would behave as an impervious material beneath the foundation horizon and so no treatment was suggested.

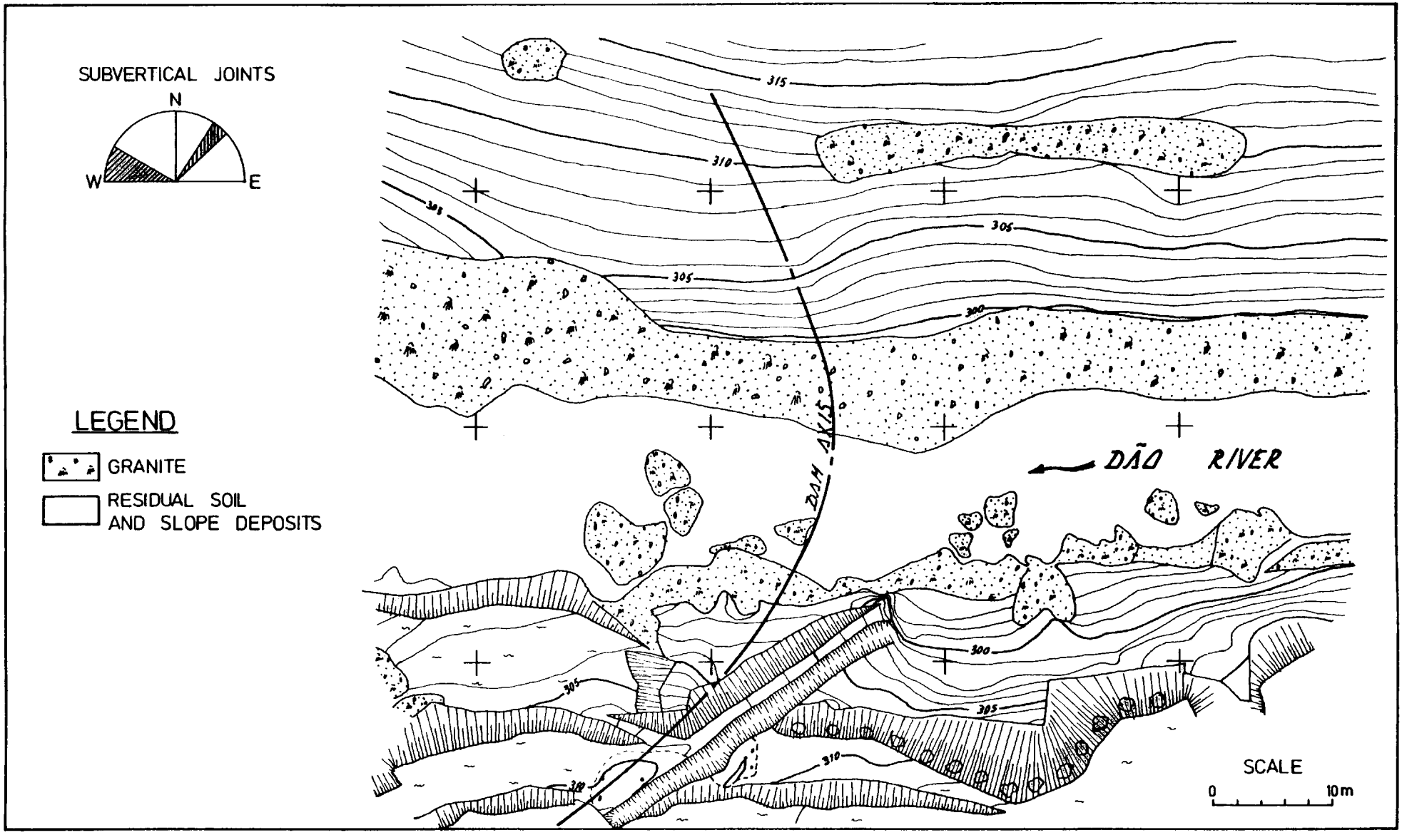


FIG. APPENDIX A4 – FAGILDE DAM SITE

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

Works performed in 1963

1. Trench

1 trench with a length of 5 m

Cost (assessed) 2×10^3 ESC (1973)

PHASE 1 (1973)

1. Reconnaissance (May 1973)

PHASE 2 (1973)

1. Boreholes (Nov - Dec 1973)

DESIGNATION		LENGTH (m)	INCLINATION TO THE HORIZ$_{(o)}$	NO OF LUGEON TESTS
LEFT BANK	S 1	30.33	60	5
	S 3	22.27	60	3
RIGHT BANK	S 2	30.12	60	4
	S 4	26.20	60	4

Total no of boreholes	4
Total length	108.9 m
Average length	27.2 m
Total no of Lugeon tests	16
Cost	100×10^3 ESC (1973)

DAM NAME — FOZ-CÔA

— GENERAL DATA —

LOCATION	—	CÔA RIVER; VILA NOVA DE FOZ-CÔA; PORTUGAL
TYPE	—	CONCRETE ARCH DAM
TOTAL CREST LENGTH	—	433 m
CREST CHORD LENGTH	—	374 m
MAX. HEIGHT ABOVE FOUNDATIONS	—	145 m
CREST CHORD / HEIGHT RATIO	—	2.58
MAX. HEIGHT ABOVE THE RIVER BED	—	130 m
WIDTH OF THE VALLEY (bottom)	—	≈50 m
RESERVOIR CAPACITY (total)	—	867x $10^6 m^3$
FLOODED AREA	—	1793 ha
DAM VOLUME	—	970 $10^3 m^3$
DATE OF THE PROJECT DESIGN	—	MAR 1981
DESIGN PERIOD	—	1977-1981
OWNER	—	ELECTRICIDADE DE PORTUGAL
DESIGNED BY	—	ELECTRICIDADE DE PORTUGAL
CONSTRUCTION COST (project estimate)	—	6000 x 10^6 ESC
COST OF THE DESIGN	—	N.A.
CONSTRUCTION PERIOD (project estimate)	—	6 YEARS
STATUS	—	UNDER CONSTRUCTION

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigations at Foz-Côa dam site were undertaken in three phases: phases 1 and 2-1977, phase 3-1978-1979.

1977 - The site investigations began with basic geological survey by field mapping.

1977 - Second phase of site investigations comprising a few trenches. Based upon the performed works a first geotechnical assessment report was performed in July 1977.

1978-1979 - The third and comprehensive phase of site investigations which consisted of geophysical exploration, trenches, galleries, boreholes, "in-situ" and laboratory tests. A final geotechnical assessment report was prepared in November 1980, which also forms part of the main design report released in March 1981.

The site was selected on both topographic and geological reasons within a 4 km long river section. The first engineering geological report was prepared for a preliminary study issued in November 1977. At that time, the proposed height for the dam was 10 m lower than the actual design. Before the final geotechnical report one other report was prepared in April 1980, dealing only with the temporary diversion structures.

The deformability modulus of the rock mass used for the design, based on the results of the mechanical characterization tests, was 20 000 MPa for the bottom of the valley and upper parts of the banks, and 15 000 MPa for the zones between these areas. The shear stress applied by the structure along the foundation plane, was assessed as being 1 MPa under the most unfavourable conditions. The highest friction angle of the rock mass needed to ensure stability is about 0,75, disregarding the cohesion, which was supposed to be exceeded by the tan $\emptyset$ of the joints in spite of a few

lower results from the joint shear tests carried out.

The dam will be built for energy purposes.

The safety structures are a 620 m long spillway with a diameter of 8.3 m, in the left bank, and 4 bottom discharges (vents) 3.85 m in diameter, through the dam body.

There is a semi-buried power house which will drive 2 x 105 Mw .

A 336 long tunnel with a diameter of 7 m, in the right bank, forms part of the provisional diversion.

FOUNDATION CONDITIONS

1. Rock Type

The rock mass consists of pre-Ordovician phyllites and interbedded greywackes. Some thin layers of marble are also present.

The top soil is not thicker than 0.7 m. Some slope deposits are present mainly in the left bank where, at the lowest levels they can be thicker than 2.2 m.

The bottom of the valley is partially covered by recent detrital material comprising of boulders, gravels and sand, with a thickness varying from 0.5 m to 2.8 m.

2. Structure

The geological formations are highly folded with folds approximately vertical and relatively tight. The fold hinges are either horizontal or dipping at about 10° NW or up to 5°SE.

Some quartz veins, not more than 0.3 m wide cut across the rock mass, some of them filling existing fault spaces.

The schistosity strike is N (55° - 70°) W and its dip is nearly vertical (70°SW to 78° NE). The bedding strike is N (55° - 68°) W and its dip is either (62° - 70°) SW or (54° - 67°) NE.

A great number of faults were assessed at the site, some of them filled by quartz veins, others by mylonite with a clay matrix. The assessed fault sets and their attitudes are presented in Fig. Appendix A6, which also shows the attitude of the quartz veins, and that of the assessed joint sets as well. Of these, the more important sets have spacings of between 0.1 m and 2.0 m ; the less frequent sets and the ones showing little frequency have spacings of between 0.1 m and 1.6 m and between 0.1 m and 5.5 m respectively.

3. State of Weathering

The decompression and weathering of the rock mass is obvious in the

banks, where it is more intense in the right bank. In the left bank the decompressed and weathered rock reachs depths of about 11m at the highest levels, about 3 m near the bottom of the valley and, about 7 m to 8 m between those levels. In the right bank these depths are about 15 m from the highest points, and 4 - 5 m near the bottom of the valley; between those two levels, two zones with different depths of weathering were assessed, the lowest one has a depth of 9 - 10 m and the other a depth of between 18 m and 20 m.

The depths of excavations suggested in the geotechnical report were 10 - 20 m in the right bank, 5 - 10 m in the left one, and not more than 5 m at the botton of the valley.

4. Permeability

A total of more than 400 Lugeon tests, were carried out together with the boreholes, along 5 m boreholes sections, which assisted to assess the permeability conditions of the rock mass. Based on their results it was concluded in the geotechnical report that the rock mass exhibits low permeability. The depth beneath which the absortion tests gave results lower than 1 Lugeon was assessed. This depth lies between 34 m and 52 m in the left bank and between 27 m and 53 m in the right bank. A grout curtain was suggested to reach 20 m below the above quoted depths in the left bank, and 30 m below that for the right bank.

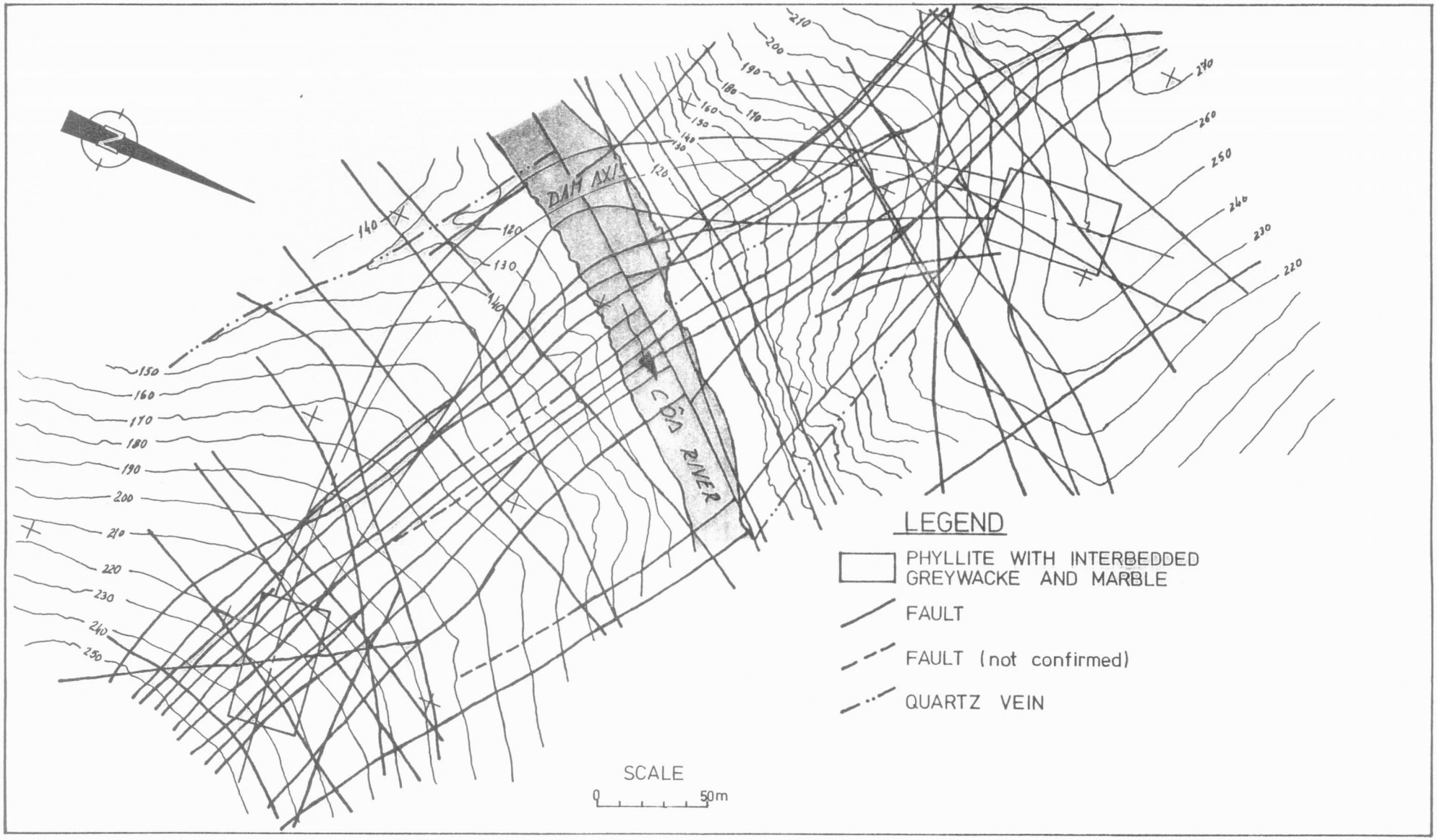


FIG. APPENDIX A5 - FOZ-CÔA DAM SITE

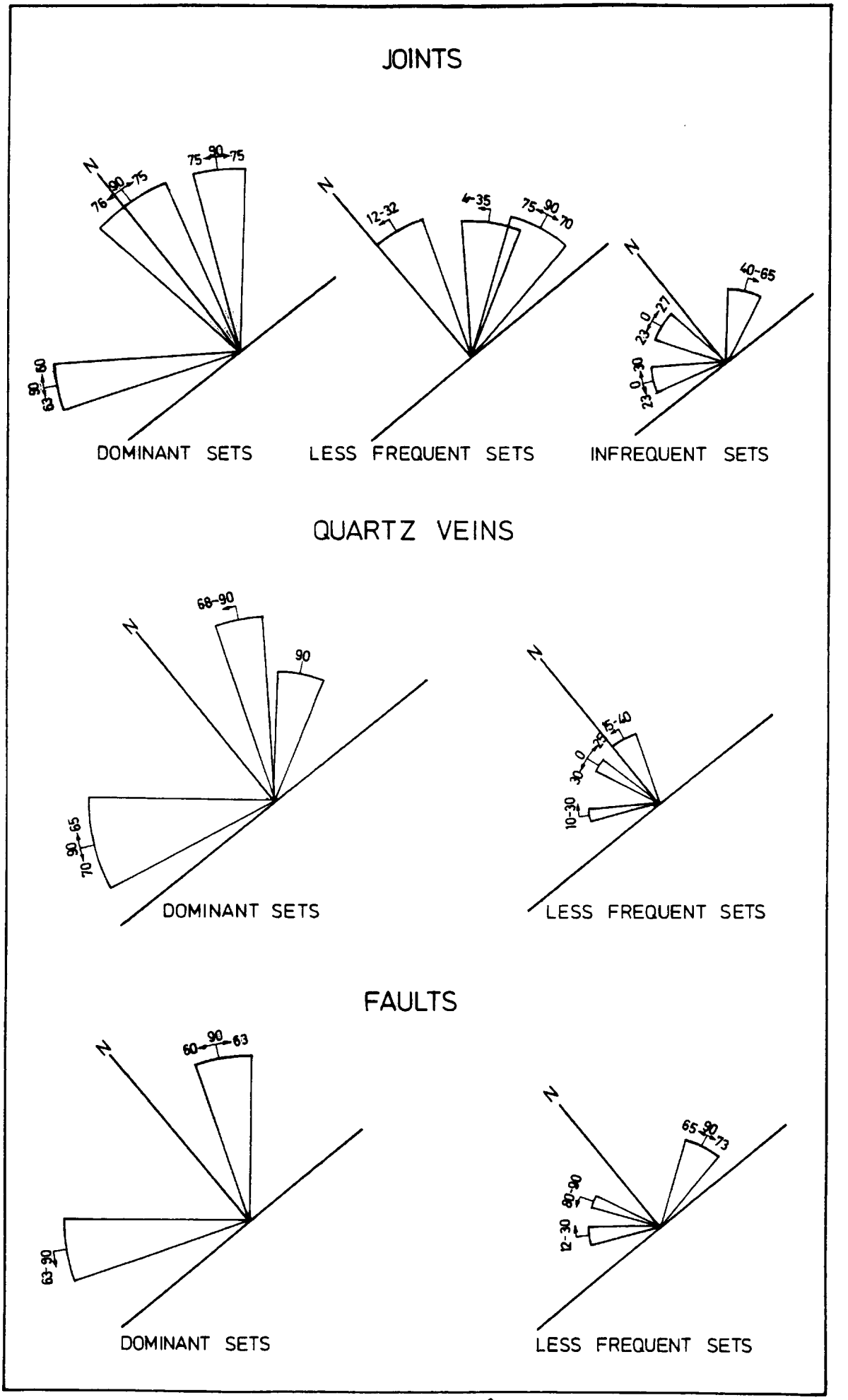


FIG. APPENDIX A6-FOZ-CÔA DAM SITE
(structural elements)

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (1977)

1. Reconnaissance (early 1977)

PHASE 2 (1977)

1. Trenches (early to mid 1977)

DESIGNATION		LENGTH (m)
LEFT BANK	TE 240	150
	TE 230	275
	TE 220	255
	TE 200	232
	TE 175	127
RIGHT BANK	T -	150

Total no of trenches	6
Total length	1189 m
Average length	198 m
Cost	1690 x 10^3 Esc. (1977)

PHASE 3 (1978 - 1979)

1. Geophysical exploration (Jan 1978)

1.1 Apparent resistivity profiles

A total of 11 apparent resistivity profiles, with an average length of 250 m were performed, whose results were used to produce an apparent resistivity map.
$\overline{AB}$ = 18 m; $\overline{MN}$ = 5 m.

1.2 Seismic refraction profiles

33 seismic refraction profiles, each 65m long, were performed. A 12 geophone array was used with direct and reverse shots.
Total length of profiles 2145 m

Cost of geophysical exploration - 300 x 10^3 ESC (1977)

2. Trenches (May 1978 - Aug 1979)

DESIGNATION		LENGTH (m)
LEFT BANK	TE 120	150
	TE 155	100
RIGHT BANK	TD 120	290
	TD 140	170
	TD 160	285
	TD 180	190
	TD 200	165
	TD 220	175
	TD 240	175

Total no of trenches	9
Total length	1700 m
Average length	189 m
Cost	3660 x 10^3 ESC (1979)

3. Galleries (May 1978 - Sep 1979)

DESIGNATION		LENGTH (m)
LEFT BANK	GE 220	52
	GE 200	63
	GE 175	50
	GE 155	33
	GE 120	56
RIGHT BANK	GD 213	70
	GD 193	61
	GD 160	58
	GD 140	103
	GD 120	55

Total no of galleries	10
Total length	601 m
Average length	60.1 m
Cost	8030 x 10^3 ESC (1979)

4. Boreholes (May - Sep 1979)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ (o)	NO OF LUGEON TESTS
RIGHT BANK	SD- 1	40.10	45	7
	SD- 2	40.30	45	7
	SD- 10	55.00	50	10
	SD- 20	60.00	65	11
	SD- 5	50.40	45	9
	SD- 7	55.00	37	10
	SD- 19	80.13	65	15
	SDV-4	80.10	90	-
	SDG-1	55.00	45	-
	SDV-3	70.00	90	-
	SDV-2	60.00	90	-
	SD- 6	50.20	45	9
	SD- 13	45.47	45	7
	SD- 14	50.10	45	9
	SD- 18	90.20	65	17
	SD- 12	50.10	45	9
	SD- 15	35.00	30	6
	SD- 16	20.10	60	3
	SDV-1	40.13	90	-
	SD- 11	45.00	60	8
	SD- 8	80.00	45	15
	SDV-5	40.15	90	-
	SD- 9	80.50	45	14
	SD- 3	45.00	45	8
	SD- 4	45.00	45	8
	SD- 21	50.00	30	9
BOTTOM OF THE VALLEY	SD- 17	90.00	60	8
	SD- 8	80.00	45	15
	SE- 1	80.40	45	15
	SE- 19	80.00	45	15
	SE- 15	40.00	45	7
LEFT BANK	SE - 18	50.00	45	9
	SE - 20	90.10	60	17
	SE - 14	40.20	30	7
	SE - 17	40.18	45	7
	SE - 16	40.05	30	7
	SE - 13	50.00	30	9
	SE - 11	65.00	45	12
	SE - 12	60.00	45	11
	SE - 9	45.00	45	8
	SE - 10	40.15	45	7
	SE - 21	75.00	60	14
	SE - 8	35.10	45	5
	SEG- 1	50.00	60	-
	SE- 6	45.00	45	8
	SE- 5	50.10	40	9
	SE- 22	65.00	60	12
	SE- 7	50.17	45	8
	SE- 4	45.00	30	8
	SE- 3	50.15	60	9
	SE- 2	70.30	45	12

Total no of boreholes	51
Total length	2839.88 m
Average length	55.68 m
Total no of Lugeon tests	430
Cost	(see bellow)

5. More boreholes (same period as above)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{(o)}$	NO OF LUGEON TESTS
SAFETY STRUCTURES	SRE - 1	45.30	45	8
	SRE - 2	60.09	60	11
	SRE - 3	80.20	60	16
	SRE - 4	90.00	60	17
	SRE - 5	60.30	45	11
	SRE - 6	35.00	35	6
	SRD - 1	36.17	45	6
	SRD - 2	40.00	45	7
	SRD - 3	40.00	45	7
	SRD - 4	30.00	30	5
	SRD - 5	30.54	45	5

Total no of boreholes	11
Total length	547.6
Total no of Lugeon tests	99
Cost (includes all boreholes)	5770 x 10^3 ESC (1979)

6. Geophysical exploration (Jul - Aug 1979)

A comprehensive geophysical exploration programme was performed which consisted of:

6.1 Cross - hole method

This method was applied by means of a special technique developed at LNEC - LISBON, using short boreholes (3 - 6 m) sunk into the floor and into the walls of 4 galleries in each bank. V_p and V_s were measured. The total number of measurements reached 300.

6.2 Measurement of V_p (direct waves)

a) Between boreholes - 15 shots

b) Between boreholes and galleries - 15 shots

c) Between galleries - 12 shots.

6.3 Ultrasonic wave velocities

A total of 82 samples taken from the short boreholes sunk inside the galleries (see 6.1) were subjected to laboratory ultrasonic wave velocity measurements (V_p and V_s)

The results of this geophysical exploration programme enabled:

a) To assess the dynamic deformability modulus of the rock mass (E_{dyn})
b) To obtain a relationship between the longitudinal wave velocities measured in the field and those measured in the laboratory:

$$I_v = \frac{V_p \text{ (field)}}{V_p \text{ (lab)}}$$

c) To obtain the following correlation between E_{dyn} and the static "in situ" deformability modulus - E_{sta} (assessed by mean of flat jack tests):

$\frac{E_{sta}}{E_{dyn}} = 1.66\ I_v - 0.45$, with a correlation coefficient equal to 0.9.

Cost of geophysical exploration - 550×10^3 ESC (1979)

7. "In situ" tests (Apr - Oct 1979)

7.1 Flat jack tests

Device used	2 LFJ (LNEC)
Area loaded	2.3 m^2
No of loading cycles	3 to 5
Maximum pressure applied	6.5 MPa (1MPa in one test)
Total no of tests	24

Three tests were performed inside each four galleries on each bank, to determine the deformability modulus in three directions: parallel and perpendicular to the schistosity and in the direction of the stresses to be imposed by the dam.

Results:

GALLERY		DEFORMABILITY MODULUS (MPa)		
		// SCHISTOSITY	⊥ SCHISTOSITY	// DAM STRESSES DIRECTION
LEFT BANK	GE 220	70 200	26 700	14 800
	GE 175	61 200	33 300	32 400
	GE 155	31 100	14 000	9 600
	GE 120	26 700	44 200	27 900
RIGHT BANK	GD 120	26 600	62 500	47 900
	GD 140	10 000	7 100	8 600
	GD 160	5 200	1 600	600
	GD 193	13 400	13 100	9 800

7.2 Joint shear tests

Total no of samples 3

Area of shear $0.7 \times 0.7 m^2$

No. of tests per sample 4

Results:

BLOCK NO	c (MPa)	∅ (o)
* GE 220 (1)	0.13	34
* GE 220 (2)	0.12	30
GE 193	0.14	35

*Joints with clay infilling

Cost of "in-situ" tests - 4570×10^3 ESC (1979)

8. Laboratory tests (same period as above)

8.1 Uniaxial tests

Total no. of tests 12

Type of samples: cylinders

Diameter 62 mm

Aspect ratio 2.1 to 4.3

Results:

E varied from 50 200 MPa to 91 400 MPa for angles between the load direction and the schistosity planes between 90^{o} and 0^{o} respectively . For the 12 tests performed, the Poisson coefficient varied from .14 to .32 being the greatest values for oblique angles between the direction of the stress and the schistosity planes.

8.2 More uniaxial tests

Total no of tests 36

Type of samples: prisms

Area: 5 x 5 cm^2 height: 12.5 cm

Results:

SAMPLES FROM GALLERY		DEFORMABILITY MODULUS (MPa)			POISSON COEFFICIENT		
		// SCHISTOS.	⊥ SCHISTOS.	DAM STRESSES DIRECTION	// SCHISTOS.	⊥ SCHISTOS.	// DAM STRESSES DIRECTION
LEFT BANK	GE 220	84 200	42 300	32 000	0.29	0.15	0.14
	GE 175	76 700	46 600	71 000	0.33	0.12	0.33
	GE 155	79 000	65 400	34 000	0.42	0.23	0.25
	GE 120	99 200	54 400	52 700	0.27	0.19	0.31
RIGHT BANK	GD 120	71 900	46 000	56 800	0.29	0.10	0.30
	GD 140	94 300	40 600	13 400	0.22	0.11	0.34
	GD 160	67 000	11 200	12 000	0.50	0.07	0.08

The prisms were cut with axes either parallel or perpendicular to the schistosity planesor parallel to the direction of the stresses which will be imposed by the dam.

8.3 Rock shear tests

Total no of samples 25
Type of samples: cut from rock cores
Area of shear 200 cm^2

The shear surfaces were parallel to the schistosity.

Results:

SAMPLE LOCATION	c (MPa)	Ø (°)
GE 220	2.32	58
GE 175	1.85	57
GE 120	3.50	65
GD 120	4.35	57
GD 160	1.58	51
GD 193	1.20	63

8.4 Joint shear tests

Total no of samples 41
Type of samples: cut from rock cores
Area of shear 200 cm^2
No of tests per sample 4

Three joint sets were tested: one subvertical , one subhorizontal and one parallel to the schistosity

Results:

JOINT SET	C (MPa)			Ø (°)		
	MAX.	MIN.	AVER.	MAX.	MIN.	AVER.
SUBVERTICAL	0.22	0.08	0.15	43.0	32.0	38.8
SUBHORIZONT.	0.21	0.11	0.17	43.5	36.0	40.2
// SCHISTOS.	0.15	0.05	0.11	40.0	32.5	35.2

Cost of laboratory test - 360×10^3 ESC (1979)

DAM NAME — FRONHAS

— GENERAL DATA —

LOCATION	—	ALVA RIVER; FRONHAS, ARGANIL; PORTUGAL
TYPE	—	CONCRETE ARCH DAM
TOTAL CREST LENGTH	—	190 m
CREST CHORD LENGTH	—	167.4 m
MAX. HEIGHT ABOVE FOUNDATIONS	—	62 m
CREST CHORD / HEIGHT RATIO	—	2.7
MAX. HEIGHT ABOVE THE RIVER BED	—	56 m
WIDTH OF THE VALLEY (bottom)	—	≃60 m
RESERVOIR CAPACITY (total)	—	$89 \times 10^6 m^3$
FLOODED AREA	—	534.6 ha
DAM VOLUME	—	$102.8 \times 10^3 m^3$
DATE OF THE PROJECT DESIGN	—	DEC 1976
DESIGN PERIOD	—	1975-1976
OWNER	—	DIRECÇÃO GERAL DOS RECURSOS E APPROVEITAMENTOS HIDRAÚLICOS - LISBOA
DESIGNED BY	—	ELECTRICIDADE DE PORTUGAL
CONSTRUCTION COST (project estimate)	—	500×10^6 ESC
COST OF THE DESIGN	—	$\simeq 4.5 \times 10^6$ ESC
CONSTRUCTION PERIOD (project estimate)	—	3 YEARS
STATUS	—	UNDER CONSTRUCTION

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

Three phases of site investigations were performed to study this dam site; phase 1-1975, phase 2-1975, phase 3 1975 - 1976.

1975 - Basic geological survey by field mapping.

1975 - 1976 - A second comprehensive phase of site investigations was carried out during September and October, by means of geophysical exploration, trenches, galleries, boreholes, "in - situ" and laboratory tests.Two geotechnical assessment reports were issued in July and November 1976.

The first site investigations carried out at this site were dated 1938, according to a report issued in 1940. In 1964 a decision to design a dam was retaken,but the site was changed to a location 3 km downstream, which would allow the creation of a greater reservoir. The site investigations performed at this new location revealed very unfavourable geotechnical conditions and the studies stopped in 1968. A geotechnical assessment report issued in 1972 led to the abandonment of this location. Thus, it was decided to return to the previously considered site. Some information was gathered from the works performed in 1938 and so these works will also be quoted here.

The results of the mechanical characterization tests were not available in time for the design which was released in December 1976. For this reason the calculations were made based on a deformability of between 10 000 MPa and 20 000 MPa for the rock mass. The calculations would be repeated if the results were not in accordance with the adopted range.

The dam is for energy and flood regularization purposes. The safety structures are two 2.2 x 2.2 m^2 bottom discharges and two similar half bottom discharges.

FOUNDATION CONDITIONS

1. Rock Type

The rock mass consists of graphite phyllites with interbedded meta - greywackes (see Fig. Appendix A7). Alluvial deposits at the bottom of the valley can reach a thickness of 7 m. Slope deposits which are present mainly at the lower points of the banks are, in general, not thicker than 1 m.

2. Structure

The rock mass is severely folded, the folds being almost isoclinal with general strike NW - SE. The direction of the schistosity is NW - SE to WNW - SSE, dipping to NE or NNE (sub -vertical).

Several quartz veins, seldom wider than 2 m in most cases with an attitude similar to that of the schistosity, and few dikes and veins of porphyry with direction NNW - SSE and NW - SE cross the rock mass. A dolerite dyke crosses the right bank (see Fig. Appendix A7)

Some faults were assessed at the dam site, the majority of them with directions of between NNW - SSE and N - S. Another less frequent set with a direction NE - SW or NNE - SSW is also present. The faults are generaly filled in by fragments of the existing rock in a clay and iron oxides matrix, and the widest fault space is of the order of 30 cm.

Several joint sets were assessed as shown in Fig. Appendix A7.

3. State of weathering

The depth of weathering and decompression of the rock mass increases from the bottom of the valley to the highest points on the banks. The expected depths of excavation varied from 2 m at the bottom of the valley to 11 m and 13 m at the left and right banks respectively.

4. Permeability

The geotechnical report concluded, based upon the results of Lugeon tests carried out with the boreholes that absorptions of 1 Lugeon were reached at depths of about 35 m.

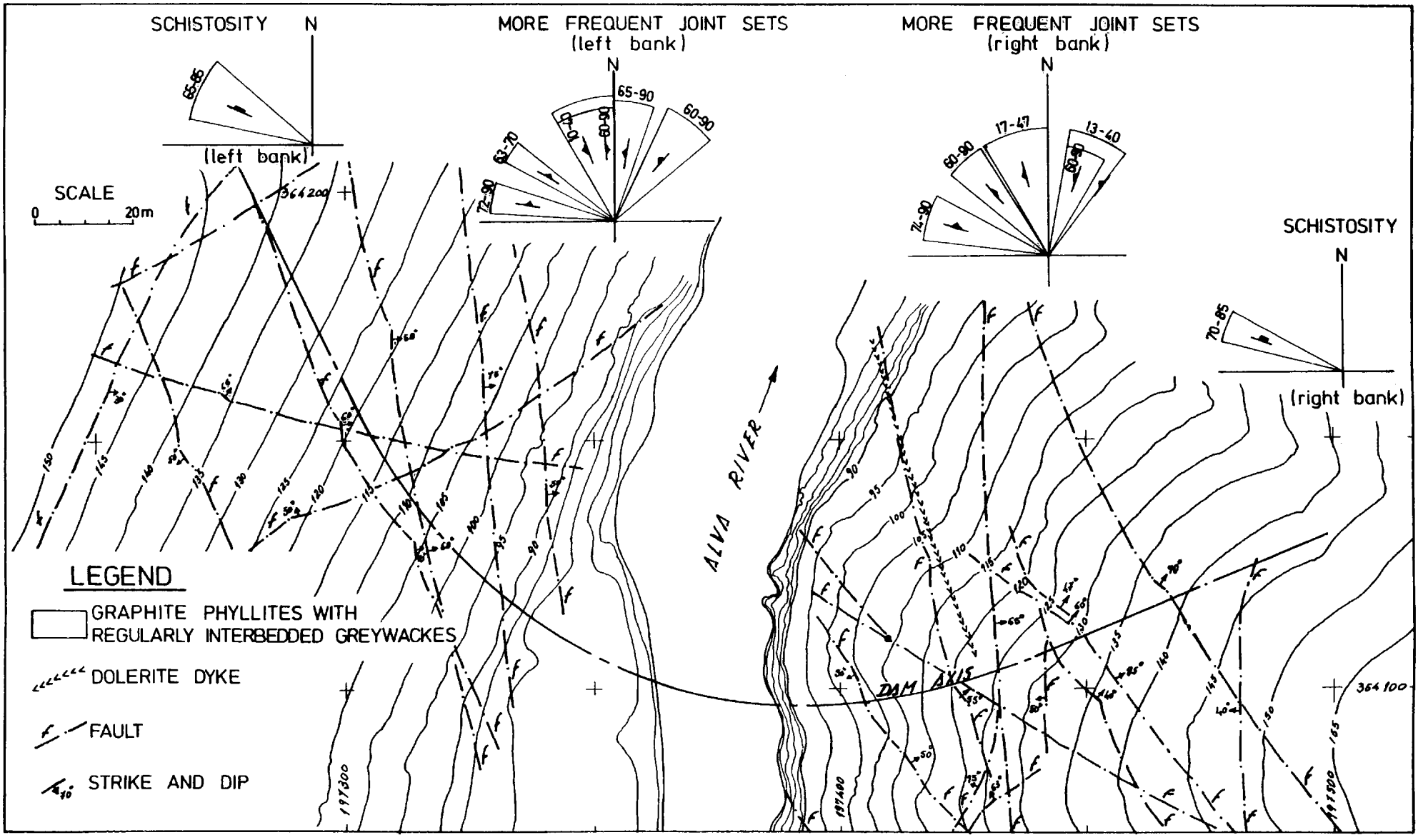


FIG. APPENDIX A7 – FRONHAS DAM SITE

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

Works performed during 1938 - 1940

1. Boreholes

DESIGNATION	LENGTH (m)
A	20.1
A1	20.4
A2	20.1
A3	20.0
A4	20.0
B	20.1
B1	20.5
B2	20.0
C	20.0
C1	20.4
C2	20.2
D	20.1
E	19.8
F	19.9

Total no of boreholes	14
Total length	281.6 m
Average length	20.1 m
Cost (assessed at 1976 prices)	390 x 10^3 ESC (1976)

2. Galleries

DESIGNATION		LENGTH (m)
LEFT BANK	G1	20.0
	G3	21.0
RIGHT BANK	G2	19.0
	G4	20.0

Total no of galleries	4
Total length	80 m

Average length 20 m

Cost (assessed at 1976 prices) 440 x 10^3 ESC (1976)

PHASE 1 (1975)

1. Reconnaissance

PHASE 2 (1975 - 1976)

1. Geophysical exploration (Sep - Oct 1975)

1.1 Apparent resistivity profiles

The results of 11 apparent resistivity profiles each with an approximate length of 125 m were used to produce an apparent resistivity map. $\overline{AB}$ and $\overline{MN}$ were 65 m and 5 m respectively.

1.2 Seismic refraction profiles

12 seismic refraction profiles each 120 m long were carried out along the same alignments used for the apparent resistivity profiles, using the direct, reverse and mid point shot method.
Total length of profiles - 1 440 m

1.3 Seismic fans

4 seismic fans with shots inside 2 galleries in each bank, with geophones at the surface.
Cost of geophysical exploration - 170 x 10^3 ESC (1975)

2. Trenches (Oct 1975 - Jul 1976)

	DESIGNATION	LENGTH (m)
RIGHT BANK	TD 135	140
	TD 120	125
	TD 105	135
	TD 90	85
	TD90/105	20

	DESIGNATION	LENGTH (m)
LEFT BANK	TE 135	135
	TE 120	130
	TE 105	90
	TE 90	120

Total no of trenches	9
Total length	980 m
Average length	108.9 m
Cost	750 x 10^3 ESC (1976)

3. Galleries (same period as above)

	DESIGNATION	LENGTH (m)
RIGHT BANK	GD 93	35
	GD 105	27
	GD 120	20
	GD 130	24

	DESIGNATION	LENGTH (m)
LEFT BANK	GE 90	24
	GE 105	23
	GE 120	20
	GE 130	30

Total no of galleries	8
Total length	209 m
Average length	26.1 m
Cost	1 150 x 10^3 ESC (1976)

4. Boreholes (Feb - Aug 1976)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{(0)}$	NO OF LUGEON TESTS
RIGHT BANK	SD 1	40.0	45	8
	SD 2	40.0	30	7
	SD 3	45.0	45	8
	SD 4	40.5	45	7
	SD 5	45.1	45	8
	SD 7	35.5	90	7
	SD 8	35.1	60	6
BOTTOM OF THE VALLEY	SD 6	60.1	45	11
	SE 5	45.0	45	7
	SE 6	45.3	45	9
	SE 7	50.0	45	8
	SE 8	50.2	45	8
LEFT BANK	SE 1	40.0	45	7
	SE 2	45.6	30	8
	SE 3	45.0	45	9
	SE 4	45.0	45	7
	SE 9	55.2	60	9
	SE 10	45.1	60	8

Total no. of boreholes	18
Total length	807.7 m
Average length	44.9 m
Total no. of Lugeon tests	142
Cost	850 x 10^3 ESC (1976)

5. "In - situ" tests (May - Nov 1976)

5.1 Dilatometer tests

Device used	BHD - LNEC
No. of boreholes tested	4
Horizons measured in each borehole	4 (5 in one borehole)
Directions measured in each horizon	8
No of loading cycles	3 - 4
Maximum pressure applied	6 MPa
Total no of horizons measured	17

The tests were performed between the depths of 12.12m and 25.55m.

Results:

(all measurements)

DILATOMETER MODULUS (MPa)		
MAX	MIN	AVER
30 300	1 500	10 480

5.2 More dilatometer tests

Tests performed in short boreholes (≃ 1 m) inside the four recent galleries, 6 boreholes in each (2 vertical, 2 horizontal and 2 parallel to the bedding).

Horizons measured in each borehole 1

Total nn of horizons measured 24

(Other test conditions as in the previous dilatometer tests).

Results:

BOREHOLES POSITION	AVERAGE DILATOMETER MODULUS (MPa) GALLERY NO			
	GD 93	GD 120	GE 90	GE 120
VERTICAL	11 700	6 100	3 500	2 200
HORIZONTAL	4 500	4 400	7 200	4 900
// TO BEDDING	2 600	2 400	7 400	1 800

5.3 Flat jack tests

Device used 2 LFJ - LNEC

Area loaded 2.3 m^2

No of loading cycles 3 - 5

Maximum pressure applied 2 - 4.5 MPa

Total no of tests 12

Three tests were performed in each gallery to determine the deformability modulus in three directions (vertical, horizontal and perpendicular to the bedding).

Results:

GALLERY	DEFORMABILITY MODULUS (MPa)		
	VERTICAL	HORIZONTAL	⊥ TO BEDDING
GD 120	7 100	9 100	5 700
GD 93	11 000	9 000	10 200
GD 120	4 000	6 900	6 400
GD 90	19 900	7 700	14 500

5.4 Joint shear tests

Total no of samples 4

Area of shear 0.7 x 0.7 m^2

No of tests per sample 4

BLOCK NO	c (MPa)	∅ (°)
GD 105 (1)	0.27	45
GD 150 (2)	0.28	24
GD 120	0.33	38
GD 130	0.20	40

Cost of "in -situ" tests - $2\ 150 \times 10^3$ ESC (1976)

6. Laboratory tests (same period as above)

6.1 Uniaxial tests

Total no of tests 48

Type samples: cut from rock cores (boreholes sunk inside the galleries).

Diameter 50 mm

Aspect ratio 2.5

ASSESSMENT OF E

No. of loading cycles 4

Maximum pressure applied 50 or 10 MPa, depending on the state of weathering.

Results:

E (MPa)		
MAX	MIN	AVER
83 300	16 800	45 800

ASSESSMENT OF σ_{uc}

Tests performed on the same samples, increasing pressure up to failure.

Results:

σ_{uc} (MPa)		
MAX	MIN	AVER
85.0	17.5	47.2

The study of the anisotropy of the rock material concerning E and σ_{uc} was also performed.

6.2 Joint shear tests

Total no of samples 33

Type of samples: cut from rock cores or from samples collected inside the galleries.

Area of shear 40 - 104 cm^2

No of tests per sample 4

Results:

JOINT SET	c (MPa)			ϕ (°)			NO OF SAMPLES
	MAX	MIN	AVER	MAX	MIN	AVER	
SUBHORIZ.	0.35	0.09	0.19	45	24	31	8
//TO BEDDING	0.32	0.11	0.20	34	22	27	14
SUBVERTIC.	0.36	0.07	0.20	37	22	32	11

Cost of laboratory tests - 140 x 10^3 ESC (1976).

DAM NAME — FUNCHO

— GENERAL DATA —

LOCATION	—	ARADE RIVER; SILVES; PORTUGAL
TYPE	—	CONCRETE ARCH DAM
TOTAL CREST LENGTH	—	412 m
CREST CHORD LENGTH	—	140 m
MAX. HEIGHT ABOVE FOUNDATIONS	—	43 m
CREST CHORD / HEIGHT RATIO	—	3.3
MAX. HEIGHT ABOVE THE RIVER BED	—	≃ 41 m
WIDTH OF THE VALLEY (bottom)	—	20 m
RESERVOIR CAPACITY (total)	—	$43.4 \times 10^6 m^3$
FLOODED AREA	—	384 ha
DAM VOLUME	—	$133 \times 10^3 m^3$
DATE OF THE PROJECT DESIGN	—	JUL 1978
DESIGN PERIOD (includes a prel. study)	—	1971-1978
OWNER	—	DIRECÇÃO GERAL DOS RECURSOS E APROVEITAMENTOS HIDRÁULICOS - LISBOA
DESIGNED BY	—	COBA, SARL - LISBOA
CONSTRUCTION COST (project estimate)	—	170×10^6 ESC
COST OF THE DESIGN	—	4.86×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	—	N. A.
STATUS	—	TO BE CONSTRUCTED

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigations at Funcho dam site were undertaken in 4 phases: phase 1-1971, phase 2-1971 - 1972, phase 3-1974 - 1975, phase 4-1976.

1971 - A geological survey by field mapping was undertaken.

1971 - 1972 - A second phase of site investigations took place, which consisted of trenches and boreholes. A preliminary study report was issued in August 1972 which included a first geotechnical assessment.

1974 - 1975 - One gallery and one trench were excavated.

1976 - A fourth phase of site investigations was performed, which included galleries, "in-situ" and laboratory tests.

This dam forms part of a scheme for water supply purposes together with the Odelouca dam and a tunnel linking both reservoirs (see Odelouca dam Funcho - Benaciate tunnel).

The preliminary report issued in August 1972, suggested a rock fill dam but after the 1974 - 75 site investigations, it was concluded that the construction of an arch dam would be possible, its construction being about 21% cheaper, and also decreasing the estimated construction period.

One project design was completed in June 1976, before the mechanical characterization tests had been concluded and so the results could not be taken into account. In this design a deformability modulus for the rock mass equal to 5 000 MPa (1/4 Ec - Ec being the deformability modulus of the concrete) was adopted. The results of the mechanical characterization (report issued in April 1977) suggested quite different values and this led to the dam being redesigned. The new design was issued in July 1978 and in this case two different deformability moduli were adopted for the rock mass:

Right bank: $E = 0.125\ Ec$

Left bank: $E = 1.6\ Ec$

For the study of the stability of the abutments, considering the existence of unfavourable joints, the following mechanical parameters were used:

Right bank: $c = 0.25$ MPa, $\phi = 25^{\circ}$

Left bank: $c = 0.25$ MPa, $\phi = 35^{\circ}$

The above values are already corrected for safety coefficients of 5 (cohesion) and 1.5 (friction angle).

The geological studies were carried out directly by the owner of the project and the related final report was issued in December 1977.

The undertaking includes the dam with a crest spillway and a bottom discharge in the right bank under the dam body, consisting of 2 conduits 3.2 m in diameter. There is also a water intake in the left bank.

FOUNDATION CONDITIONS

1. Rock Type

The lithology at the site consists mainly of clay - graphite shales, phyllites and greywackes in alternating series (see Fig. Appendix A8). Greywackes are dominant in the left bank while shales and phyllites are the main rock types in the right bank. The thickness of the slope deposits is about 5 m while that of the alluvium is seldom greater than 3 m.

2. Structure

The orogenic movements which affected the region in the geological past, led to the presence of several faults and severe folding, the folds being frequently isoclinal.

At the site, no major faults were found.

The strike of the bedding and schistosity falls between N10°E and N 25°W while the dip varies from 40° to 60° E being quite lower at the bottom of the left bank. The joint assessment was carried out inside two galleries because, at the surface, joints show a wide scatter due to creep. The assessed joint sets are presented in Fig. Appendix A8. Sets marked as A and A' are coincident with the schistosity.

3. State of Weathering

In the right bank, the depth of weathered rock varies between 9 m and 15 m under the highest points. However, because of the low values of the deformability of the rock mass in this bank the expected depths for the excavations will be several metres more. The state of weathering is better in the left bank and it seemed to be enough to remove the superficial zone to reach good rock for the foundation, but for some reasons related to the design, a depth of 15 m will have to be excavated.

4. Permeability

The depth of the water table varies from about 35 m in the left bank, to 25 - 30 m beneath the river bed, to 45 m in the right

bank.

Based upon the results of Lugeon tests performed together with the boreholes,it was assessed that the upper zone of the rock mass is highly pervious. The depth of that zone was calculated as being about 35 m on average, being around 25 m under the bottom of the valley. One borehole sunk in the right bank gave for that zone a depth of 42 m.

A grout curtain whose depth would be defined during the cons - truction phase was suggested in the geotechnical report.

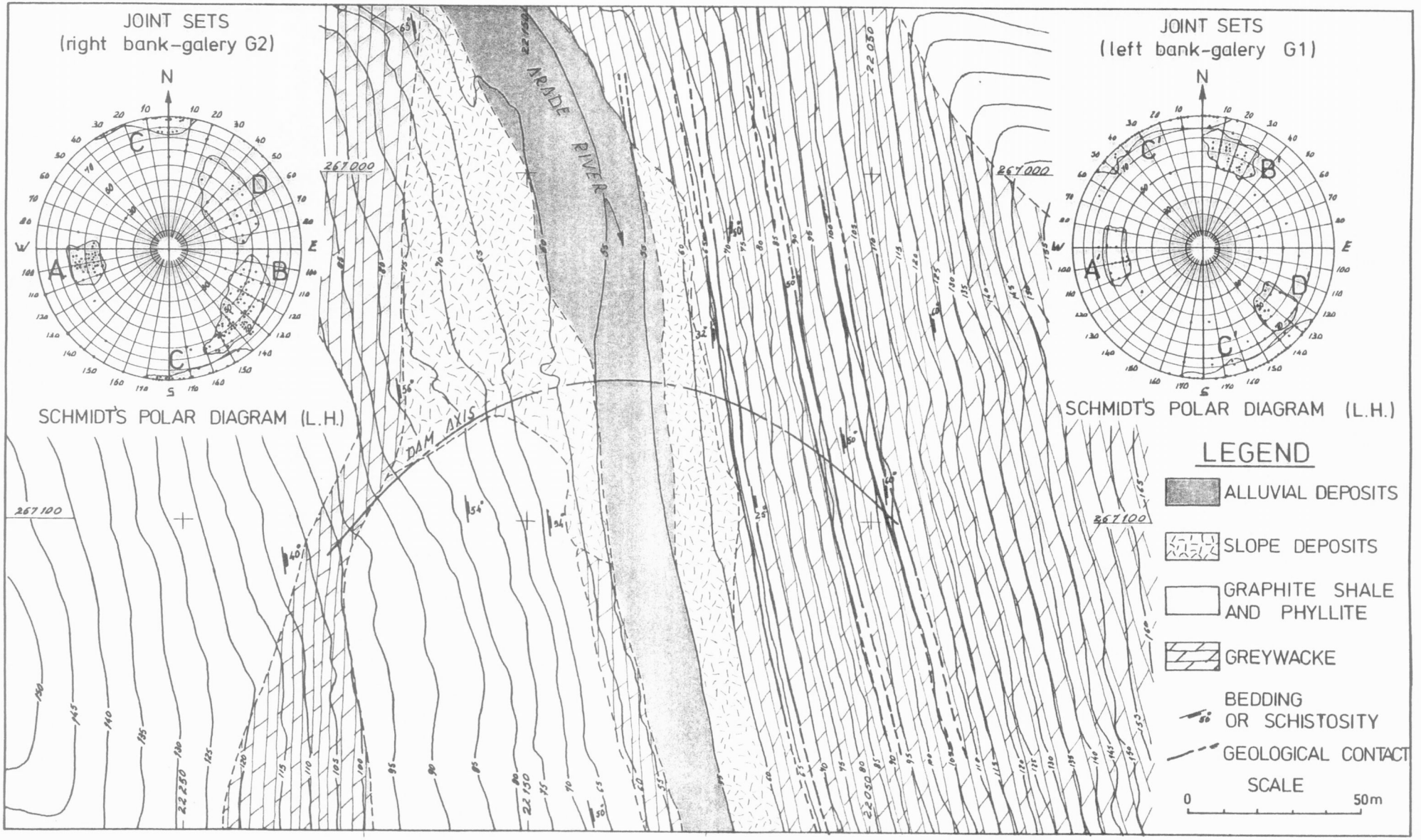


FIG. APPENDIX A8 – FUNCHO DAM SITE

SITE INVESTIGATIONS - DETAILS AND COST OF THE PERFORMED WORKS

PHASE 1 (1971)

1. Reconnaissance

PHASE 2 (1971 - 1972)

1. Trenches (Sep 1971 - May 1972)

	DESIGNATION	LENGTH (m)
RIGHT BANK	VD 1	88
	VD 2	88
LEFT BANK	VE 1	14
	VE 2	42
	VE 3	8
	VE 4	43

Total no of trenches	6
Total length	283 m
Average length	47 m
Cost	60 x 10^3 ESC (1971)

2. Boreholes (Jan - May 1972)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ.(o)	NO OF LUGEON TESTS
BOTTOM	4 PO	59	75	10
	5 PO	10.4	90	-
	6 PO	85.6	35	13
RIGHT BANK	1 j6	41	75	8
	2 j4	70	75	13
	2 P2	43.6	75	8
LEFT BANK	7 P1	36	75	6
	8 j3	48	75	9
	9 j5	43	75	8

Total no of boreholes	9
Total length	436.6 m
Average length	48.5 m
Total no of Lugeon tests	75
Cost	300×10^3 ESC (1972)

PHASE 3 (1974 - 1975)

1. Gallery (1974)

DESIGNATION	LENGTH (m)
LEFT BANK G1	30

Cost 200×10^3 ESC (1974)

2. Trenches (Mar 1975)

DESIGNATION	LENGTH (m)
RIGHT BANK	53

Cost 20×10^3 ESC (1975)

PHASE 4 (1976)

1. Galleries (Jan - Nov 1976)

DESIGNATION		LENGTH (m)
RIGHT BANK	G2	33
	G4	33
LEFT BANK	G1	11*
	G3	27.7

* Extension of the gallery performed in 1974

Total no of galleries	3
Total length	104.7 m
Cost	980 x 10^3 ESC (1976)

2. "In -situ" tests (Jun - Sep 1976)

2.1 Flat jack tests

Device used	2 LFJ - LNEC
Area loaded	2.3 m^2
No of loading cycles	3 -4
Maximum pressure applied	3 MPa (left bank) and 1 MPa (right bank)
Total no of tests	5

Results:

GALLERY	ROCK TYPE	E (MPa)
G 3	GREYWACKE	45 000
G 3	GREYWACKE	41 000
G 3	GREYWACKE	32 000
G 2	SHALE	1 300
G 2	SHALE	1 100

2.2 Dilatometer tests

Device used	BHD - LNEC
No of boreholes tested	12
Horizons measured in each borehole	2
Directions measured in each horizon	4 (greywackes) 8 (shales)
No of loading cycles	3 - 4
Maximum pressure applied	6 MPa (greywackes) 3 MPa (shales)
Total no of horizons measured	24

The tests were performed along shallow boreholes (2 m) drilled for this purpose inside the galleries G 2 and G 3. This made it possible for the dilatometer results to be compared with those obtained by means of the

large flat jacks.

Results:

GALLERY	ROCK TYPE	DILAT. MODULUS (MPa)		
		MAX.	MIN.	AVER.
G 1	GREYWACKE	23 400	2 300	8 300
G 3	GREYWACKE	36 800	5 100	17 500
G 2	SHALE	14 600	500	3 400

Cost of "in-situ" tests 1050 x 10^3 ESC (1976)

3. Laboratory tests (same period as above)

3.1 Uniaxial tests

Total no of tests 24 (12 greywacke)
(12 shales)

Type of samples: Prisms extracted from the galleries.
Area - 16 cm^2; height - 10 cm.
Aspect ratio - 2.5

ASSESSMENT OF E

No of loading cycles 3 (greywackes)
4 (shales)

Maximum pressure applied 25 MPa (greywackes)
5 or 10 MPa (shales)

Results:

ROCK TYPE	E (MPa)		
	MAX.	MIN.	AVER.
GREYWACKE	82 000	62 000	73 000
SHALE	51 000	8 000	25 000

ASSESSMENT OF σ_{uc}

Tests performed on the same samples, increasing pressure up to failure.

Results:

ROCK TYPE	σ_{uc} (MPa)		
	MAX.	MIN.	AVER.
GREYWACKE	180	90	137
SHALE	61	15	32

3.2 Rock shear tests

Total no of samples 11

Type of sample: Prisms cut from rock cores drilled inside the galleries

Area of shear 100 cm^2

Results:

GALLERY	ROCK TYPE	c (MPa)	Ø (°)	NO OF SAMPLES
G 3	GREYWACKE	2.90	69	4
G 3	GREYWACKE / /SHALE CONTACT	1.85	56	4
G 3	SHALE	1.05	42	3

3.3 Joint shear tests

Total no of sample 18

Type of samples: Prisms cut from samples collected at the foundation zone

Area of shear 100 cm^2

No of tests per sample 4

Results:

SAMPLES FROM GALLERY	ROCK TYPE	c (MPa)		Ø (o)		NO OF SAMPLES
		MAX	MIN	MAX	MIN	
G 3	GREYWACKE	0.52	0.21	44	30	12
G 2	SHALE	0.45	0.09	28	23	6

Cost of the laboratory tests 85×10^3 ESC (1976)

DAM NAME — TORRÃO

— GENERAL DATA —

LOCATION	—	TAMEGA RIVER; 3.5 KM UPSTREAM OF THE CONFLUENCE WITH THE DOURO RIVER - PORTUGAL
TYPE	—	CONCRETE ARCH DAM
TOTAL CREST LENGTH	—	213.6 m
CREST CHORD LENGTH	—	195 m
MAX. HEIGHT ABOVE FOUNDATIONS	—	62 m
CREST CHORD / HEIGHT RATIO	—	3.1
MAX. HEIGHT ABOVE THE RIVER BED	—	≃ 59 m
WIDTH OF THE VALLEY (bottom)	—	≃ 40 m
RESERVOIR CAPACITY (total)	—	$124 \times 10^6 m^3$
FLOODED AREA	—	≃ 600 ha
DAM VOLUME	—	$120 \times 10^3 m^3$
DATE OF THE PROJECT DESIGN	—	SEP 1969
DESIGN PERIOD	—	1968-1969
OWNER	—	ELECTRICIDADE DE PORTUGAL
DESIGNED BY	—	ELECTRICIDADE DE PORTUGAL (HIDROELECTRICA DO CÁVADO)
CONSTRUCTION COST (project estimate)	—	360×10^6 ESC
COST OF THE DESIGN	—	1.3×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	—	25 MONTHS
STATUS	—	TO BE CONSTRUCTED

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigations for this energy purpose dam were performed in two phase: phase 1 1968; phase 2 1968 - 1970.

1968 - Basic geological survey by field mapping

1968 - 1970 - A second comprehensive phase begun by the end of 1968, which comprised trenches, galleries, boreholes, geophysical exploration, "in - situ" and laboratory tests. A geotechnical assessment report was presented in August 1969, also forming part of the design issued in September 1969.

By the time the project design was issued, some of the investigation works had not been completed yet, namely the mechanical characterization tests and the second geophysical exploration programme. For the rock mass, a deformability modulus of E = 15 000 MPa and a Poisson ratio of 0.22 were adopted.

After the design, for some reasons (ve. an increase in power output) a change of the dam type to a gravity one was designed between 1978 - 1979. For this new design, two deformability moduli were used for the rock mass: 10 000 MPa and 20 000 MPa, being the lowest value used for the first 5 m beneath the foundation. A value of $\tan \phi = 0.6$ was adopted for the rock mass, this value being already corrected by a safety factor of 1.5, the cohesion being disregarded.

For the new design, a minor phase of site investigations was carried out, mainly dealing with the new dam's safety structures. The final location and the dam curvature radius were conditioned by the geological conditions, namely the state of weathering in the right bank and the faults crossing the foundation area.

Only the 1969 design and the related site investigations will be considered here.

FOUNDATION CONDITIONS

1. Rock Type

The lithology, at the site, is of pseudoporphyritic granite and coarse grain granodiorite (see Fig. Appendix A9). The topsoil is 0.2 m to 1.5 m thick. Sediments at the bottom of the valley are thin and consist of sandy gravels and some boulders.

2. Structure

Veins of several types cross the rock mass. The most common are aplite - pegmatite but biotite microgranite, pegmatite and quartz veins also occur.

The most important faults are represented in Fig. Appendix A9 as well as the main joint sets. Of the faults, the principal set is the one presenting an attitude of N (10^{o} - 40^{o}) E; (58^{o} - 80^{o}) SE and a fault space of 10 cm.

The geotechnical report also called the attention to some minor joint sets which could allow sliding due to their attitude. The fact that the valley has been excavated along the near W - E joint set was also pointed out.

3. State of weathering

The depth of weathering is greater in the right bank. The predicted depth to be excavated in order to reach a fresh to slightly weathered rock mass varied from 2 - 3 m at the bottom of the valley, to 8 - 9 m at the top of the left bank, and to about 20 m at the top of the right bank.

4. Permeability

Lugeon tests were performed systematically together with the borehole programme. The depths beneath which absorptions were less than one Lugeon varied from less than 10 m beneath the river bed to about 25 m and 50 m beneath the highest points of the right and left bank respectively.

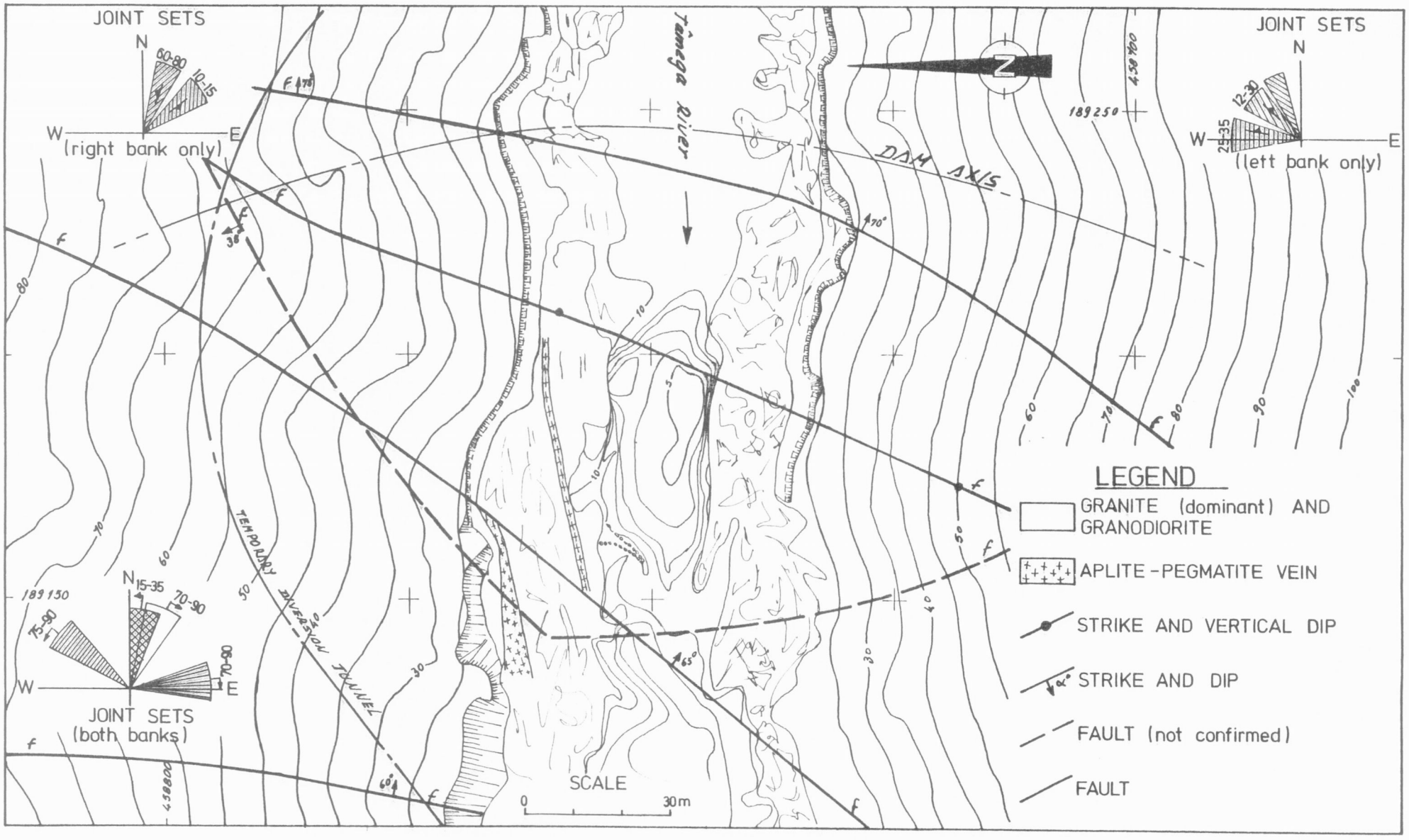


FIG. APPENDIX A9 - TORRÃO DAM SITE

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (1968)

1. Reconnaissance

PHASE 2 (1968 - 1970)

1. Trenches (end of 1968 - Aug 1969)

	DESIGNATION	LENGTH (m)
RIGHT BANK	TD 1	150
	TD 2	211
	TD 3	232
	TD 4	186
	TD 5	191

	DESIGNATION	LENGTH (m)
LEFT BANK	TE 1	216
	TE 2	169
	TE 3	201
	TE 4	65
	TE 5	52

Total no of trenches 10

Total length 1673 m

Average length 167.3 m

Cost (assessment, basedupon the total cost of trenches + galleries) 720×10^3 ESC (1969)

2. Galleries (same period as above)

	DESIGNATION	LENGTH (m)
RIGHT BANK	G4	13
	G10	20
	G18	23
	G17	22
	G19	5
	G16	33.5
	G9	10
	G5	30.5
	G15	34
	G11	22
	G14	62
	G3	17
	G2	13
	G13	26
	G12	15

	DESIGNATION	LENGTH (m)
LEFT BANK	G1	12
	G6	35.5
	G7	5
	G8	3

Total no of galleries	19
Total length	401.5 m
Average length	21.1 m
Cost (assessment, based upon the total cost of trenches + galleries)	1 730 x 10^3 ESC (1969)

3. Geophysical exploration (June 1969)

3.1 Apparent resistivity maps

2 apparent resistivity maps by means of 10 apparent resistivity profiles, each 180 m long, covering the same area but concerning two different depths of investigation (7 m and 14 m). The Schlumberger configuration was used, with $\overline{AB}$ = 21m and $\overline{MN}$ = 3 m (for the 7 m depth); $\overline{AB}$ = 39 m and $\overline{MN}$ = 3 m (for the 14 m depth)

3.2 Geoelectrical soundings

A total of 4 geoelectrical soundings were performed using a $\overline{MN}$ = = 5 m and a max $\overline{AB}$ of 180 m.

Cost of this geoph. exploration - 180 x 10^3 ESC (1969)

4. Boreholes (Jun - Oct 1969)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ.(α)	NO OF LUGEON TESTS
RIGHT BANK	S1	50	90	5
	S2	50	90	6
	S3	50	60	7
	S4	50	90	7
	S5	33.7	60	4
	S6	35	60	5
	S7	40	60	6
	S8	50	60	8
	S9	40	60	4
	S10	34.8	60	6
	S11	42.6	90	3
BOTTOM OF THE VALLEY	S12	60	43	10
	S14	55	43	9
LEFT BANK	S13	47.5	60	7
	S15	31	HORIZ.	4
	S16	31	HORIZ.	4
	S17	31.5	HORIZ.	4
	S18	30	60	4
	S19	30	60	4

Total no of boreholes	19
Total length	792.1 m
Average length	41.7 m
Total no of Lugeon tests	107
Cost	460 x 10^3 ESC (1969)

5. "In - situ" tests (Aug 1969 - Mar 1970)

5.1 Dilatometer tests

Device used	BHD - LNEC
No of boreholes tested	7
Horizons measured in each borehole	1 - 6 (aver 4)
Directions measured in each horizon	2
No of loading cycles	2 - 4
Maximum pressure applied	8 MPa
Total no of horizons measured	28

The tests were performed at depths varying between 10.5 m and 29.7 m.

Results:

DILATOMETER MODULUS (MPa)		
MAX	MIN	AVER
30 850	10 300	18 000

5.2 Rock shear tests

Total no of samples	9
Area of shear	.49 m^2

Results:

GALLERY	c (MPa)	Ø (o)	NO OF SAMPLES
G 10	4.0	62	3
G 14	4.6	60	3
G 15	4.5	61	3

5.3 Joint shear tests

Total no. of samples	3
Area of shear	.49 m^2
No. of tests per sample	3

Results:

GALLERY	c (MPa)	∅ (o)
G 10 *	0.36	42
G 15 *	0.39	59
G 15	0.10	42

* Artificial joint

Cost of the "in - situ" tests - 1 200 x 10^3 ESC (1969)

6. Laboratory tests (same period as above)

6.1 Uniaxial tests

Total no of tests	32
Type of sample: cut from rock cores	
Diameter	60 mm
Aspect ratio	2.5

ASSESSMENT OF E

No. of loading cycles	2
Maximum pressure applied	20 MPa

Results:

E (MPa)			NO OF SAMPLES
MAX	MIN	AVER	
72 200	12 600	35 000	32

ν			NO OF SAMPLES
MAX	MIN	AVER	
0.20	0.06	0.13	7

ASSESSMENT OF σ_{uc}

Tests performed on the same samples, increasing pressure up to failure.

Results:

σ_{uc} (MPa)			NO OF TESTS
MAX	MIN	AVER	
138	56	104	32

6.2 More uniaxial tests

These tests were carried out to study the anisotropy of the rock material.

Total no of tests	18
Type of samples: prisms	
Cross sectional area	42.25 cm^2
Height	17 cm
Aspect ratio	2.6

ASSESSMENT OF E

No of loading cycles	2
Maximum pressure applied	12 MPa

Results:

ROCK TYPE	E (MPa)			NO OF TESTS
	MAX	MIN	AVER	
FRESH GRANITE (LOCAL A)	51 000	32 800	43 300	9
WEATHERED GRANITE (LOCAL B)	25 000	11 600	17 700	9

ASSESSMENT OF σ_{uc}

Tests performed on the same samples, increasing pressure up to failure.

Results:

ROCK TYPE	σ_{uc} (MPa)			NO OF TESTS
	MAX	MIN	AVER	
FRESH GRANITE LOCAL A	130	89	111	9
WEATHERED GRANITE (LOCAL B)	71	50	60	9

6.4 Rock shear tests

Total no of samples 13

Type of samples: prisms collected from inside the galleries

Area of shear 150 - 300 cm^2

Results:

GALLERY	c (MPa)	Ø (o)	NO OF SAMPLES
G 10 (LOCAL A)	4.8	62	5
G 10 (LOCAL B)	2.9	62	4
G 15	5.8	62	4

6.5 Joint shear tests

Total no of samples 6

Type of samples: prisms cut from inside the galleries

Area of shear 150 - 300 cm^2

No of tests per sample 4

Results:

GALLERY	c (MPa)	Ø (o)
G 10 (A)	0.03	41
G 10 (B)	0.07	40
G 10 (C)	0.27	55
G 15 (A)	0.34	40
G 15 (B)	0.08	35
G 15 (C)	0.14	45

6.6 Ultrasonic wave velocities

Type of samples: from rock cores
Length of samples 120 - 150 mm
No. of samples tested 6
Measured velocities V_p

Results:

Values of V_p varied from 3 300 to 4 100 m/sec.

The dynamic elasticity modulus was assessed for two of the above samples, which were also submitted for uniaxial compressive tests.

Results:

ROCK TYPE	STATIC ELASTICITY MODULUS E_e (MPa)	STATIC POISSON'S RATIO	DINAMIC ELASTICITY MODULUS E_d (MPa)	E_d/E_e
MODERATELY WEATHERED GRANITE	20 270	0.14	27 870	1.37
FRESH GRANITE	33 800	0.14	40 960	1.21

Cost of laboratory tests - 100×10^3 ESC (1969)

7. Geophysical exploration (Apr 1970)

7.1 Seismic refraction profiles

A total of 6 seismic refraction profiles, each 60 mm long, were performed.
Total length of profiles - 360 m

7.2 Seismic fans

A total of 17 seismic fans were performed:
a) between galleries
b) between one gallery and the surface
c) between one borehole and the surface

Cost of geophysical exploration - 110×10^3 ESC (1970).

DAM NAME — JAMBA - IA - MINA

— GENERAL DATA —

LOCATION	— CUNENE RIVER; HUÍLA; ANGOLA
TYPE	— MIXED (CONCRETE GRAVITY + EARTH) DAM
TOTAL CREST LENGTH	— TOTAL - 1 467 m CONC. - 937 m EARTH - 530 m
MAX. HEIGHT ABOVE FOUNDATIONS	— 23.5 m
MAX. HEIGHT ABOVE THE RIVER BED	— ≈ 20 m
WIDTH OF THE VALLEY (bottom)	— 700 m
RESERVOIR CAPACITY (total)	— 568.1 x $10^6 m^3$
FLOODED AREA	— 7 180 ha
DAM VOLUME	TOTAL - 420 x $10^3 m^3$ CONC. - 172 x $10^3 m^3$ EARTH - 248 x $10^3 m^3$
DATE OF THE PROJECT DESIGN	— OCT 1974
DESIGN PERIOD (includes a prel. study)	— 1973-74
OWNER	— GANINETE DO PLANO DO CUNENE - ANGOLA
DESIGNED BY	— COBA, SARL - LISBOA
CONSTRUCTION COST (project estimate)	— 300 x 10^6ESC
COST OF THE DESIGN (includes the prel. study)	— 16 x 10^6ESC
CONSTRUCTION PERIOD (project estimate)	— 4 YEARS
STATUS	— TO BE CONSTRUCTED

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigations at Jamba - Ia - Mina dam site were performed in two phases: phase 1 1967, phase 2 1971.

1967 - The site investigations began with a first geological survey by field mapping.

1971. - Second phase of site investigations which consisted of geophysical exploration and boreholes. A geotechnical assessment report was produced this year.

The above mentioned site investigations were carried out before the consultant was charged with the responsibility of both the preliminary study and the design itself. The existing information was reassessed and a second geotechnical assessment report was produced forming part of the preliminary study in January 1974. When the design was released in October 1974, a final geotechnical report was also included.

The dam is for energy purposes. It is of a mixed type - a concrete buttress section and an earth section. The spillway is of a crest type in the concrete section. The bottom discharge is a 3 x 3.75 m^2 vent through the bottom of the concrete section, near the left bank. The scheme is completed by an important hydraulic circuit which comprises of a water intake, a 6 581 m long tunnel, and an underground power station to generate 3 x 63 Mw.

It must be said that a 3rd phase of site investigations by means of geophysical exploration and boreholes was performed in 1973, but it dealt exclusively with the very long tunnel and the power house.These works will not be taken into account here.

Neither "in - situ" nor laboratory tests were performed.

For the concrete section of the dam, a deformability modulus for the rock mass of 21 000 MPa, i.e. equal to that of the concrete used in the design, and a friction coefficient for the concrete - rock contact of tan ϕ = 1 was adopted.

FOUNDATION CONDITIONS

1. Rock Type

The rock type present over all the site is a medium to coarse monzonite granite. Residual soils overlie this granite with special importance in the highest points of the banks. In the bottom of the valley the rock crops out or, the residual soil is not thicker than 2 m, and the alluvial complex is of small importance at the site of the dam foundations (see Fig. Appendix A10).

2. Srtucture

The granitic rock mass is highly jointed. Two main alignments were detected by studying aerial photographs but it was concluded that they are not connected with faults, but with two main joint sets, one being parallel the other perpendicular to the river direction (see Fig. Appendix A10); another less numerous subhorizontal set, dipping southwards was also assessed. Some of the joints are ocupied by crushed veins.

3. State of Weathering

The rock mass was classified as completely weathered on both sides of the valley. The depth of this material increases from the lower part of the banks to their highest points, reaching 10 m in the left bank. Beneath that the rock mass was classified as fresh.

The anticipated depths for the excavations were 3 - 4 m for the concrete section of the dam and 1 m for the earth section.

4. Permeability

The Lugeon tests performed together with the boreholes led to the conclusion that the rock mass is, almost, impervious. However, loss of water was noticed during the tests, through open joints or through joints filled by crushed veins, especially in some of the zones dealing with the concrete section of the dam. Following this, grout treatment was suggested down to depths varying between 10 and 30 m, i.e. until depths where absorptions are not greater than 3 l/min. m at pressures of 1 MPa, to be assessed by means of Lugeon tests performed in borehole sections of 5 m.

The assessed safety coefficient against sliding was 1.57 and the one against turnover was 1.95. A safety coefficient against turnover of 1.85 was obtained after corrections for earthquake effects.

The presence of a near horizontal joint set was thought capable of causing problems for the dam's stability. Nevertheless it was considered that the grout treatment which was suggested would eliminate the problem.

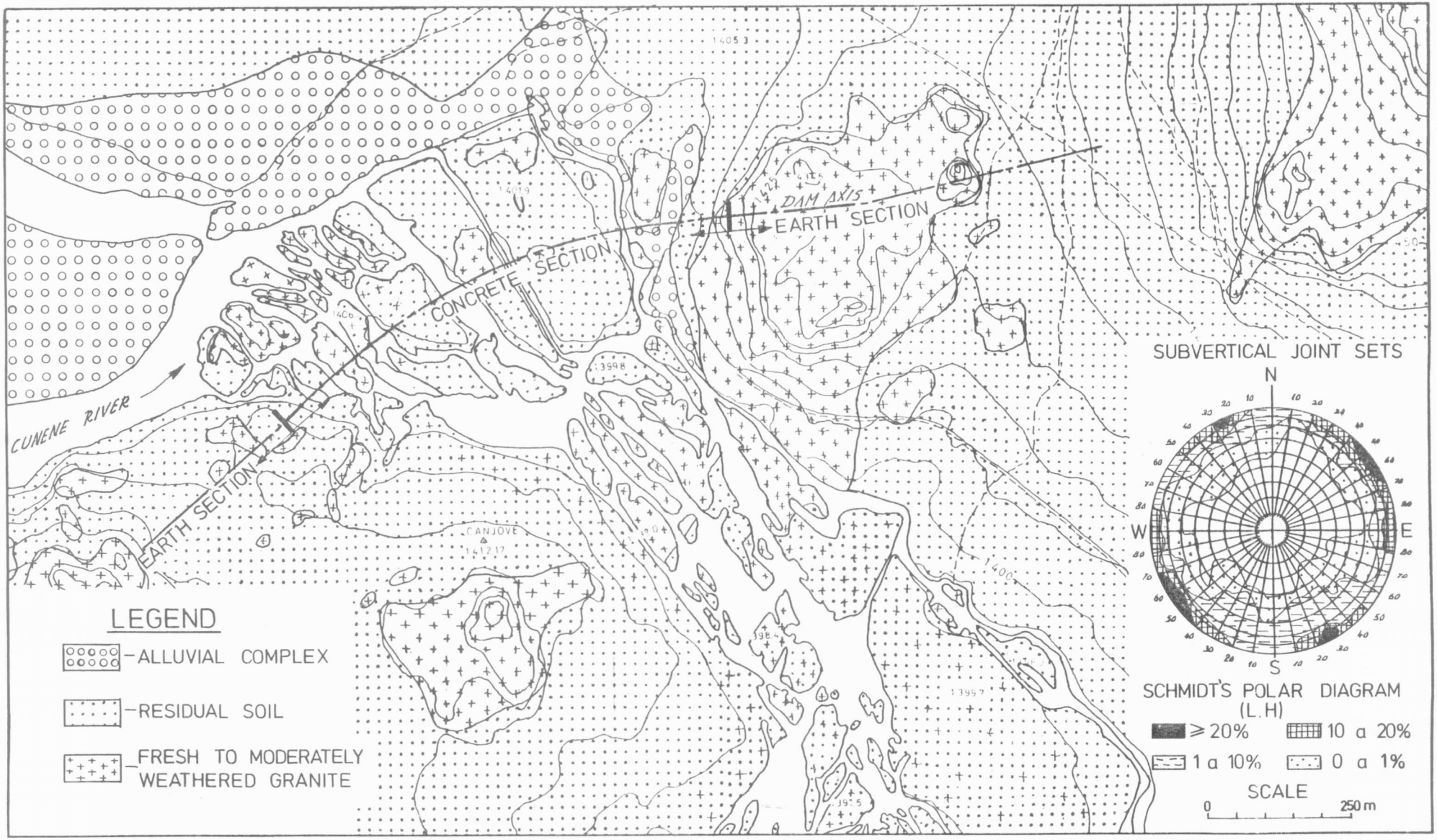


FIG. APPENDIX A10 - JAMBA-IA-MINA DAM SITE

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (1967)

1. Reconnaissance

PHASE 2

1. Boreholes (Feb 1971 ⟶ ?)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{(0)}$	NO OF LUGEON TESTS
RIGHT BANK	Si 1	30.00	45	5
	Sv 1	24.50	90	4
	Sv 2	31.00	90	5
	Si 3	45.00	45	9
	Si 3A	50.00	40	10
BOTTOM OF THE VALLEY	Si 4A	51.50	40	-
	Si 8A	50.50	40	10
	Si 4	55.00	40	10
	Sv 5	31.50	90	5
	Si 5	39.00	45	8
	Si 6A	38.00	50	6
	Si 6	39.00	40	7
	Sv 7	30.50	90	4
LEFT BANK	Si 7	30.00	45	5
OTHERS	Sv 9	32.50	90	4
	Si 8	30.00	45	5
	Si 2	31.50	45	5

Total no of boreholes 17

Total length 639.5 m

Average length 37.6 m

Total no of Lugeon tests 102

Cost 1050×10^3 ESC (1971)

2. Geophysical exploration (Jun - Jul 1971)

2.1 Apparent resistivity profiles

6 apparent resistivity profiles, each 1 300 m long were performed. The results were used to produce an apparent resistivity map of both banks.

Total length of profiles - 7 800 m

$\overline{AB}$ = 180 m; $\overline{MN}$ = 5 m

2.2 Geoelectrical soundings

A total of 6 geoelectrical soundings were carried out on the left bank.

MAX $\overline{AB}$ = 180 m; $\overline{MN}$ = 5 m

2.3 Seismic refraction profiles

79 seismic refraction profiles, each 65 m long, were performed, some of them along the same alignments as the apparent resistivity profiles. A 12 geophone array was used, with direct and reverse shots.

Total length of profiles - 5 135 m

Cost of geophysical exploration - 280 x 10^3 ESC (1971)

DAM NAME — JAMBA - IA - OMA

— GENERAL DATA —

LOCATION	— CUNENE RIVER; HUÍLA; ANGOLA
TYPE	— MIXED (CONCRETE GRAVITY + EARTH) DAM
TOTAL CREST LENGTH	TOTAL - 2 940.5 m CONCR.- 490.5 m EARTH - 2 450.0 m
MAX. HEIGHT ABOVE FOUNDATIONS	— 46.2 m
MAX. HEIGHT ABOVE THE RIVER BED	— 41 m
WIDTH OF THE VALLEY (bottom)	— 250 m
RESERVOIR CAPACITY (total)	— 1088.3×10^6 m^3
FLOODED AREA	— 11 010 ha
DAM VOLUME	TOTAL - $1\,788 \times 10^3 m^3$ CONCR.- $348 \times 10^3 m^3$ EARTH - $1\,440 \times 10^3 m^3$
DATE OF THE PROJECT DESIGN	— MAR 1976
DESIGN PERIOD (includes a prel. study)	— 1975-76
OWNER	— GABINETE DO PLANO DO CUNENE - ANGOLA
DESIGNED BY	— COBA, SARL - LISBOA
CONSTRUCTION COST (project estimate)	— 350×10^6 ESC
COST OF THE DESIGN (includes the prel. study)	— 7.8×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	— 5 YEARS
STATUS	— TO BE CONSTRUCTED

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigations for the Jamba - Ia - Oma dam site were performed in 3 phases: phase 1 1964, phase 2 1970 - 1971 and phase 3 1975.

1964 - Site investigations began with a geological survey by field mapping.

1970 - 1971 - A second phase took place which consisted of a borehole programme. A first geotechnical assessment report was issued, taking into account the works performed until then.

1975 - Last phase of site investigations comprising of geophysical exploration and boreholes. A geotechnical assessment report was produced which formed part of a preliminary design report issued in June 1975, and a final one was released with the main design report in March 1976.

The information gathered from site investigations performed for a neighbouring dam site (Jamba - Ia - Mina), of the same geological formation, was taken into account in the present studies, according to the geotechnical report.

The deformability modulus of the rock mass adopted in the design was 10 000 MPa (1/2 of the elasticity modulus of the concrete), with a Poisson ratio of 0.2. The friction coefficient adopted for the concrete - - rock mass contact was tan $\phi = 1$.

The main design report also quotes safety coefficients against sliding along the foundation as varying between 2.28 and 3.45, and a safety coefficient against turnover of 1.8.

The dam is for energy purposes, and it is of a mixed type (earth section + concrete buttress section - see Fig. Appendix A11). The spillway is of a crest type. The bottom discharge (a 2.20 x 3.10 m^2 vent) and a wa-

ter intake were also designed in the concrete section of the dam. A dam foot type power house to drive 2 x 25 Mw, completes the scheme. All these structures are on the right bank.

FOUNDATION CONDITIONS

1. Rock Type

The dam site is composed of a granitic rock mass overlain by residual soils over large areas. The granite crops out in some areas of the bottom of the valley.

2. Structure

Some structural alignments were observed by studying aerial photographs, being grouped into two sets with strikes N (5^{o} - 30^{o}) E and N (20^{o} - 60^{o}) W; some of them were confirmed later as faults or crushed zones, either by inspection or by exploration works (see Fig. Appendix A11).

The rock mass is highly jointed, the joints showing a large scatter in attitude. The main sets assessed are presented in Fig. Appendix A11. The grout treatment suggested in the geotechnical report was expected to eliminate the sliding trend especially along the existing near horizontal set.

3. State of weathering

The depth of residual soils can reach a maximum of 35 m in both banks and beneath that depth an average of 8 m of jointed, moderately weathered rock occurs. Fresh granite exists thereafter.

For the concrete section of the dam, the anticipated depth to be excavated was 5 - 8 m beneath the residual soils which when they are present in this zone reach an average thickness of 7 - 8 m, except in a particular region where they reach about 15 m. The earth section of the dam will be founded within the residual soils, after an excavation of about 1 m.

4. Permeability

According to the geotechnical report the depth to the water table measured during the borehole programme was between 2.5 and 4.5 m.

The Lugeon tests performed together with the boreholes gave hi-

gh absorptions whereas for the fresh granite they were equal to or lower than 1 Lugeon. A grout treatment was suggested to reach the impervious rock mass. This treatment should be extended throughout the banks up to the zones were the height of the dam is less than 8 m.

Pressure - permeameter tests performed in the residual soils gave permeability coefficients of between 10^{-4} cm /sec and 6×10^{-5} cm /sec.

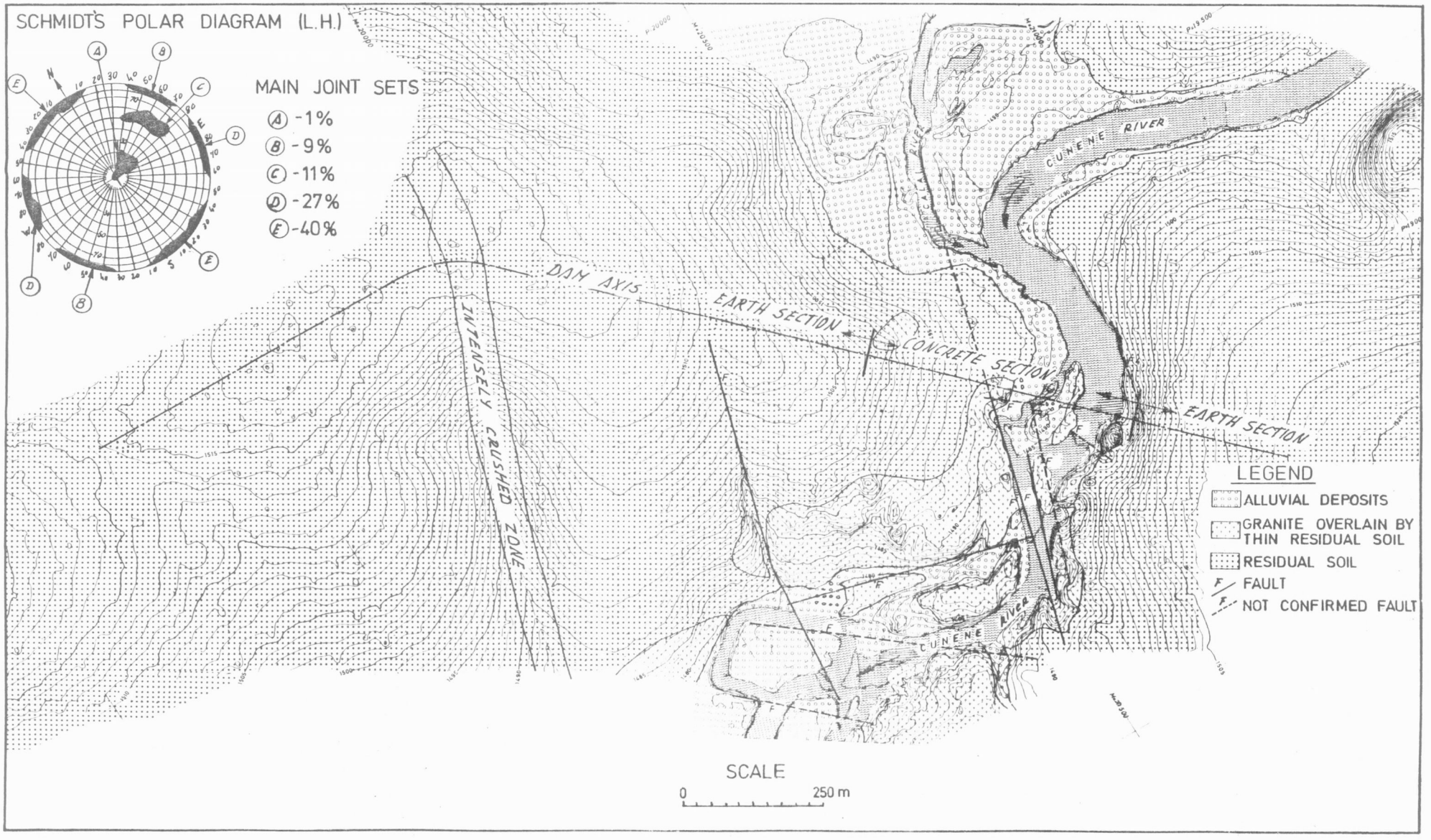


FIG. APPENDIX A11 - JAMBA-IA-OMA DAM SITE

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (1964)

1. Reconnaissance (? 1964)

PHASE 2 (1970 - 1971)

1. Boreholes (Oct 1970 - Apr 1971)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{(0)}$	NO OF LUGEON TESTS
RIGHT BANK	S 1	40.80	90	3
	S 2	23.60	90	2
	S 3	43.80	90	2
	S 5	52.60	90	5
	S 7	46.00	90	2
	S 8	38.00	90	3
	S 9	38.00	45	3
	S 12	40.00	90	3
	S 14	39.60	90	5
	S 15	42.20	45	7
	S 16	43.60	45	8
	S 17	35.00	60	7
LEFT BANK	S 19	37.20	90	2
	S 20	39.40	90	2
	S 21	41.00	90	2

Total no of boreholes	15
Total length	600.8 m
Average length	40 m
Cost	(see below)

2. Pressure-permeameter tests

10 tests were carried out on the residual soil, in the upper zones of 4 boreholes. The assessed permeability coefficients varied from 10^{-4} to 6×10^{-5} cm /sec.

Cost (included in the cost of boreholes - see below)

3. More boreholes (same period as above)

DESIGNATION		LENGTH (m)	INCLINATION TO THE HORIZ$_{(o)}$	NO OF LUGEON TESTS
OTHERS	SI 13	22.00	90	3
	SI 22	25.60	90	2
	SI 23	16.00	90	2
	SI 24	44.80	45	7

Total no of boreholes 4

Total length 108.4 m

Total no of Lugeon tests 14

Cost (all boreholes of phase 2) 970 x 10^3 ESC (1971)

PHASE 3 (1975)

1. Geophysical exploration (Feb 1975)

1.1 Seismic refraction profiles

5 seismic refraction profiles each 65 m long were performed perpen icular to the dam axis.

8 seismic refraction profiles each 120 m long were performed in o- ther zones.

Total length of profiles - 1285 m

Cost of geophysical exploration 180 x 10^3 ESC (1975)

2. Boreholes

DESIGNATION		LENGTH (m)	INCLINATION TO THE HORIZ$_{(o)}$	NO OF LUGEON TESTS
RIGHT BANK	SA 1	17.55	90	2
	SA 2	25.00	90	-

Cost 150 x 10^3 ESC (1975)

DAM NAME — TOUVEDO

— GENERAL DATA —

LOCATION	—	LIMA RIVER; PONTE DA BARCA; PORTUGAL
TYPE	—	CONCRETE GRAVITY DAM
TOTAL CREST LENGTH	—	155.3 m
MAX. HEIGHT ABOVE FOUNDATIONS	—	48 m
MAX. HEIGHT ABOVE THE RIVER BED	—	37.4 m
WIDTH OF THE VALLEY (bottom)	—	≃ 30 m
RESERVOIR CAPACITY (total)	—	27.4×10^6 m^3
FLOODED AREA	—	225 ha
DAM VOLUME	—	131.5×10^3 m^3
DATE OF THE PROJECT DESIGN	—	DEC 1978
DESIGN PERIOD	—	1977 - 1978
OWNER	—	ELECTRICIDADE DE PROTUGAL
DESIGNED BY	—	ELECTRICIDADE DE PORTUGAL
CONSTRUCTION COST (project estimate)	—	490×10^6 ESC
COST OF THE DESIGN	—	3×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	—	2.5 YEARS
STATUS	—	TO BE CONSTRUCTED

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigations for Touvedo dam were performed in two phases: phase 1 1977, phase 2 1977 - 1978.

1977 - Basic geological survey by field mapping, which led to the preparation of a first geotechnical assessment report.

1977 - 1978 - A major phase of site investigations comprising trenches and boreholes. A final geotechnical assessment report was produced in July 1978, which also formed part of the main design report released in December 1978.

For reasons based on the geotechnical conditions, this site was selected among 5 suggested ones. The comparison between these sites was made based upon the results of a first reconnaissance conducted for all of them, in some cases followed also by a small number of trenches and boreholes. The geotechnical assessment report compared the geological conditions at the five sites. For example, the first site considered (S. Miguel), would have increased the costs of both excavations and concrete by about 50% in relation to the actual site, due to the worse state of weathering exhibited by the rock mass near the surface.

In the actual site, two very close locations for the dam were investigated; the decision was made taking into account the locations of some faults and the volume of weathered rock mass which would be excavated.

The scheme which is for energy production, comprises the dam, a crest spillway, two bottom discharges under the dam body, a power house (a dam foot type which will drive 30Mw a and a small hydraulic circuit between a water intake set on the left bank and the power house located on the same bank.

No mechanical characterization tests were performed during the site investigations. The values adopted in the design were extrapolated from

the tests carried out for an abandoned dam site, 14 km upstream, near Alto Lindoso dam site, which had been studied in 1962. The values used for the design of the Alto Lindoso dam had been extrapolated from the same tests as well.

While for the Alto Lindoso dam a rock mass deformability modulus of 20 000 MPa was used, the value adopted for the rock mass at Touvedo dam site was 15 000 MPa, to take into account the more weathered and decompressed state of the rock mass here. The assumed Poisson coefficient was equal to 0.2.

The adopted shear strength parameters were based on the laboratory tests carried out for the actual Alto Lindoso dam site, performed in 1977, which were: $c = 0.24$ MPa and $\phi = 37^{\circ}$. In the calculations, $\tan \phi$ was taken as 0.6, the cohesion being disregarded.

FOUNDATION CONDITIONS

1. Rock Type

The dominant rock type is a medium to coarse muscovite - biotite granite, sometimes with a migmatitic character, and includes mica schist xenoliths . Alluvium filling the bottom of the valley presents small thickness, being comprised of granitic sand and some boulders.

2. Structure

Aplite - pegmatite, pegmatite, and less frequently fractured quartz veins cross the rock mass. Some faults were assessed at the site, their attitude and fault spaces being represented in Fig. Appendix A12. They are almost filled by a mylonitic material in a clay matrix. The rock mass is very highly jointed, more intensively on the left bank and at the bottom of the valley.An extensive study of the joint and vein attitude give the results presented in Fig. Appendix A13.

3. State of weathering

At the surface, the state of weathering varies from slightly to highly weathered; the left bank is more weathered than both the bottom of the valley and the right bank; this is due to the more intense jointing also present in this bank. Based on the variation in the state of weathering with depth, the minimum expected excavation depths were, according with the geotechnical report, 4 - 6 m in the left bank, 10 m at the bottom of the valley and about 3 m in the right bank.

4. Permeability

The depth of the water table increases from the bottom of the valley towards the top of the banks, where it reaches more than 15 m. The assessment of rock mass permeability was done on the basis of the results of Lugeon tests performed together with the boreholes. The depth beneath which the absorptions were less than 1 Lugeon was calculated and a grout curtain was suggested, which should reach more than 10 m below the excavation levels.

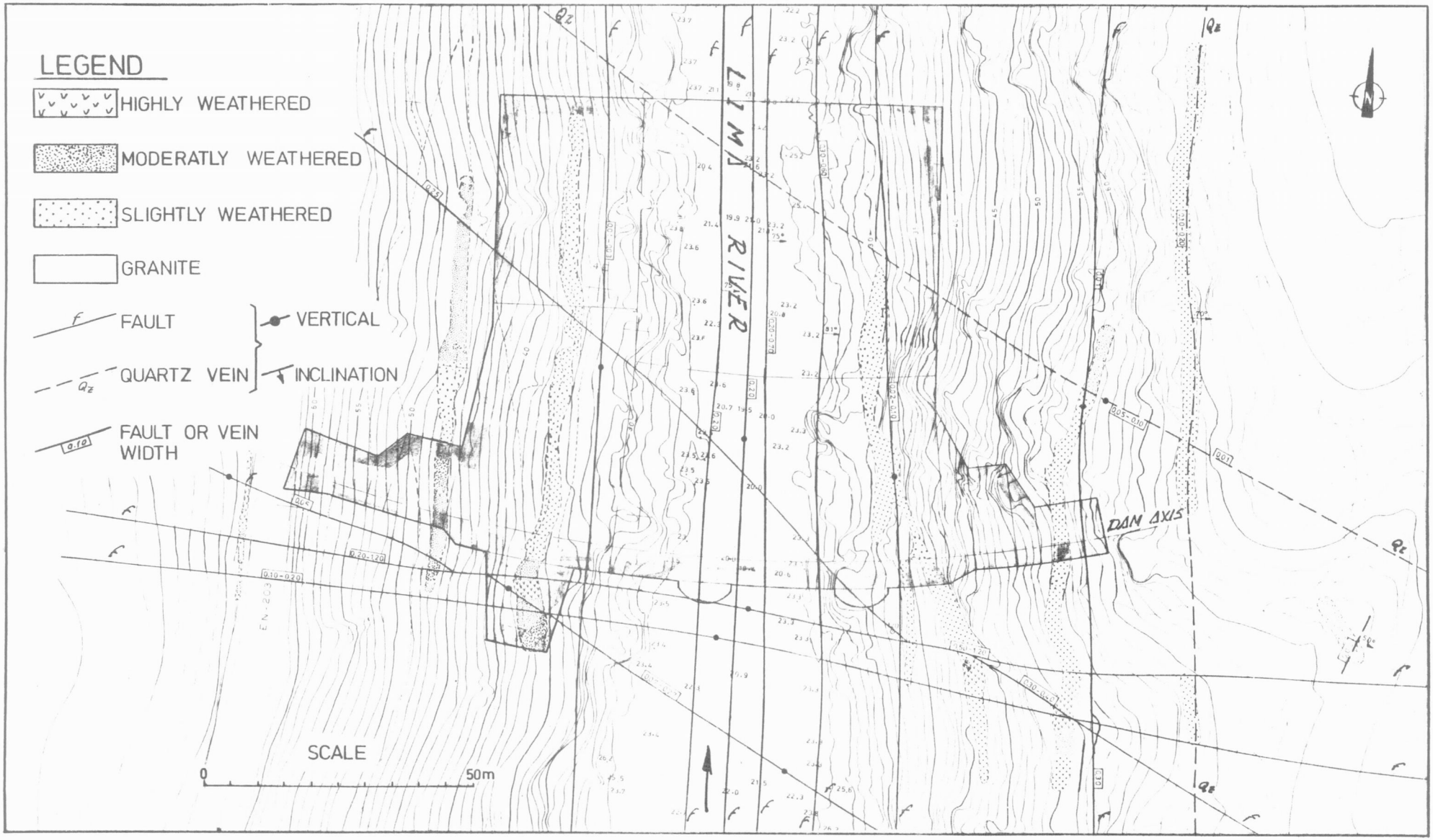


FIG. APPENDIX A12 – TOUVEDO DAM SITE

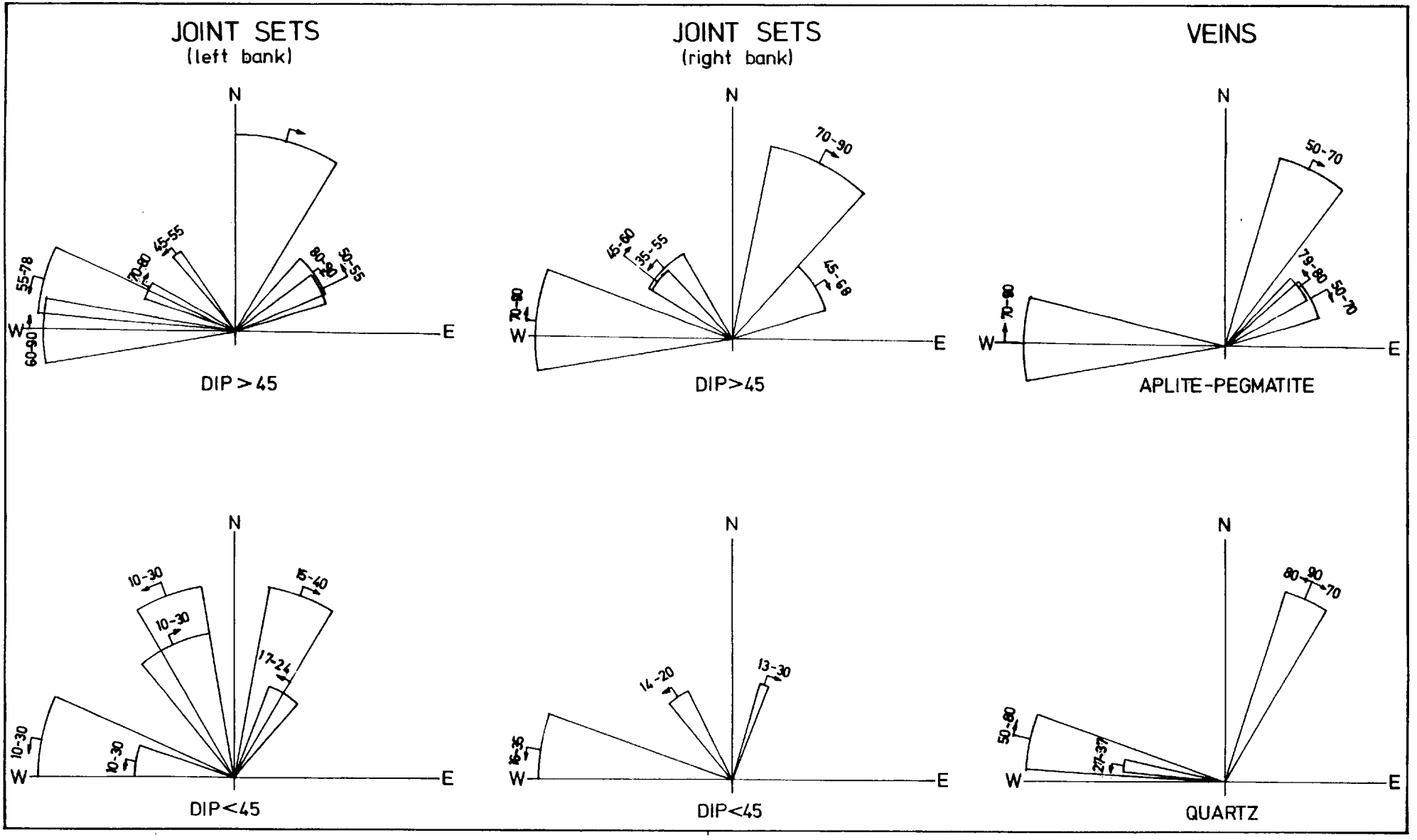


FIG. APPENDIX A13–TOUVEDO DAM SITE (structural elements)

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (1977)

1. Reconnaissance (mid 1977)

PHASE 2 (1977 - 1978)

1. Trenches (Aug - Oct 1977)

	DESIGNATION	LENGTH (m)
RIGHT BANK	TD 1	100
	TD 2	88
	TD 3	88
	VD 1	13
	VD 2	15
	BE	58
LEFT BANK	TE 1	78
	TE 2	103

Total no of trenches	8
Total length	543 m
Average length	67.9 m
Cost	230 x 10^3 ESC (1977)

2. Boreholes (Aug 1977 - Mar 1978)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{(o)}$	NO OF LUGEON TESTS
RIGHT BANK	SD- 1	30.5	45	5
	SD- 2	31.4	45	5
	SD- 3	30.1	45	4
	SD- 4	25.1	45	4
	SD- 5	81.5	45	15
	SD- 6	40.0	60	7
	SD- 7	72.5	45	13
	SD- 8	72.7	45	9
	SD- 9	65.1	60	12
	SD- 10	38.0	60	6
	SD- 11	10.6	90	1

... /...

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ.(o)	NO OF LUGEON TESTS
LEFT BANK	SE- 1	37.3	45	-
	SE- 2	41.1	45	7
	SE- 3	36.2	45	6
	SE- 4	60.1	45	11
	SE- 5	30.5	45	5
	SE- 6	26.6	45	4
	SE- 7	70.0	45	10
	SE- 8	75.2	45	13
	SE- 9	35.5	45	6
	SE-10	30.8	45	3
	SE-11	55.6	45	10
	SE-12	35.1	45	6

Total no of boreholes	23
Total length	1 032 m
Average length	44.8 m
Total no of Lugeon tests	162
Cost	$1\ 430 \times 10^3$ ESC (1977)

DAM NAME — RAIVA

— GENERAL DATA —

LOCATION	—	MONDEGO RIVER; PENACOVA; COIMBRA; PORTUGAL
TYPE	—	CONCRETE GRAVITY DAM
TOTAL CREST LENGTH	—	200 m
MAX. HEIGHT ABOVE FOUNDATIONS	—	34 m
MAX. HEIGHT ABOVE THE RIVER BED	—	≃ 25 m
WIDTH OF THE VALLEY (bottom)	—	100 m
RESERVOIR CAPACITY (total)	—	21×10^6 m^3
FLOODED AREA	—	300 h a
DAM VOLUME	—	70×10^3 m^3
DATE OF THE PROJECT DESIGN	—	OCT 1974
DESIGN PERIOD	—	1972 - 1974
OWNER	—	ELECTRICIDADE DE PORTUGAL
DESIGNED BY	—	ELECTRICIDADE DE PORTUGAL
CONSTRUCTION COST (project estimate)	—	200×10^6 ESC
COST OF THE DESIGN	—	4.2×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	—	2 YEARS
STATUS	—	CONSTRUCTED

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigation were performed in 4 phases: phase 1 1972, phase 2 1972, phase 3 1973, phase 4 1974.

1972 - The site investigations commenced with a basic geological survey by field mapping.

1972 - Second phase of site investigations which consisted of percussion drills.

1973 - Trenches, a special study of jointing and geophysical exploration were carried out. Based upon the performed works, a first geotechnical assessment report was issued in November 1973.

1974 - Final phase of investigation by means of a comprehensive borehole programme. A final geotechnical assessment report was released together with the main design report in October 1974.

It seems to be worthwhile to mention that for the 4th phase of site investigations, only 9 boreholes had been programmed in the first instance. However, because of problems related to the existence of faults which needed to be clarified, 10 more boreholes were drilled for this purpose. This led to the drilling of an extra 288.3 m, - more than 50% of the total borehole length drilled at the site.

Neither "in - situ" nor laboratory mechanical characterization tests were performed, certainly due to the knowledge gathered from other sites of similar geological formations.

In accordance with the stability calculations presented in the design report, the assessed sliding coefficient along horizontal surfaces (relation between the horizontal stresses which can create the sliding and the vertical stresses which mobilise friction to resist sliding -

- $\frac{\sum \text{hor. stress}}{\sum \text{vert. stress}}$) varied between 0.67 and 0.78 m. The safety coefficient against turnover assessed in that report was between 1.54 and 1.89 m.

The dam which is for energy and irrigation purposes, has a crest spillway and a bottom discharge (a rectangular shaped outlet) with a cross sectional area of 2.0 x 1.5 m^2 underthe central part of the dam. A power house (a dam foot type) at the left bank which will drive 18 Mw also forms part of the scheme.

FOUNDATION CONDITIONS

1. Rock Type

The lithology of the site consists of graphite phyllites, greywackes and intermediate rocks, belonging to the Portuguese ante - Ordovician complex (see Fig. Appendix A14).

Sandy alluvial deposits with gravels and boulders fill the bottom of the valley in an area which corresponds to the flood plain.

Slope deposits occur, overlying the rock mass, especially at the right bank where their thickness exceeds 2 m at the highest levels.

2. Structure

The site is severely folded and faulted, and the valley could have been excavated along the alignment of some major failure of this type. Folds are generally tight, sometimes of the isoclinal type. The most frequent faults constitute a set with attitude N (65^{o} - 90^{o}) E; (45^{o} - 70^{o}) SE or S and with fault spaces varying generally from 0.03 m to 0.20 m. One of these faults, with a fault space of 5 - 6 m crosses the foundation zone at the upper part of the right bank.

Three sets of quartz veins were assessed crossing the whole rock mass, their width generally not greater than 0.20 m. One of them, 1 - 1.5 m wide, and a 1.5 m wide dolerite dyke cross the foundation zone (see Fig. Appendix A14).

The general attitude of schistosity is N 70^{o}W to W-E or to N 80^{o}E, dipping 65^{o} N to 70^{o} S.

The geotechnical report refers to the following main joint sets:

SET	STRIKE (°)	DIP (°)
A	N (20-60) W	(45-72) NE
B	N (20-50) W	RIGHT BANK (12-35) SW
		LEFT BANK (40-80) SW
C	N (0 -40) E	(35-60) E OR SE
D	N 12E-N10W	65W to VERT. to 50E
E	N (80-90) E	(45-65) N

A set parallel to the schistosity and another situated subhorizontal were assessed as well.

3. State of weathering

The rock mass is highly weathered at the surface, the improvement in depth varying along the dam axis. The excavation depths suggested by the geotechnical report varied from 2.5 - 6m beneath the river sediments, to 5 - 11 m at the left bank and to 12 - - 20 m on the right bank.

4. Permeability

Lugeon tests were performed together with the boreholes, using pressure steps of 0.25; 0.5; 1; 0.5; 0.25 MPa along borehole sections of 5 metres. From the results grouting was recommended, which should reach a minimum depth of 35 m.

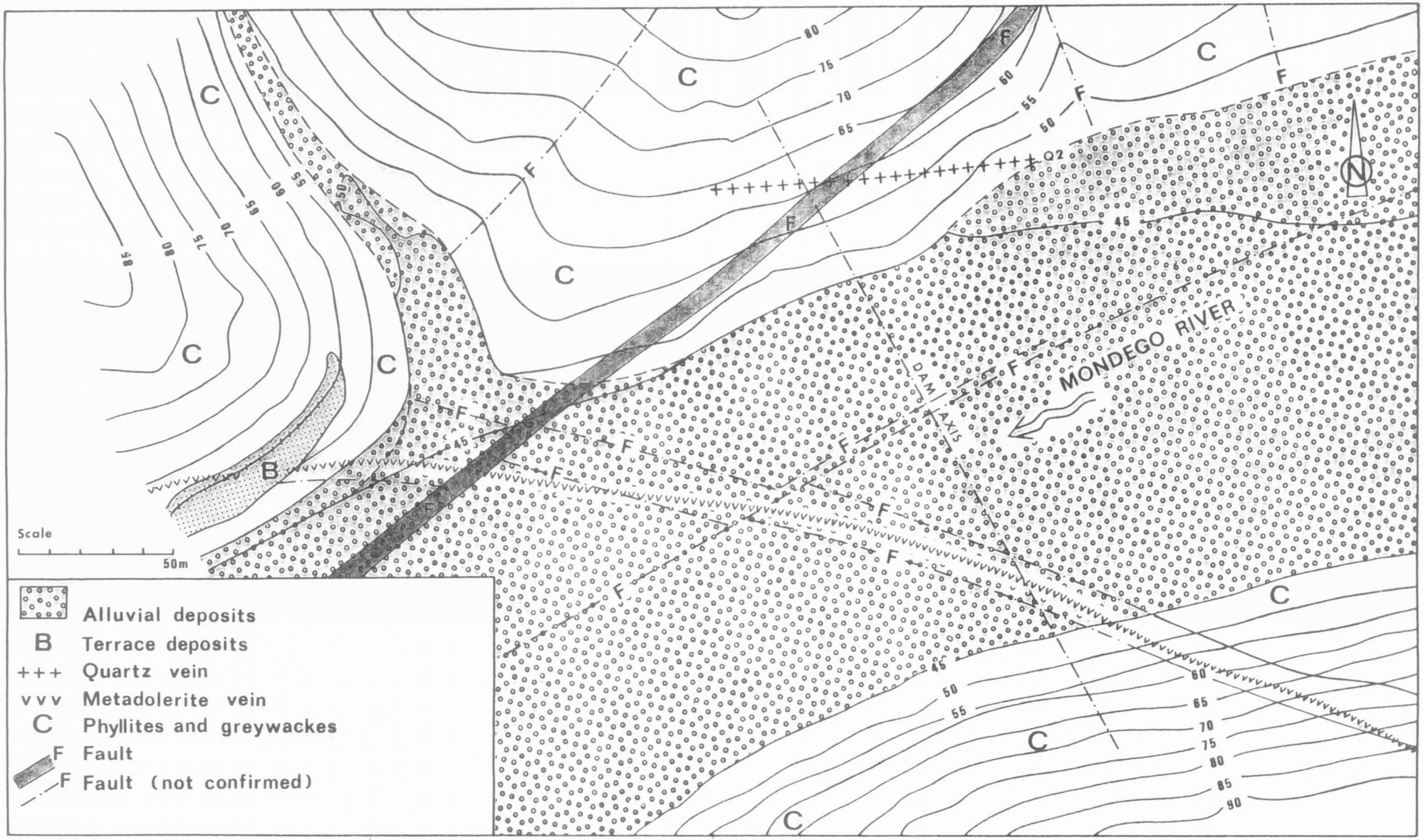


FIG. APPENDIX A14-RAIVA DAM SITE

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (1972)

1. Reconnaissance (early 1972)

PHASE 2

1. Percussion drills (early 1972)

62 short percussion drills with depths varying between 0.5 m and 3.8 m were performed to assess the depth of the alluvial deposits.

Cost (assessed) - 50×10^3 ESC (1972)

PHASE 3

1. Trenches (late 1973 - early 1973

	DESIGNATION	LENGTH (m)
RIGHT BANK	TD	75
LEFT BANK	TE	35

Cost - 20×10^3 ESC (1973)

2. Special study of jointing (mid 1973)

Cost - 20×10^3 ESC (1973)

3. Geophysical exploration (mid 1973)

3.1 Seismic refraction profiles

3 seismic refraction profiles each 65 m long. A 12 geophone array was used, with direct and reverse shots.

3.2 Apparent resistivity profiles

11 apparent resistivity profiles which led to the production of an apparent resistivity map.

$\overline{AB}$ = 45 m $\overline{MN}$ = 3 m

Total length of profiles - 495 m

Cost of geophysical exploration - 80 x 10^3 ESC (1973)

PHASE 4 (1974)

1. Boreholes (Feb - Aug 1974)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ.(o)	NO OF LUGEON TESTS
RIGHT BANK	S.8	30.00	45	5
	S.9	22.48	45	-
	S.10	45.00	90	6
	S.11	30.00	65	4
	S.19	22.50	30	3
BOTTOM OF THE VALLEY	S.5	30.00	45	4
	S.6	30.00	45	5
	S.7	30.00	90	5
	S.15	30.00	45	5
	S.16	30.00	90	5
LEFT BANK	S.1	33.78	45	6
	S.2	32.12	45	6
	S.3	32.70	45	6
	S.4	32.88	45	6
	S.12	26.70	90	4
	S.13	25.80	90	4
	S.14	31.30	45	6
	S.17	22.00	45	3
	S.18	25.00	45	3

Total no of boreholes	19
Total length	562.26 m
Average length	29.6 m
Total no of Lugeon tests	86
Cost	370 x 10^3 ESC (1974)

DAM NAME — VALEIRA

— GENERAL DATA —

LOCATION	—	DOURO RIVER; PORTUGAL
TYPE	—	CONCRETE GRAVITY DAM
TOTAL CREST LENGTH	—	≃ 300 m
MAX. HEIGHT ABOVE FOUNDATIONS	—	48 m
MAX. HEIGHT ABOVE THE RIVER BED	—	42 m
WIDTH OF THE VALLEY (bottom)	—	≃ 200 m
RESERVOIR CAPACITY (total)	—	97×10^6 m^3
FLOODED AREA	—	795 h a
DAM VOLUME	—	$765 \times 10^3 m^3$ *
DATE OF THE PROJECT DESIGN	—	NOV 1969
DESIGN PERIOD	—	N.A.
OWNER	—	ELECTRICIDADE DE PORTUGAL
DESIGNED BY	—	ELECTRICIDADE DE PORTUGAL (HIDOURO)
CONSTRUCTION COST (project estimate)	—	500×10^6 ESC
COST OF THE DESIGN	—	N.A.
CONSTRUCTION PERIOD (project estimate)	—	3.5 YEARS
STATUS	—	CONSTRUCTED

* Volume of concrete for the complete scheme

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigation for Valeira dam were performed in three phases: phase 1 1956, phase 2 1964, phase 3 1967 - 1968.

1956 - Basic geological survey by field mapping. A first geotechnical assessment report was issued in February 1957.

1964 - A second phase of site investigations which consisted of some boreholes.

1967 - 1968 - Last phase of site investigations comprising boreholes, trenches and galleries. A final geotechnical assessment report was presented in October 1969, which also forms part of the main design report issued in November 1969.

Although the site investigations towards the project design can be considered as having begun in 1956, two boreholes had been sunk at the site in 1948 when a general layout of the hydroelectrical schemes to use the potential of the Douro river took place. The results from these boreholes were taken into account when the first geotechnical assessment report was prepared and so they will be quoted later when dealing with the details of the site investigations and their costs.

The actual site is the one indicated by the 1948 study and was selected among other possible sites, but not for geological reasons. However, the geological conditions were taken into account when placing each of the components of the scheme.

Some of the boreholes at the bottom of the valley were sunk from rafts which slightly increased their costs.

Neither "in - situ" nor laboratory mechanical characterization tests were performed. For the calculations dealing with the stability of the dam, concrete deformability modulus / rock mass deformability modulus relationships varying between 1 and 5 were used. The shear parameters a

dopted for a special stability assessment of the left bank were c = 0.045 MPa and $\phi = 40^{\circ}$. These values were justified by quoting bibliographic references and results from other sites.

The sliding coefficient along horizontal surfaces assessed in the design report (relation between the horizontal stresses which can create the sliding and the vertical stresses installed, $\left(\frac{\sum \text{hor. stress}}{\sum \text{ver. stress}}\right)$ varied between 0.58 and 0.46, and the safety coefficient against turnover varied from 2.35 to 1.94.

The dam will serve a double purpose - energy production and regularization of the river in order to make the navigation upstream of the site possible. The spillway is of a crest type. There is a buried power house in the right bank which will drive 3 x 67 Mw and a flood gate in the left bank.

FOUNDATION CONDITIONS

1. Rock Type

Pre - Ordovician phyllites and greywackes are the main rock types that occur in the region, but at the dam site the phyllites are dominant. Alluvial deposits of sandy type are generally thin, from a few centimeters to a maximum of about 3.5 m. A debris fan of a tributary to the Douro river is present at the site (see Fig. Appendix A15).

2. Structure

The phyllites show pronounced schistosity striking about NW - SE and dipping at angles of between 30^{o} and 70^{o} SW. They are folded, which reflects not only a regional characteristic but also a deformation due to a neighbouring granitic intrusion. This accounts for some differences in the bedding attitude between the two banks, the predominant one being N (50^{o} - 60^{o}) W; (54^{o} - 65^{o}) SW in the left and N (32^{o} - 45^{o}) W; (40^{o} - 60^{o}) SW in the right bank respectively.

Some aplite - pegmatite and, more frequently quartz veins, having a maximum thickness of 0.5 m, cross the rock mass.

The major joint sets assessed are presented in Fig. Appendix A15, which also contains the major faults assessed at the site. The faults were grouped in three main sets. Of these fault sets, the more frequent one has an attitude N (18^{o} - 60^{o}) E; 70^{o} SE to Vert, the attitude of the others being N (44^{o} - 50^{o}) W; (48^{o} - 78^{o}) SW and N 8^{o} E to N 2^{o} W; (70^{o} - 80^{o}) E. The maximum fault spaces observed were about 0.5 m and most of them are filled in either by mylonite or quartz veins.

3. State of weathering

The thickness of the top soil is no greater than 1 m and the rock mass shows good conditions at shallow depths in relation to the state of weathering. Borehole recoveries were always very good. The geotechnical assessment report anticipated the following excavation depths: 4 - 5.5 m at the left bank, 4 - 6 m at the bottom of the valley and 6 - 6.5 m at the right bank.

4. Permeability

The characteristics of some adverse joint sets and the attitude of the schistosity facilitate percolation through the rock mass.

Lugeon tests were performed together with the boreholes, along borehole sections of 5 m, using pressure steps of either 0.25 ; 0.5; 1 ; 0.5; 0.25 MPa or 0.5; 1 ; 0.5; MPa. The depths beneath which absorptions were less than 1 Lugeon were assessed. From these assessments, it was suggested in the geotechnical report that grouting should extend to depths greater than 30 to 34 m.

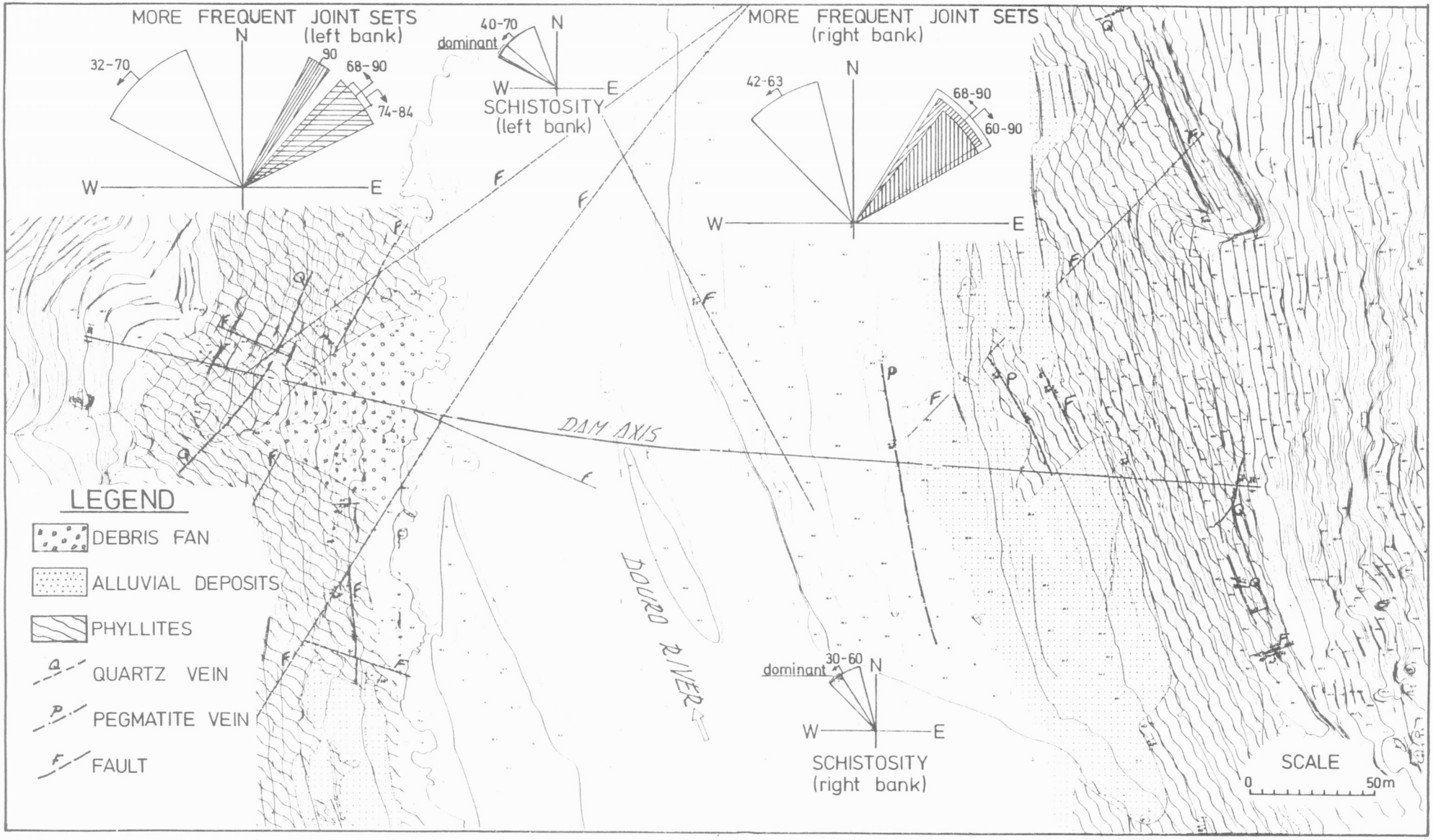


FIG. APPENDIX A15 - VALEIRA DAM SITE

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

Works performed in 1948

Boreholes

DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ.(o)	NO OF LUGEON TESTS
BOTTOM OF THE VALLEY	11.26	90	2
	11.72	90	2

Cost (assessed) 50×10^3 ESC (1969)

PHASE 1 (1956)

1. Reconnaissance

PHASE 2 (1964)

1. Boreholes (Apr - Jun 1964)

DESIGNATION		LENGTH (m)	INCLINATION TO THE HORIZ.(o)	NO OF LUGEON TESTS
BOTTOM OF THE VALLEY	S. 1	62.36	37	11
	S. 2	45.70	90	8
	S. 3	36.20	90	6
	S. 4	60.50	48	12

Total no of boreholes 4
Total length 204. 76 m
Average length 51.19 m
Total no of Lugeon tests 37
Cost 220×10^3 ESC (1964)
(A raft was used to perform boreholes S.2 and S.3.)

PHASE 3 (1967 - 1968)

1. Boreholes (Aug - Nov 1967)

DESIGNATION		LENGTH (m)	INCLINATION TO THE HORIZ.(o)	NO OF LUGEON TESTS
RIGHT BANK	S 11	43.0	90	8
	S 12	65.0	90	12
	S 13	53.0	46	10
	S 14	50.0	52	9
	S 15	50.0	49	9
LEFT BANK	S 7	52.0	50	9
	S 8	50.0	67	9
	S 9	40.5	90	6
	S 10	37.0	90	7

Total no of boreholes 9
Total length 440.5 m
Average length 48.9 m
Total no of Lugeon tests 79
Cost 260×10^3 ESC (1967)

2. Galleries (1967 – ?)

DESIGNATION		LENGTH (m)
RIGHT BANK	GD 80	58
LEFT BANK	GE 80	48
	GE 82.2	22
	GE 81.1	21

Total no of galleries 4
Total length 149 m
Average length 37.3 m
Cost (assessed) 380×10^3 ESC (1968)

3. More boreholes (Jul - Nov 1968)

DESIGNATION		LENGTH (m)	INCLINATION TO THE HORIZ.(°)	NO OF LUGEON TESTS
LEFT BANK	S. 22	45.10	51	9
	S. 24	50.10	51	10
BOTTOM OF THE VALLEY	S. 17	50.00	52	10
	S. 18	30.00	90	6
	S. 19	35.50	90	7
	S. 20	30.20	90	5
	S. 21	30.35	90	6
	S. 23	60.16	41	12
	S. 25	50.40	53	10
	S. 26	50.09	50	10
	S. 27	30.30	90	6
	S. 28	35.20	90	7
RIGHT BANK	S. 16	50.20	90	10
	S. 29	60.04	46	12
	S. 30	55.14	46	11
	S. 31	50.10	45	10
	S. 32	60.28	45	12
	S. 33	60.06	45	12

Total no of boreholes 18

Total length 833.2 m

Average length 46.3 m

Total no of Lugeon tests 165

Cost 410×10^3 ESC (1968)

4. Trenches (Nov 1968)

DESIGNATION		LENGTH (m)
LEFT BANK	TE	130
RIGHT BANK	TD	120

Cost 50×10^3 ESC (1968)

DAM NAME — VALHELHAS

— GENERAL DATA —

LOCATION	— ZEZERE RIVER; MANTEIGAS; GUARDA; PORTUGAL	
TYPE	— CONCRETE GRAVITY DAM	
TOTAL CREST LENGTH	—	525 m
MAX. HEIGHT ABOVE FOUNDATIONS	—	60 m
MAX. HEIGHT ABOVE THE RIVER BED	—	50 m
WIDTH OF THE VALLEY (bottom)	—	145 m
RESERVOIR CAPACITY (total)	—	21.4×10^6 m^3
FLOODED AREA	—	103.5 ha
DAM VOLUME	—	350×10^3 m^3
DATE OF THE PROJECT DESIGN	—	JUL 1973
DESIGN PERIOD	—	1972 - 1973
OWNER	— ELECTRICIDADE DE PROTUGAL	
DESIGNED BY	— ELECTRICIDADE DE PORTUGAL	
CONSTRUCTION COST (project estimate)	—	280×10^6 ESC
COST OF THE DESIGN	—	4.6×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	—	N.A
STATUS	— TO BE CONSTRUCTED	

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigations for Valhelhas dam were performed in 2 phases: phase 1- 1972, phase 2- 1973.

1972 - A basic geological survey by field mapping was carried out late in 1972.

1973 - A comprehensive phase of site investigation was performed, consisting of trenches , geophysical exploration, boreholes and laboratory tests. A geotechnical assessment report was produced in June 1973, also forming part of the main design report issued in July the same year.

The site was selected from six others, based upon geological and other reasons. Two of the sites were rejected exclusively for geological reasons during a first reconnaissance carried out for all of them.

For mechanical characterization, only some laboratory tests were performed.

Stability calculations were made considering a relationship between the deformability modulus of concrete and the deformability modulus of the rock mass which varied between 4 and 1. After the completion of the laboratory tests, a deformability modulus of 10 000 MPa and a Poisson ratio of 0.2 were adopted for the rock mass: the preliminary stability calculations were thus accepted as being representative of actual conditions.

The effect of subhorizontal joints on the dam stability against sliding was studied taking into account the results of the laboratory joint shear tests. The assessed sliding coefficient (relationship between the horizontal stresses which can create the sliding and the vertical stresses to be installed - $\frac{\Sigma \text{ hor. stress}}{\Sigma \text{ ver. stress}}$) adopted in the design was 0.6 ; this value was already affected by a 1.5 safety coefficient.

The dam which is for water supply, has a crest spillway and a bottom

discharge (a 1.5 m diameter vent) under the dam body. A water intake completes the scheme.

FOUNDATION CONDITIONS

1. Rock Type

The rock mass at the site consists of graphite phyllites with interbedded greywackes which belong to the Portuguese ante - Ordovician Complex. The rock mass is overlain by slope deposits, mainly on the right bank; their maximum thickness along the dam axis is about 10 m (see Fig. Appendix A16).

Alluvial deposits consisting of sands, gravels and boulders fill the bottom of the the valley, their average and maximum depths being 7 and 11 m respectively.

2. Structure

A 2.5 - 6 m thick dolerite dyke crosses the foundation zone with an attitude similar to that of the schistosity, which is N (35^{o} - -50^{o}) W; (72^{o} - 78^{o}) SW. Some thin quartz veins, with almost the same attitude as the schistosity also occur.

The rock mass is intensively folded, showing isoclinal structures with a NW - SE strike and 75^{o} SW dip.

The main joint sets assessed are presented in Fig. Appendix A16 , the two most important ones being those which are near parallel and perpendicular to the schistosity.

Three fault sets were reported, as follows:

DIRECTION (o)	INCLINATION (o)
N (35 -50) E	75 SE TO VERT.
N (40 -75) W	VERT.
N (10 -28) W	65 ENE TO VERT.

The faults are generally not wider than 0.10 m and are filled with mylonitic material in a clay matrix.

3. State of weathering

The rock mass is highly or moderately weathered down to depths of 7 - 11 m at the left bank and to 2.5 - 11 m at the right bank. The average excavation depths considered in the design, based on the geotechnical assessment report were: 11 m at the bottom of the valley; 14 m at the right bank and 11 m at the left bank.

4. Permeability

The geotechnical report presents limited data on the rock mass permeability. About 140 Lugeon tests were carried out together with the boreholes. A grout curtain was predicted in the design report to penetrate at least 20 m into the lower impervious rock mass where absortions are less than 1 Lugeon.

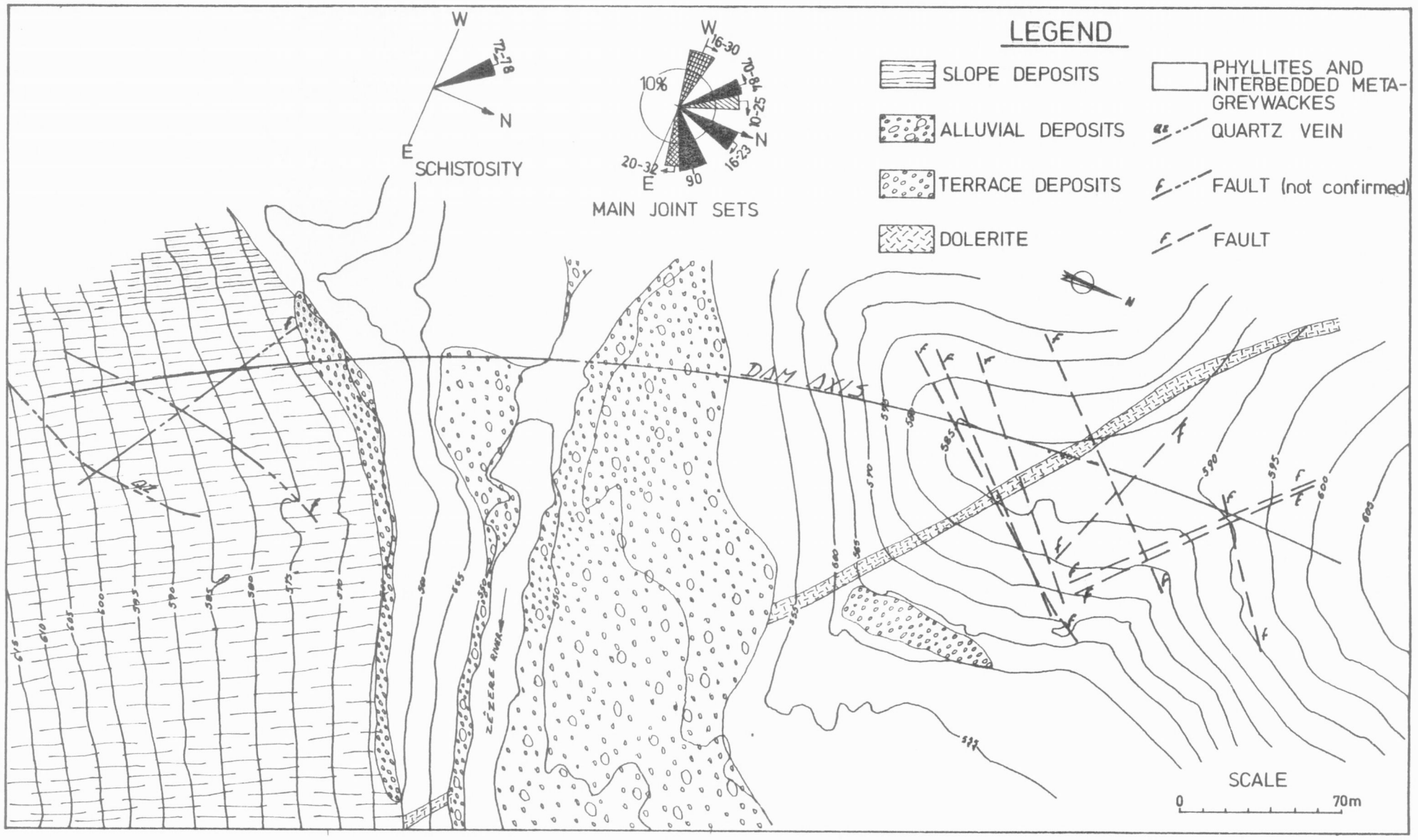


FIG. APPENDIX A16-VALHELHAS DAM SITE

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (1972)

1. Reconnaissance (late 1972)

PHASE 2 (1973)

1. Trenches (Jan - May 1973)

DESIGNATION		LENGTH (m)
LEFT BANK	VME 1	75
	VME 2	90
	VME 3	65
	VME 4	150
	VME 5	130

Total no of trenches	5
Total length	510 m
Average length	102 m
Cost	200×10^3 ESC (1973)

2. Special study of jointing (Apr 1973)

Cost 20×10^3 ESC (1973)

3. Geophysical exploration (Apr 1973)

3.1 Seismic refraction profiles

25 seismic refraction profiles with lengths varying from 54 m to 102 m. A 12 geophone array was used with direct and reverse shots. Total length of profiles - 1 900 m

3.2 Apparent resistivity profiles

12 apparent resistivity profiles with lengths varying between 63 m and 65 m. Results were used to produce an apparent resistivity map comprising both banks.

$\overline{AB}$ = 63 m and $\overline{MN}$ = 5 m (right bank)

$\overline{AB}$ = 65 m and $\overline{MN}$ = 3 m (left bank)

3.3 Geoelectrical soundings

A total of 10 geoelectrical soundings were performed using a $\overline{MN}$ = 3 m and max $\overline{AB}$ = 65 m.

Cost of Geophysical exploration 65 x 10^3 ESC)1973)

4. Boreholes (Jan - May 1973)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{(o)}$	NO OF LUGEON TESTS
RIGHT BANK	SD 1	40.44	60	6
	SD 3	21.10	30	2
	SD 5	26.09	60	-
	SD 6	38.67	30	4
	SD 7	40.37	30	3
	SD 8	39.20	45	5
	SD 9	34.00	45	5
	SD 10	26.48	45	3
	SD 11	35.13	45	5
	SD 13	42.70	45	6
	SD 14	40.07	45	6
	SD 15	40.00	45	4
	SD 16	9.60	90	-
	S1 D1	24.50	60	-
BOTTOM OF THE VALLEY	SE 1	10.70	90	1
	SE 2	12.60	90	1
	SE 3	21.12	90	3
	SE 4	12.90	90	1
	SE 10	12.75	90	1
	SD 2	60.00	45	10
	SD 12	60.00	45	10
	SE 18	36.65	45	5
	SE 19	65.30	45	11
	SE 20	11.28	45	-
	SE 20A	25.00	45	3
	SE 21	10.30	45	-
	SE 21A	5.10	45	-
	SE 26	60.10	45	10
LEFT BANK	SE 13	47.12	45	7
	SE 14	40.50	30	6
	SE 16	27.50	45	3
	SE 17	44.05	30	6
	SE 23	38.94	30	5
	SE 24	35.95	45	5
	SE 25	35.50	45	5
	SE 27	11.00	90	-
	SE 28	9.40	90	-
	S1 E1	25.20	60	-
	S1 E2	25.50	60	2

Total no of boreholes	39
Total length	1202.81 m
Average length	30.84 m
Total no of Lugeon tests	144
Cost	700 x 10^3 ESC (1973)

5. Integral sampling (Apr 1973)

Integral sampling was performed on boreholes S 1O1, S 1E1 , S 1E2 , 15 m in each.

Total length	45 m
Cost	110 x 10^3 ESC (1973)

6. Laboratory tests (Apr - May 1973)

6.1 Uniaxial tests

Total no of tests	18
Type of samples: cut from rock cores.	
Diameter	54 - 60 mm
Aspect ratio	2.5

ASSESSMENT OF E

No of loading cycles	3
Max. pressure applied	50 or 10 MPa

Results:

Results obtained varied from 6 400 MPa to 76 600 MPa, with the majority around 30 000 MPa.

6.2 Joint shear tests

Total no of samples	18
Type of samples prepared from rock core pieces.	
Area of shear	15 - 59 cm^2
No of tests per sample	5

Results:

c (MPa)			Ø (°)		
MAX	MIN	AVER	MAX	MIN	AVER
0.36	0.09	0.21	39.5	26.5	34.2

Cost of laboratory tests 35×10^3 ESC (1973)

DAM NAME — AZIBO

— GENERAL DATA —

LOCATION	—	AZIBO STREAM; MACEDO DE CAVALEIROS; PORTUGAL
TYPE	—	EARTH DAM
TOTAL CREST LENGTH	—	551 m
MAX. HEIGHT ABOVE FOUNDATIONS	—	56 m
MAX. HEIGHT ABOVE THE RIVER BED	—	52 m
WIDTH OF THE VALLEY (bottom)	—	20 m
RESERVOIR CAPACITY (total)	—	54.47×10^6 m^3
FLOODED AREA	—	410 h a
DAM VOLUME	—	$1561.2 \times 10^3 m^3$
DATE OF THE PROJECT DESIGN	—	DEC 1976
DESIGN PERIOD (includes a prel. study)	—	1971 - 1976
OWNER	—	DIRECÇÃO GERAL DOS RECURSOS E APROVEITAMENTOS HIDRAÚLICOS - LISBOA
DESIGNED BY	—	COBA, SARL - LISBOA
CONSTRUCTION COST (project estimate)	—	230×10^6 ESC
COST OF THE DESIGN	—	3.2×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	—	28 MONTHS
STATUS	—	UNDER CONSTRUCTION

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigations were undertaken in three phases: phase 1 1971, phase 2 1971 - 1972, phase 3 1976. The site had been selected before phase 1 commenced. The design was completed in December 1976. The following history of investigations had been constructed from the data available.

1971	- The site investigations began by means of a first field mapping.
1971 - 1972	- A second phase of site investigations was performed, comprising trenches and boreholes. A geotechnical assessment report was produced with the main report in December 1972.
1976	- Third phase of site investigations which consisted of boreholes and trenches was completed in September. A final geotechnical assessment report was produced with the main report in December 1976.
Post 1976	- In the December 1976 report there were two basic designs for the dam. For hydraulic reasons the spillway position was moved from the left to the right bank. As a result, a further (small) phase of investition was undertaken, consisting of 2 trenches.

The first site investigation works were carried out directly by the owner before consultant involvement (April 1972). The first geotechnical assessment (December 1972) was produced by the consultant based on results given by the owner and also on data collected by the consultant during some visits to the site, as the owner's geological report was not yet available.

Neither "in - situ" nor laboratory tests were performed to assess the

mechanical characteristics of both rock material and rock mass. Nevertheless values for the deformability modulus of the rock mass were suggested in the engineering geological report as follows:

a) Anphybolite formation: 10 000 to 15 000 MPa
b) Soft schist formation: 4 000 to 8 000 MPa
c) For the rock mass at depths for the tunnel excavation, in the right bank: 5 000 to 10 000 MPa

In order to study construction materials, 8 boreholes and the usual laboratory tests were carried out which are not considered here.

The undertaking comprises the dam, a spillway which is a channel in the right bank, and a tunnel of 2 m diameter and a length of 330 m in the same bank, which will function firstly as a temporary diversion and later as both bottom discharge and water intake.

FOUNDATION CONDITIONS

1. Rock Type

The main rock types present at the site are shown in Fig. Appendix A17. Slope deposits and soils produced by alteration are not greater than 1 m in depth in general, but at the lower part of the left bank an exceptionally thick deposit of 7.5 m was found. Colluvial - alluvial deposits reach a depth of 5 - 6 m on the valley bottom.

2. Structure

The 150 km length Vilariça fault showing a horizontal displacement of about 6 km, is the most important feature of the tectonic events which affected the region and produced a quite complex structure. In some areas, recent deposits are interrupted not only by that fracture but also by other minor faults with displacements of about 30 cm. Proof of former tectonic activity at the dam site is provided by folding with orientation NE - SW to NNW--SSE, and by several faults affecting the foundation rock mass, particulary in the right bank and at the top of the left one, where a mylonitic zone was found.

The attitude of the formations is N (20^{o} - 60^{o}) E; (35^{o}- 80^{o})S. They are less inclined in the left bank which, together with the difference in lithology can explain the difference in slope between both banks.

The rock mass is crossed in several directions by a number of quartz veins, some centimeters wide. At the base of the right bank some of those veins, 10 cm wide, occur with an orientation different from that exhibited by the schistosity.

The assessment of the major jointsets gave the results shown in Fig. Appendix A17.

3. State of weathering

Beneath the surface deposits, to depths of 10 to 15 m, the rock mass is W 3 - 4, being less weathered on the left bank where the

recovery was almost 100%. On the right bank the recovery was about 20 to 30% to depths that can exceed 20 m. Beneath the W 3 - - 4 zone and under the river bed deposits the rock mass is W 1 - - 2.

The excavation were expected to be not greater than 2 m except in the zones of alluvial and slope deposits, contrasting with the much deeper excavation necessary for a concrete dam alternative.

4. Permeability

Based upon the Lugeon test results, most of which were carried out in section of 5 m using pressures of mainly 0.25; 0.5; 1; 0.5; 0.25 MPa, the rock mass was divided into two zones.

a) Upper part (down to 20 - 25 m depth), with K between 2.5×10^{-5} and 5×10^{-4} cm /sec.

b) Lower part (beneath the above mentioned depth), with K between 5×10^{-7} and 2.5×10^{-5} cm /sec..

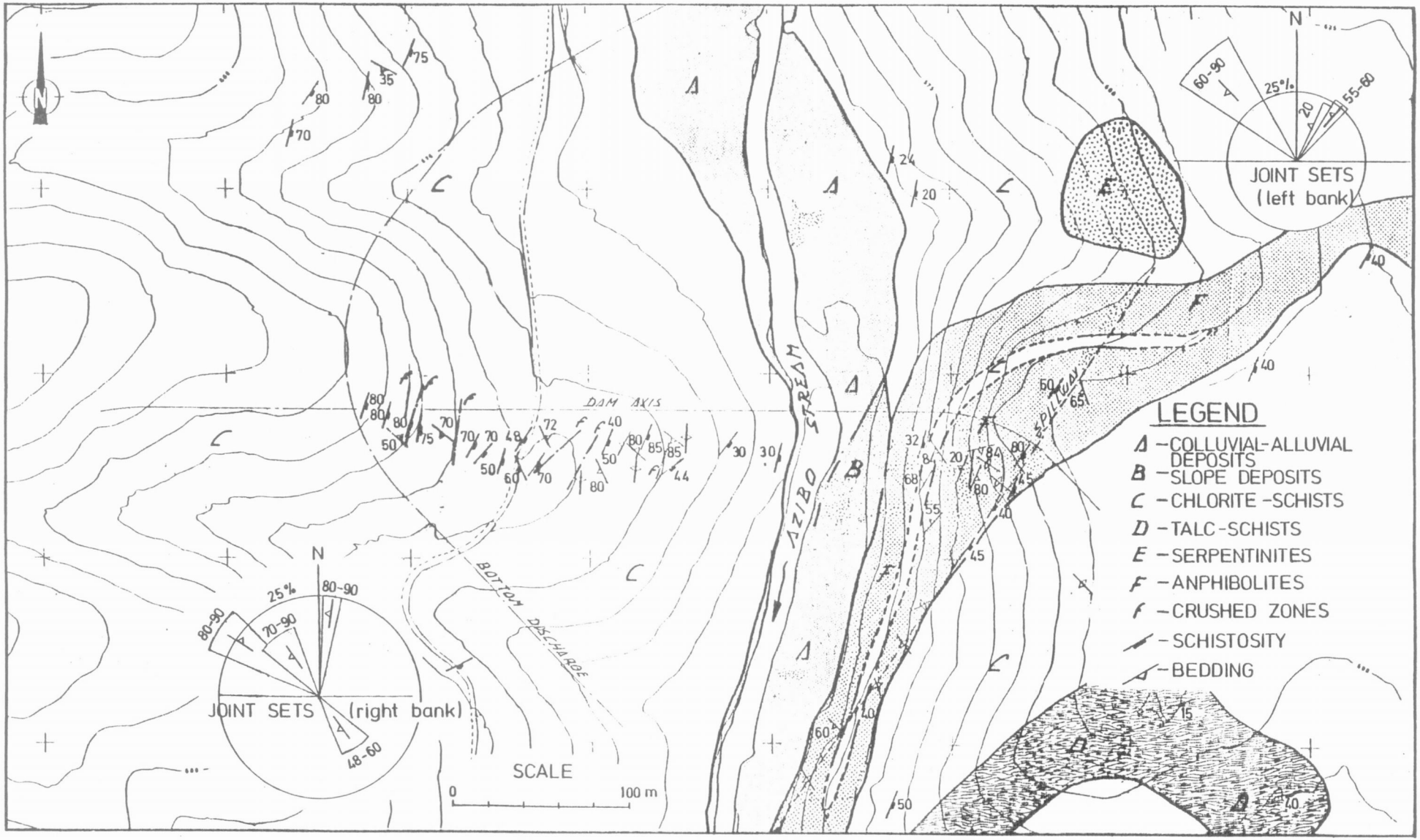


FIG. APPENDIX A17 - AZIBO DAM SITE

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (1971)

1. Reconnaissance (Jun 1971)

PHASE 2 (1971 - 1972)

1. Trench (Jun 1971 - Mar 1972)

One discontinuous trench along the dam axis

Total length - 370 m

Cost - 80×10^3 ESC (1971)

2. Boreholes (Jul 1971 - Jul 1972)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ.(o)	NO OF LUGEON TESTS
RIGHT BANK	15 P12	60.0	45	9
	14 P10	50.0	45	8
	1 P 8	40.0	45	7
	12 P 6	40.0	45	7
	13 P 6	40.0	50	6
	11 P 4	45.0	45	6
	10 P 2	40.0	45	7
BOTTOM OF THE VALLEY	9 P 0	8.0	90	-
	7 P 0	7.0	90	-
	8 P 0	9.0	90	-
	2 P 0	80.0	55	14
	6 P 0	11.5	90	-
	5 P 0	6.0	90	-
LEFT BANK	3 P 1	55.0	30	9
	4 P 3	40.0	30	7
	16 P 5	75.0	90	14

Total no of boreholes 16

Total length 606.5 m

Average length 38 m

Total no of Lugeon tests 94

Cost 430×10^3 ESC (1972)

More boreholes (same period as above)

DESIGNATION		LENGTH (m)	INCLINATION TO THE HORIZ (°)	NO OF LUGEON TESTS
SPILLWAY	17 EC	20	90	3
	18 EC	20	90	1

Cost - included above

PHASE 3 (1976)

1. Boreholes (Mar-Apr 1976)

DESIGNATION		LENGTH (m)	INCLINATION TO THE HORIZ (°)	NO OF LUGEON TESTS
BOTTOM OF THE VALLEY	SDF 2	8.0	90	-
	SDF 3	6.2	90	-
	SDF 5	8.0	90	-

Total no of boreholes	3
Total length	22.2 m
Average length	7.4 m
Cost	(see below)

More boreholes (same period as above)

DESIGNATION		LENGTH (m)	INCLINATION TO THE HORIZ (°)	NO OF LUGEON TESTS
SAFETY STRUCTURES	SDF 1	9.0	90	-
	SDF 4	6.5	90	-
	SDF 6	22.0	90	2
	STA 1	28.0	90	-
	STA 2	28.0	90	-

Total no of boreholes	5
Total length	93.5 m
Cost (all boreholes of phase 3)	320 x 10^3 ESC (1976)

2. Trench (Aug-Sep 1976)

One trench along the spillway axis

Total length - 430 m

Cost 100 x 10^3 ESC (1976)

PHASE 4 (1978 - after the project design)

1. Trenches

DESIGNATION	LENGTH (m)
V1 J	110
V2 J	110

Cost 30 x 10^3 ESC (1978)

DAM NAME — CAPINHA

— GENERAL DATA —

LOCATION	— POLDRAS STREAM; FUNDÃO; PORTUGAL	
TYPE	— EARTH DAM	
TOTAL CREST LENGTH	—	231.2 m
MAX. HEIGHT ABOVE FOUNDATIONS	—	18 m
MAX. HEIGHT ABOVE THE RIVER BED	—	16 m
WIDTH OF THE VALLEY (bottom)	—	< 60 m
RESERVOIR CAPACITY (total)	—	$0.522 \times 10^6 m^3$
FLOODED AREA	—	9.7 h a
DAM VOLUME	—	$82 \times 10^3 m^3$
DATE OF THE PROJECT DESIGN	—	NOV 1977
DESIGN PERIOD	—	1976 - 1977
OWNER	— DIRECÇÃO GERAL DOS RECURSOS E APROVEITAMENTOS HIDRAÚLICOS - LISBOA	
DESIGNED BY	— COBA, SARL - LISBOA	
CONSTRUCTION COST (project estimate)	—	27×10^6 ESC
COST OF THE DESIGN	—	0.65×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	—	6 MONTHS
STATUS	— CONSTRUCTED	

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigations for this small dam were undertaken in three phases: phase 1 1973, phase 2 1974, phase 3 1977.

1973 - A first geological survey by field mapping, carried out at the end of the year.

1974 - A second phase of site investigations by means of one trench and one borehole. A preliminary geotechnical assessment report was issued with a design for a gravity-buttress dam in May 1975.

1977 - The final phase of site investigations which comprised one trench and some boreholes.

The project design was ordered in August 1973 and in May 1975 a design for a concrete gravity-buttress dam was produced. At that time it was thought that the site was equally suitable either for an earth dam or for a concrete gravity dam and the last type was selected. Of the exploration works programmed for that phase (1 trench + 4 boreholes) only the trench was excavated and one borehole was near completion, at the time when the 1975 geotechnical assessment report was prepared. So, that report had a preliminary character due to the lack of the data available. Later, when the exploration works were resumed, it was observed that the rock mass conditions were much worse than the ones assessed in 1975. Unfortunately, the results of the single borehole sunk in 1974 were better than those obtained from later holes. Due to the new set of data available, it was decided to change the dam type and a new design for an earth dam was ordered. This design was released in November 1977.

A buried 120 m long conduit in the left bank, with a cross sectional area of 9 m^2 was designed to serve either as spillway or bottom discharge. A water intake in the right bank completes the scheme. The dam is for water supply purposes.

This project can be seen as an example of how unreliable a geotechni-

cal assessment can be, if based upon insufficient information, even for a small dam like this one. In this particular case, the performance of limited but adequate exploration works in time for the first report, would have avoided the costs of the abandoned first design.

FOUNDATION CONDITIONS

1. Rock Type

The rock type at the site is a pseudoporphyritic granite, belonging to the so - called "Ganito das Beiras Complex". A colluvial--alluvial deposit with a thickness of between 1 and 4 m fill in the bottom of the valley across a width corresponding to the flood plain (see Fig. Appendix A18).

2. Structure

Two main joint sets were assessed, having directions of N (45^{o} - -60^{o}) W and N (20^{o} - 40^{o}) E, with a subvertical inclination. A minor subhorizontal joint set was also assessed, with a direction of around N 40^{o} W. The joint spacing is almost greater than 1 m. No major defects were assessed.

3. State of weathering

The rock mass is completely weathered down to depths of several meters. Zero recovery was obtained in consecutive sections of some boreholes. Nevertheless, less weathered granite crops out in some areas across the valley. The expected depth of the excavation was, according to the geotechnical report, about 0.5 m after the removal of the colluvial - alluvial deposits.

4. Permeability

A few Lugeon tests were performed together with some of the boreholes. Their results showed that down to depths of 15 - 20 m the rock mass exhibits an equivalent permeability coefficient of 2×10^{-4} cm/sec., under a water pressure of 0.5 MPa. The decision on the eventual need for a grout curtain was postponed until the construction phase.

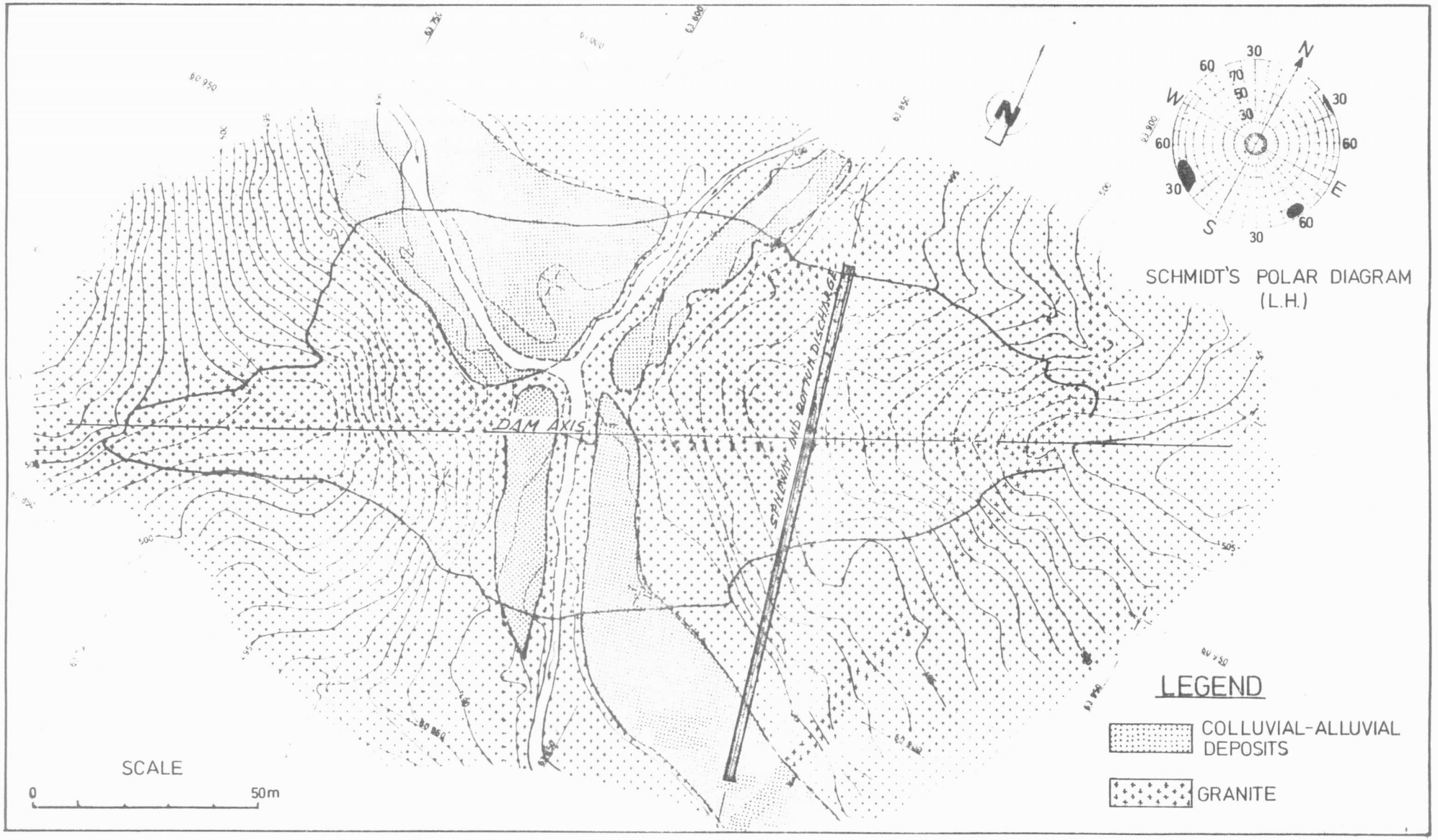


FIG. APPENDIX A18 – CAPINHA DAM SITE

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (1973)

1. Reconnaissance (end 1973)

PHASE 2 (1974)

1. Trench (Jun 1974)

One trench along the dam axis, with a length of 75 m

Cost (assessed) 15×10^3 ESC (1974)

2. Boreholes (Jul 1974)

DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ (o)	NO OF LUGEON TESTS
RIGHT BANK 1 SC2	45	45	4

Cost (assessed) 75×10^3 ESC (1974)

PHASE 3 (1977)

1. Boreholes (Jun - Aug 1977)

DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ (o)	NO OF LUGEON TESTS
RIGHT BANK 3 SC 4	35.0	40	5
RIGHT BANK 4 SC 6	23.8	70	1
RIGHT BANK 5 SC 1	15.4	70	-
LEFT BANK 6 SC 3	35.5	45	6
LEFT BANK 7 SC 3	15.2	60	-

Total no of boreholes	5
Total length	124.9 m
Average length	24 .98 m
Total no of Lugeon tests	12
Cost	250×10^3 ESC (1977)

2. Trench (Oct 1977)

One trench along the spillway, with a length of 60 m.

Cost 20×10^3 ESC (1977)

3. More boreholes (Oct 1977)

DESIGNATION		LENGTH (m)	INCLINATION TO THE HORIZ.(o)	NO OF LUGEON TESTS
SPILLWAY	SV 3	11.95	VERT	-
	SV 4	13.00	VERT	-

Total length of boreholes 24.95 m

Cost (assessed) 100×10^3 ESC (1977).

DAM NAME — MEIMOA

— GENERAL DATA —

LOCATION	—	MEIMOA STREAM; PENAMACOR; CASTELO BRANCO; PORTUGAL
TYPE	—	EARTH DAM
TOTAL CREST LENGTH	—	796 m
MAX. HEIGHT ABOVE FOUNDATIONS	—	65 m
MAX. HEIGHT ABOVE THE RIVER BED	—	61 m
WIDTH OF THE VALLEY (bottom)	—	125 m
RESERVOIR CAPACITY (total)	—	641×10^6 m^3
FLOODED AREA	—	295 ha
DAM VOLUME	—	3697×10^3 m^3
DATE OF THE PROJECT DESIGN	—	MAY 1974
DESIGN PERIOD	—	1973 - 1974
OWNER	—	DIRECÇÃO GERAL DOS RECURSOS E APROVEITAMENTOS HIDRAÚLICOS - LISBOA
DESIGNED BY	—	COBA, SARL - LISBOA
CONSTRUCTION COST (project estimate)	—	290×10^6 ESC
COST OF THE DESIGN	—	10×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	—	48 MONTHS
STATUS	—	UNDER CONSTRUCTION

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigations were undertaken in 3 phases: phase 1 1971, phase 2 1971 and phase 3 1973 - 1974.

1971 - A first geological survey by field mapping was performed.

1971 - Second phase of site investigation by means of trenches and boreholes.

1973 - 1974 - Last phase of site investigations which consisted of trenches, auger borings and boreholes.

The first two phases of site investigations were performed for a preliminary study conducted by the owner. The geotechnical assessment report issued together with the main design report, in May 1974, could not take into account a small part of the works performed during the third phase of site investigations because their results were not available yet. Nevertheless a final geotechnical assessment report was released in June 1975, by the owner of the project, to joint the main report.

A geophysical exploration programme, though established for the last phase, was not performed.

The site was selected during the first phase of investigations. The dam type was chosen taking into account both topographic and geological conditions. It was estimated that for a concrete dam an average depth of 10 m but reaching 30 m in some zones would need to be excavated.

Three of the driven boreholes (17 P 4, 18 P 4 and 20 P4 with a total length of 73.5 m) were exclusively to clarify a problem arising from the excavation of one trench which exposed a completely weathered dolerite dyke.

As safety structures, there is a spillway (a 426 m long channel), and a 323.5 m long tunnel with a diameter of 4.3 m which will serve first as a temporary diversion and later as a bottom discharge.

A 555 m long and 4.3 m diameter tunnel for water intake completes this irrigation purpose dam which forms part of a larger scheme (see Sabu - gal dam).

FOUNDATION CONDITIONS

2. Rock Type

The rock type present at the site consists of phyllites and greywackes belonging to the Portuguese ante - Ordovician complex. In the right bank the excavations for the foundation will be into the phyllites while at the central part of the dam and in part of the left bank the greywackes are dominant (see Fig. Appendix A19). A 3.5 m thick alluvial terrace overlies the bottom of the valley corresponding to a 125 m wide flood plain. Slope deposits and residual soils that overlie the rock mass are not thicker than 1 m.

2. Structure

The strata are highly folded, and isoclinal folds are present. Some highly weathered dolerite dykes, as well as some quartz veins which are of little geotechnical importance cross the rock mass.

The strike of the schistosity is nearly NW - SE, being parallel or making a small angle with the dam axis, and its inclination is almost vertical (85^{o} SW - 85^{o} NE).

The main joint sets assessed at each bank are presented in Fig. Appendix A19.

Some crushed zones were detected but no major tectonic fractures were revealed by the investigation.

3. State of weathering

Beneath the residual soils, the rock mass was classified as moderately weathered (W 3) down to depths varying between 3 and 5 m in the lower parts of the banks. This thickness increases towards the top of the valley sides, where boreholes revealed depths of 11 m and 36 m at the left and right bank respectively. Beneath the river bed the moderately weathered rock extends down to depths of 5 - 8 m.

The average depth to be excavated was estimate as 4 m.

4. Permeability

The performance of Lugeon tests showed high loss of water (sometimes total) down to depths varying between 20 and 25 m beneath both the river bed and the lower parts of the banks. That depth increases to a maximum of 35 m below ground level at the highest points of the river banks. A grout curtain was anticipated to reach depths where absorptions are equal to or less than 2 Lugeons.

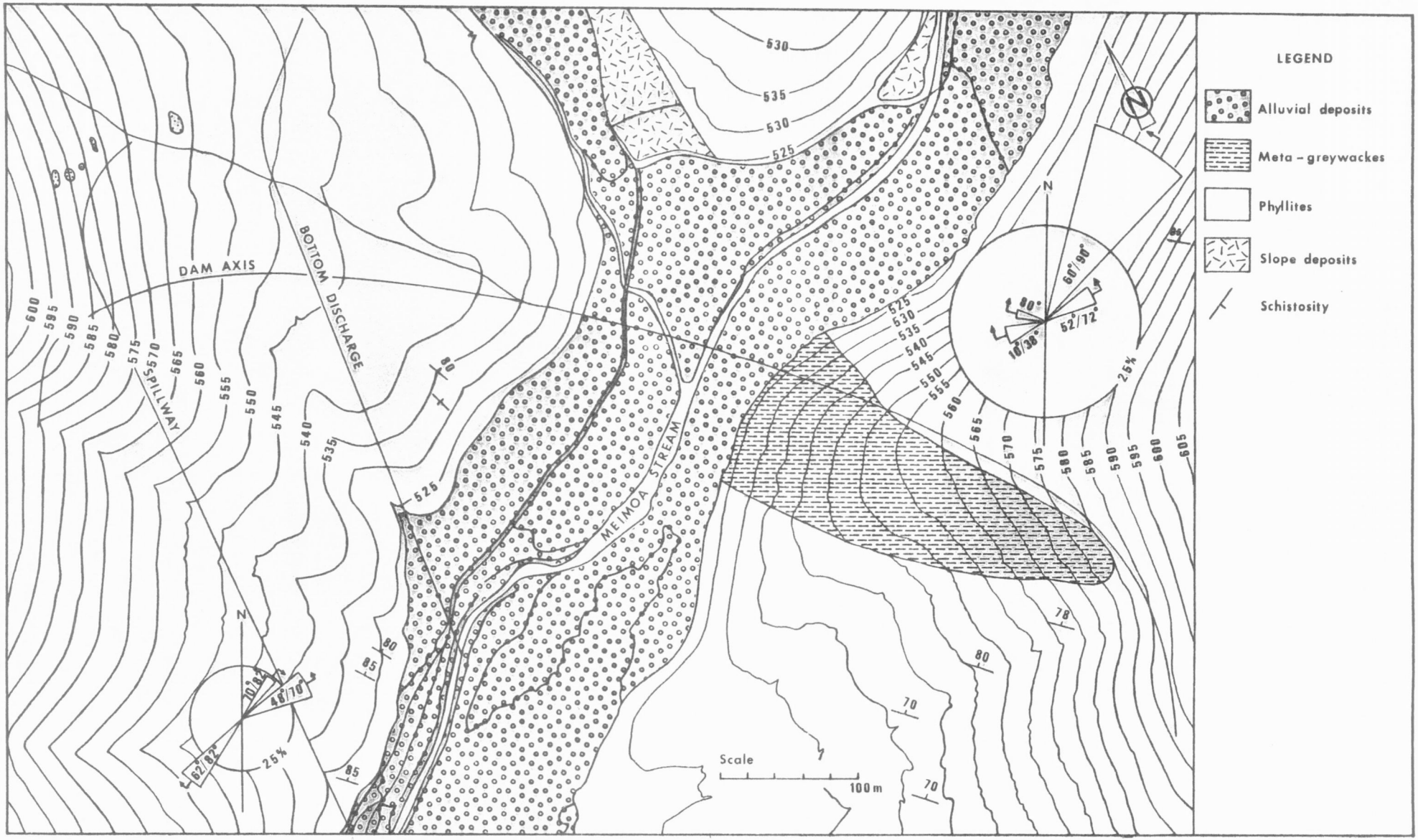


FIG. APPENDIX A19-MEIMOA DAM SITE

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (1971)

1. Reconnaissance (Jan 1971)

PHASE 2 (1971)

1. Trenches (Feb - Dec 1971)

	DESIGNATION	LENGTH (m)
DAM AXIS	V 1E	280
	V 2E	106
OTHERS	V 1J	10
	V 4J	12
	V 4M	26
	V 6	25
	V 8	32

Total no of trenches 7

Total length 491 m

Cost 240×10^3 ESC (1971)

2. Boreholes (Feb - Dec 1971)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{(o)}$	NO OF LUGEON TESTS
RIGHT BANK	1 P8	50.00	45	7
	2 P6	65.00	45	10
	3 P4	45.00	45	8
	4 P2	20.00	45	2
	16 P2	8.00	90	-
	17 P4	42.00	35	-
	18 P4	13.50	90	-
	19 P2	5.50	90	-
	20 P4	18.00	45	-
BOTTOM OF THE VALLEY	5 P0	8.00	90	-
	8 P0	80.00	35	14
	9 P0	13.00	90	-
	10 P0	10.00	90	-
	11 P0	9.00	90	-
	12 P0	3.00	90	-
	13 P0	9.50	90	-
	14 P0	9.50	90	-
	15 P0	8.50	90	-
LEFT BANK	6 P1	25.00	45	4
	7 P3	60.00	45	11

Total no of boreholes	20
Total length	502.5 m
Average length	25.1 m
Total no of Lugeon tests	56
Cost	330 x 10^3 ESC (1971)

PHASE 3 (1973 - 1974)

1. Trenches (Dec 1973 - Oct 1974)

	DESIGNATION	LENGTH (m)
SPILL.	VE C	345
OTHERS	TR 1	48
	TR 2	48
	TR 3	43
	TR 4	62
	VTA1	58
	VTA2	40

Total no of trenches	7
Total length	644 m
Cost	2 260 x 10^3 ESC (1974)

2. Auger borings (May 1974)

	DESIGNATION	LENGTH (m)
SPILL.	TEC 1	7.30
BOTTOM DISCH.	TD 1	8.03
	TD 2	8.13

Total no of auger borings	3
Total length	23.49 m
Cost	(see below)

3. Boreholes (Mar - Jul 1974)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{(o)}$	NO OF LUGEON TESTS
BOTTOM DISCHARGE SPILL.	SEC 1	25.40	60	-
	SD 1	30.10	45	-
	SD 2	44.80	45	-
	SD 3	40.12	45	-
	SD 4	25.00	45	-
OTHERS	STA 1	25.21	45	-
	STA 2	75.22	60	3
	STA 3	100.10	60	3
	STA 3A	35.16	40	-
	STA 4	35.63	60	-
	STA 5	20.21	45	-
	STA 6	30.16	70	-

Total no of boreholes	12
Total length	487.11 m
Total no of Lugeon tests	6
Cost (boreholes + auger borings)	460×10^3 ESC (1974).

DAM NAME — MORGAVEL

— GENERAL DATA —

LOCATION	— MORGAVEL STREAM; SINES; PORTUGAL
TYPE	— EARTH DAM
TOTAL CREST LENGTH	— 2 700 m
MAX. HEIGHT ABOVE FOUNDATIONS	— 46 m
MAX. HEIGHT ABOVE THE RIVER BED	— 40 m
WIDTH OF THE VALLEY (bottom)	— ≃ 30 m
RESERVOIR CAPACITY (total)	— 32.5×10^6 m^3
FLOODED AREA	— 320 ha
DAM VOLUME	— 2600×10^3 m^3
DATE OF THE PROJECT DESIGN	— JUN 1975
DESIGN PERIOD (includes a prel. study)	— 1973 - 1975
OWNER	— GABINETE DA ÁREA DE SINES — LISBOA
DESIGNED BY	— HIDROPROJECTO, SARL - LISBOA
CONSTRUCTION COST (project estimate)	— 200×10^6 ESC
COST OF THE DESIGN (includes the prel. study)	— 8.3×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	— 25 MONTHS
STATUS	— CONSTRUCTED

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigation for the Morgavel dam were carried out in three phases: phase 1 1973, phase 2 1973 and phase 3 1975.

1973 - A first geological survey by field mapping was performed in January.

1973 - A second phase of site investigations was performed between March and July, comprising of boreholes and percussion drills. A preliminary geotechnical assessment report was issued in July 1973.

1975 - A third minor phase was carried out in October by means of geophysical exploration. A final geotechnical assessment report was issued in May 1975

This site was selected from two possible ones based upon the more favourable geological and geotechnical conditions encountered. A geotechnical assessment report comparing both sites was issued in July 1973.

Only the site investigations performed at the selected site will be considered here.

After the site investigations had been completed it was decided to modify the shape of the dam axis. Neverthless it was thought that performing further works was not justifiable.

The dam is for flood regularization and for water supply to the Sines industrial complex. There is a 680 m long spillway which is a channel with a depth varying between 2 m and 3 m, and a bottom discharge at the left bank (a 450 m long tunnel with a diameter of 3 m). This would serve as a provisional diversion. A water intake completes the scheme.

FOUNDATION CONDITIONS

1. Rock Type

The rock types present at the site are phyllites, shales and greywackes which belong to the Alentejo's Carboniferous complex.These formations are overlain by clay - sand Plio - Pleistocene deposits. Same alluvial deposits with minor importance occur at the bottom of the valley.

The foundation of the central part of the dam lies on the Carboniferous formations,while the lateral parts which are less than 20 m in height and constitute more than 50% of the total length of the dam lie on the Plio - Pleistocene materials (see Fig. Appendix A20).

2. Structure

The Carboniferous formations are intersected by several quartz veins, a few centimeters in width. Some of them are associated with crushed zones. Some volcanic rocks related to the Sines gabbro intrusion occur interbedded with the Carboniferous formations. These, are strongly folded, with very tight to isoclinal folds. The general orientation of both bedding and schistosity is around NNE - SSE, almost subvertical.

Aerial photograph interpretation pointed out some possible faults which were not confirmed. Some of the detected alignments may correspond to the predominant directions of the more important joint sets. Some of the crushed zones near the dam axis were also mapped as suspected faults.

The more important joint sets assessed were:

DIRECTION (o)	INCLINATION (o)
N (20-40) E	70-90 SE
N 20 E - N 20 W	30-70 W

3. State of weathering

The shales and greywackes were classified as highly to completely weathered from the surface down to depths of about 3 to 16 m. This depth is less where the Carboniferous rock mass is still overlain by the Plio - Pleistocene deposits. From 3 to 16 m downwards the rock mass is moderately weathered.

The anticipated depths for the excavations were: 1 to 2 metres for the central part of the dam and 0.5 to 1 m (top soil removal) for the lateral parts. As for the bottom of the valley, the removal of all of the alluvial deposits was recommended.

4. Permeability

The water table lies about 5 m beneath the surface in the zones covered by the Plio - Pleistocene deposits. In the zones where the Carboniferous complex crops out, that depth reaches 15 to 20 m.

Based upon the results of the Lugeon tests carried out, it was assessed that the rock mass would behave as pervious down to depths of 15 and 25 m beneath the bottom and the lateral parts of the valley respectively. A grout treatment down to these depths was suggested. The results of the Lefranc tests gave an average permeability coefficient of 10^{-4} cm/sec for the Plio - Pleistocene materials.

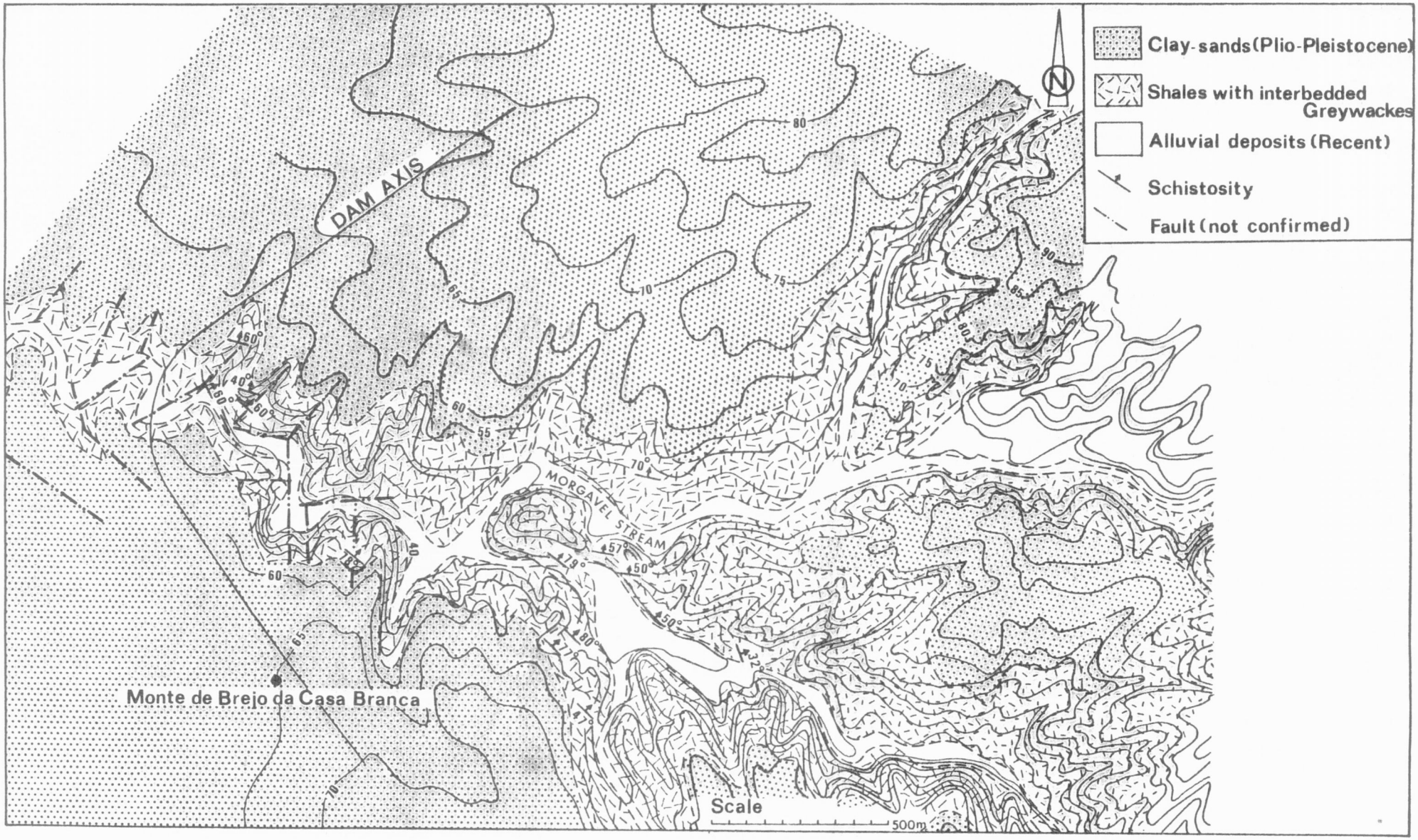


FIG. APPENDIX A20-MORGAVEL DAM SITE

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (1973)

1. Reconnaissance (Jan 1973)

PHASE 2 (1973)

1. Boreholes (Mar - Jul 1973)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{10}$	NO OF LUGEON TESTS
RIGHT BANK	MR 1	25.05	60	4 *
	MR 2	25.05	60	7 *
	MR 3	30.05	60	9
	MR 4	45.08	60	14
	MR 5	35.25	75	12
	MR 6	50.77	45	16
BOTTOM OF THE VALLEY	MR 7	35.20	90	7
LEFT BANK	MR 8	35.00	60	11
	MR 9	30.10	60	9
	MR 10	35.72	60	11
	MR 11	29.44	60	9 *
	MR 12	20.16	60	1 **
	MR 13	20.66	60	1 **

*Also 2 Lefranc tests **Also 4 Lefranc tests

Total no of boreholes	13
Total length	417.47 m
Average length	32.1 m
Total no of Lugeon tests	111
Total no of Lefranc tests	14
Cost	(see below)

2. More boreholes (same period as above)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{(0)}$	NO OF LUGEON TESTS
BOTTOM DISCHARGE	S-1R	15.05	90	-
	S-2R	15.13	90	-
	S-3R	12.21	90	-

Total no of boreholes 3
Total length 42.39 m
Cost (see below)

3. Percussion drills (same period as above)

	DESIGNATION	LENGTH (m)	NO OF SPT'S	NO OF LEFRANK TESTS
RIGHT BANK	MP 1	7.70	3	3
	MP 2	8.05	4	3
	MP 3	13.40	8	4
	MP 4	10.60	6	5
	MP 5	10.50	8	5
LEFT BANK	MP 6	7.85	3	4
	MP 7	15.78	10	5
	MP 8	14.08	8	5
	MP 9	15.60	6	5
	MP 10	16.90	6	5

Total no of percussion drills 10
Total length 120.46 m
Average length 12 m
Total no of SPT'S 62
Cost (see below)

4. More percussion drills (same period as above)

	DESIGNATION	LENGTH (m)	NO OF SPT'S	NO OF LEFRANK TESTS
BOTTOM DISCHARGE	S-4P	7.93	3	-
	S-5P	8.91	4	-
	S-6P	7.00	2	-

Total no of percussion drills 3

Total length 23.84 m

Cost (all boreholes + percussion drills) 500 x 10^3 ESC (1973)

PHASE 3 (1975)

1. Geophysical exploration (Oct 1975)

5 seismic refraction profiles, each 108 m long, along the bottom discharge axis. A 12 geophone array with direct and reverse sh ots was used.

Cost of geophysical exploration 25 x 10^3 ESC (1975)

DAM NAME — ODELOUCA

— GENERAL DATA —

LOCATION	—	ODELOUCA STREAM; ALGARVE; PORTUGAL
TYPE	—	EARTH DAM
TOTAL CREST LENGTH	—	440 m
MAX. HEIGHT ABOVE FOUNDATIONS	—	80 m
MAX. HEIGHT ABOVE THE RIVER BED	—	75.5 m
WIDTH OF THE VALLEY (bottom)	—	90 m
RESERVOIR CAPACITY (total)	—	$239.1 \times 10^6 m^3$
FLOODED AREA	—	1 000 ha
DAM VOLUME	—	$3853.6 \times 10^3 m^3$
DATE OF THE PROJECT DESIGN	—	JAN 1977
DESIGN PERIOD (includes a prel. study)	—	1971 - 1977
OWNER	—	DIRECÇÃO GERAL DOS RECURSOS E APROVEITAMENTOS HIDRAÚLICOS - LISBOA
DESIGNED BY	—	COBA, SARL - LISBOA
CONSTRUCTION COST (project estimate)	—	620×10^6 ESC
COST OF THE DESIGN (includes the prel. study)	—	8.9×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	—	3.5 YEARS
STATUS	—	TO BE CONSTRUCTED

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigations at Odelouca dam site were performed in 4 pha - ses: phase 1 1971, phase 2 1971 - 1972, phase 3 1976 and phase 4 1977.

1971 - The site investigations began by a first geological survey by field mapping.

1971 - 1972 - Second phase of site investigations, comprising trenches, boreholes and a geophysical exploration programme.

1976 - Galleries, shafts and new geophysical exploration were performed. A final geo - technical assessment report was issued together with the main design report in January 1977.

1977 (pos - design) - Last phase of site investigations by means of boreholes and trenches.

After the site had been selected, the two first phases of site investigations were performed by the owner, having an earth dam in mind. Meanwhile, a preliminary report issued in August 1972 suggested that a multiple vault concrete dam could also be considered. The third phase of site investigations was then performed to also investigate this second possibility, by means of appropriate works. This led to the conclusion that an earth dam would be more suitable for the site's geotechnical characteristics.

The last phase of site investigations, dealing mainly with the safety structures, though previously programmed, was not finished in time and so its results could not be considered in the final geotechnical assessment report.

This dam forms part of a water supply scheme, with another dam (Funcho dam) and a tunnel linking both reservoirs.

The spillway is a 234 m long channel. A 422 m long tunnel with a diame

ter of 4 m which will serve as a temporary diversion will be adapted afterwards to function both as a bottom discharge and a water intake. All these structures are in the left bank.

FOUNDATION CONDITIONS

1. Rock Type

The main rock types are shales and greywackes in alternating series (flysch facies) dated as Carboniferous.

Slope deposits, 1 to 2 m thick, overlie the rock mass, reaching 3 to 4 m in some zones.

Coarse alluvial deposits fill the bottom of the valley in an area corresponding to the flood plain, their average and maximum thicknesses being 3 and 4 m respectively.

2. Structure

Several syenite and "monchiquite" veins crossing the rock mass in several directions, were detected in both banks.

The schistosity attitude, being parallel to that of the bedding lies between N 15° E - N 25° W; (20° - 45°) E. The direction of the stream is also similar to that of the schistosity and the inclinations of the banks, being controlled by the structure, are different: 35° - 40° in the left bank and 20° - 25° in the right one.

An assessment of the jointing gave the results presented in Fig. Appendix A21. Joints are smooth or a little rough and fracture intercept is moderate (20 - 60 cm). One well defined set is parallel to both bedding and schistosity. A little variation in the joint set attitude between the two banks was noticed.

No major tectonic fractures were revealed.

3. State of weathering

The rock mass is generally highly to completely weathered (W 4 - 5) in the banks, down to depths of between 2 and 5 m, and slightly to moderately weathered (W 2 - 3) thereafter down to depths of 10 to 15 m. Under the top of the banks the last refered

degree of weathering can reach 19 m in depth.

At the bottom of the valley, under the alluvial deposits, the rock mass is fresh to slightly weathered (W 1 - 2).

The excavation depth for the proposed earth dam is expected to vary between 2 m and 7 m.

4. Permeability

The rock mass was divided in two zones on the basis of the results of the performed Lugeon tests: a) An upper zone (depth down to 20 - 25 m), with 3×10^{-5} cm/sec. $< K <$ 6.8×10^{-4} cm/sec.; b) A lower zone (depth greater than the previous ones) with 4.5×10^{-7} cm/sec. $< K <$ 3×10^{-5} cm/sec..

The geotechnical report recommended a grout curtain down to depths where absorptions are less than 3.5 l/min. m at a test pressure of 1 MPa. This will lead to depths varying between 15 and 30 m.

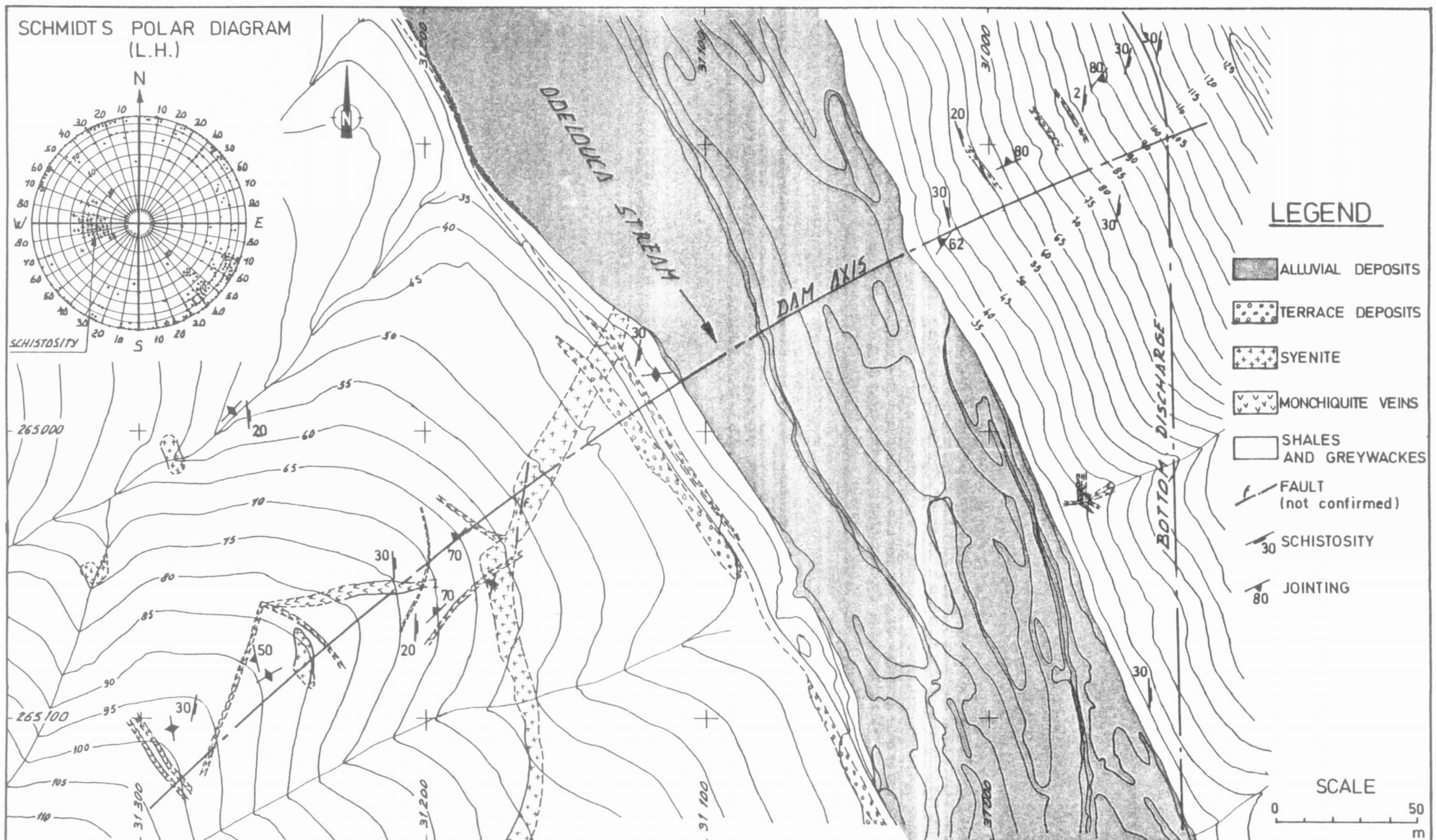


FIG. APPENDIX A21–ODELOUCA DAM SITE

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (1971)

1. Reconnaissance (Oct 1971)

PHASE 2 (1971 - 1972)

1. Trenches (late 1971 - early 1972)

	DESIGNATION	LENGTH (m)
RIGHT BANK	V 2M	30
	V 4M	25
	V 6M	56
	V 8M	20
	V10M	11
	V12M	8
	V 2J	40
	V 4J	30
	V 6J	20
	V 8J	8
	V10J	12
	V12J	21
LEFT BANK	V 1J	15
	V 3J	6

Total no of trenches	14
Total length	302 m
Average length	25.6 m
Cost (assessed)	65×10^3 ESC (1972)

2. Boreholes (Dec 1971 - Jun 1972)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ.(o)	NO OF LUGEON TESTS
RIGHT BANK	9 P8	35.00	90	6
	10 P10	35.00	90	6
	13 P2	65.70	40	12
	14 P4	50.10	40	8
	15 P6	74.00	40	13
	16 P6	40.00	45	7
BOTTOM OF THE VALLEY	1 P0	35.00	90	6
	2 P0	14.00	90	-
	3 P0	35.00	90	7
	4 P0	10.00	90	-
	5 P0	60.00	90	11
	12 P0	65.10	30	11
	18 P0	55.20	40	10
LEFT BANK	6 P1	40.20	90	7
	7 P3	40.10	90	7
	8 P5	35.20	90	6
	11 P0	95.10	35	18
	17 P1	35.00	90	6

Total no of boreholes	18
Total length	819.7 m
Average length	45.5 m
Total no of Lugeon tests	141
Cost	320 x 10^3 ESC (1972)

3. Geophysical exploration (Jun 1972)

3.1 Seismic refraction profiles

19 seismic refraction profiles each 65 m long (18 at the left bank; 1 at the right one), were performed. A 12 geophone array was used with direct and reverse shots.
Total length of profiles - 1 170 m

3.2 Geoelectrical soundings

A total of 10 geoelectrical soundings were carried out at the left bank, using a $\overline{MN}$ = 1 m and max $\overline{AB}$ = 140 m.

3.3 Apparent resistivity profile

A 120 m long apparent resistivity profile was performed at the right bank.

$\overline{AB}$ = 63 m $\overline{MN}$ = 3 m

Cost of geophysical exploration - 110 x 10^3 ESC (1972)

PHASE 3 (1976)

1. Galleries (Jan - Nov 1976)

DESIGNATION		LENGTH (m)
LEFT BANK	G1	40
RIGHT BANK	G2	60

Cost 940 x 10^3 ESC (1976)

2. Geophysical exploration (Feb 1976)

2.1 Apparent resistivity profiles

6 apparent resistivity profiles were carried out at the right bank whose results were used to produce an apparent resistivity map. $\overline{AB}$ and $\overline{MN}$ were 63 m and 3 m respectively.

2.2 Seismic refraction profiles

10 seismic refraction profiles, each 76 m long, were performed at the right bank.

Cost of geophysical exploration - 60 x 10^3 ESC (1976)

3 Shafts (Sep - Nov 1976)

DESIGNATION	AREA (m^2)	DEPTH (m)
(LEFT BANK) P1	3.2	12
(RIGHT BANK) P2	3.2	14

Cost 580 x 10^3 ESC (1976)

PHASE 4 (1977)

(Pos - design)

1. Boreholes (Mar - Sep 1977)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{(o)}$	NO OF LUGEON TESTS
BOTTOM DISCHARGE RESERVOIR SPILLWAY	SDS 3	15.00	90	-
	SDS 1	15.00	90	-
	SDS 2	46.90	90	-
	SME 2	40.00	20	6
	SME 1	48.00	-	7
	SME 3	38.00	20	6
	SDF 3	21.40	90	3
	SDF 2	90.15	90	28
	SDF 1	20.00	90	3
	SL I	8.00	90	-
	SL 2	8.30	90	-
	SL 3	6.62	90	-

Total no of boreholes 12

Total length 357.37 m

Total no of Lugeon tests 53

Cost (assessed) 600 x 10^3 ESC (1977)

2. Trench (Aug - Sep 1977)

A 240 m long trench (3 DS) was excavated along the spillway axis.

Cost (assessed) - 60 x 10^3 ESC (1977)

DAM NAME — SABUGAL

— GENERAL DATA —

LOCATION	—	CÔA RIVER; SABUGAL; GUARDA; PORTUGAL
TYPE	—	EARTH DAM
TOTAL CREST LENGTH	—	≈ 960 m
MAX. HEIGHT ABOVE FOUNDATIONS	—	58.5 m
MAX. HEIGHT ABOVE THE RIVER BED	—	52.5 m
WIDTH OF THE VALLEY (bottom)	—	100 m
RESERVOIR CAPACITY (total)	—	$110.6 \times 10^6 m^3$
FLOODED AREA	—	680 h a
DAM VOLUME	—	$1843 \times 10^3 m^3$
DATE OF THE PROJECT DESIGN	—	AUG 1978
DESIGN PERIOD	—	1973 - 1978
OWNER	—	DIRECÇÃO GERAL DOS RECURSOS E APROVEITAMENTOS HIDRAÚLICOS - LISBOA
DESIGNED BY	—	COBA, SARL - LISBOA
CONSTRUCTION COST (project estimate)	—	250×10^6 ESC
COST OF THE DESIGN	—	6.21×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	—	26 MONTHS
STATUS	—	TO BE CONSTRUCTED

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigations for the Sabugal dam were undertaken in 4 phases: phase 1 1971, phase 2 1971 - 1972, phase 3 1974 - 1975 and phase 4 1976.

1971 - A first geological survey by field mapping was performed.

1971 - 1972 - Second phase of site investigations by means of a trench and boreholes.

1974 - 1975 - A third phase by means of boreholes and geophysical exploration.

1976 - The last phase of site investigations which comprised only geophysical exploration. A final geotechnical assessment report was issued, together with the main design report in August 1978.

The first two phases of site investigations were carried out directly by the owner of the project, to assist the production of a preliminary study. The choice of the type of dam was based on both topographic and geotechnical conditions present at the site.

The dam will be built for irrigation and energy purposes, and will form part of a large scheme incorporating three other dams - Valhelhas dam, Capinha dam and Meimoa dam, and a tunnel linking the Sabugal and Meimoa reservoirs. This tunnel ends with a power house to generate 35 Mw.

The safety structures are a spillway, which consists of a 320 m long channel, and a bottom discharge - a 225 m long tunnel with a diameter of 4.1 m. This tunnel will also function as a temporary diversion during construction. All these structures will be located in the right bank.

FOUNDATION CONDITIONS

1. Rock Type

The regional rock mass consists mainly of schists and greywackes which belong to the Portuguese Ante - Ordovician Complex. At the dam site the lithology is of spotted slates, the result of con - tact metamorphism caused by a neighbouring granitic intrusion(Beira's granite).

The rock mass is overlain by residual soils and slope deposits whose thicknessis greater in the left bank (4 m) than in the right bank (2 m).

Slope and alluvial deposits of sand - clay matrix fill in the bottom of the valley for a width of about 100 m along the dam axis. Their thickness can reach 4 m.

2. Structure

The average attitude of both bedding and schistosity is around N 45° W; (70° - 85°) SW, its strike being near parallel to that of the river. Surface creep was noticed, mainly at the right bank.

The major joint sets are presented in Fig. Appendix A22. No major tectonic fractures were revealed at the site, although some crushed zones parallel to the schistosity had been detected:their width varies from a few centimeters to 3 m.

3. State weathering

Under the overlying deposits and residual soils the rock mass was classified, in general, as moderately to highly weathered (W 3-4) down to a depth which can reach almost 20 m beneath the highest points of the banks. This depth decreases to 4 - 5 m at the river level. Beneath those depths the rock mass is fresh to sligly weathered (W 1 - 2).

It was considered that even the "in - situ" weathered rock mass was suitable for the dam foundation. The predicted depth to be excavated is 6 - 7 m under the river bed and an average of 2 m

in the banks.

4. Permeability

The Lugeon tests performed with the boreholes revealed a permeable rock mass (sometimes with total loss of water during the tests) down to depths of between 20 and 40 m, this depth being smaller beneath the bottom of the valley. The tests were carried out using pressure steps of 0.25; 0.5; 1; 0.5; and 0.25 MPa along borehole sections of 5 m each.

The depth predicted for grouting works in between 20 m and 40 m.

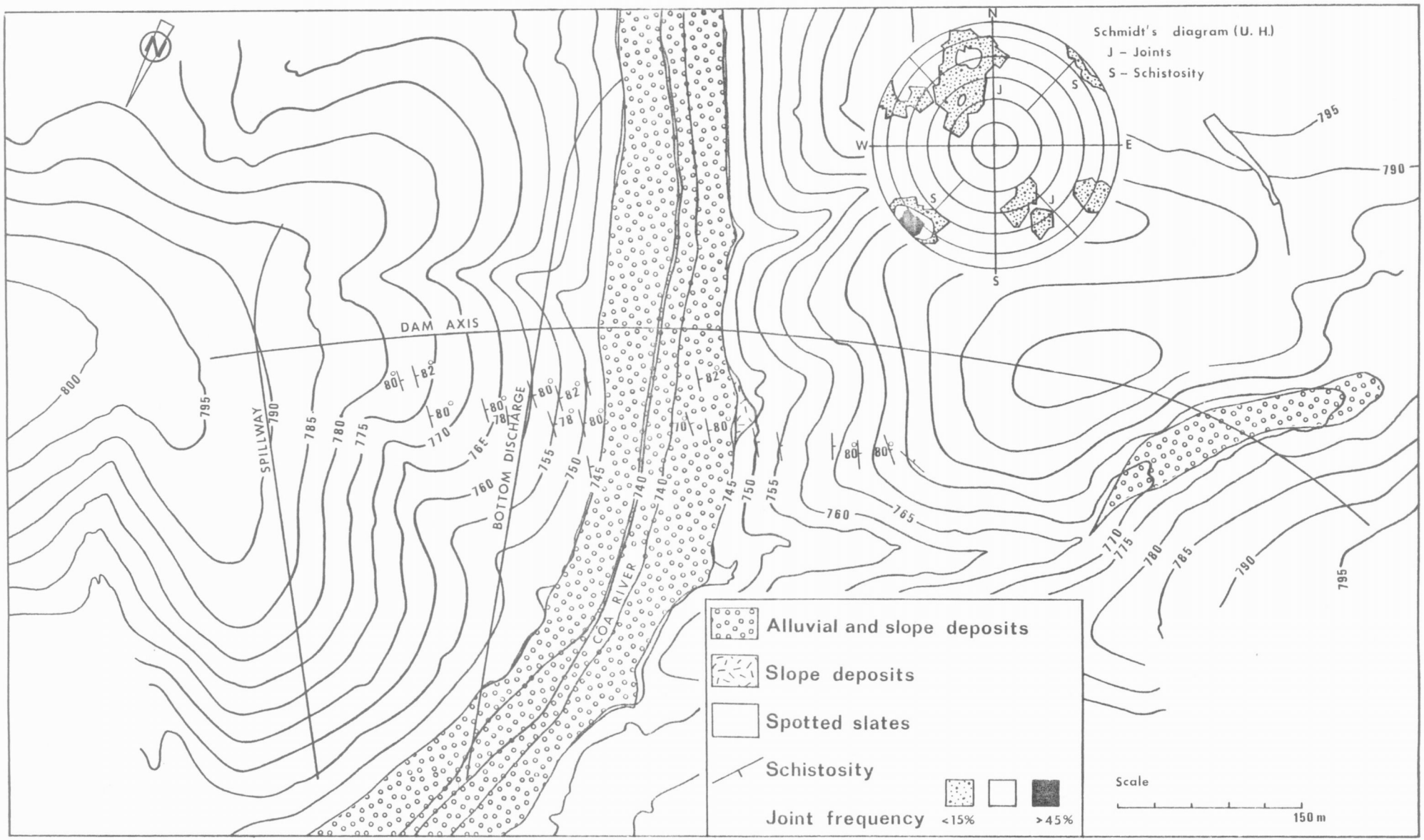


FIG. APPENDIX A22-SABUGAL DAM SITE

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (1971)

1. Reconnaissance (Apr 1971)

PHASE 2 (1971 - 1973)

1. Trench (Aug 1971 - Nov 1972)

One trench along the dam axis.

Total length 395 m

Cost 230×10^3 ESC (1972)

2. Boreholes (Sep 1971 - Jan 1973)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ (°)	NO OF LUGEON TESTS
RIGHT BANK	9 P8	50.50	55	6
	8 P6	45.50	45	8
	7 P4	45.00	45	8
	6 P2	35.50	45	6
BOTTOM OF THE VALLEY	10P0	71.00	45	12
	2 P0	23.00	90	-
	1 P0	86.00	35	16
LEFT BANK	3 P1	51.00	40	9
	4 P3	56.00	45	10
	5 P5	81.00	45	15

Total no of boreholes 10

Total length 544.5 m

Average length 54.5 m

Total no of Lugeon tests 90

Cost 360×10^3 ESC (1972)

PHASE 3 (1974 - 1975)

1. Boreholes (May 1974 - Jan 1975)

DESIGNATION		LENGTH (m)	INCLINATION TO THE HORIZ (°)	NO OF LUGEON TESTS
LEFT BANK	SS 1	71.00	45	12
	SS 3	71.00	30	7
RIGHT BANK	SS 2	66.00	45	11

Total no of boreholes	3
Total length	208 m
Average length	69.3 m
Total no of Lugeon tests	30
Cost	(see below)

2. More boreholes (same period as above)

DESIGNATION		LENGTH (m)	INCLINATION TO THE HORIZ (°)	NO OF LUGEON TESTS
SPILLWAY	SEC 1	4.6	90	-
PROV. DIVER.+ BOTTOM DISCH.	SD 1	30.2	45	4
	SD 2	54.6	50	4
	SD 3	24.6	45	2
	SD 4	2.8	90	-

Total no of boreholes	5
Total length	116.8 m
Total no of Lugeon tests	10
Cost (all boreholes of phase 3)	270×10^3 ESC (1974)

3. Geophysical exploration (Dec 1974)

3.1 Seismic refraction profiles

6 seismic refraction profiles (SS1 - SS6), each 60 m long, performed perpendicular to the dam axis.

8 seismic refraction profiles (SS19 - SS 26), each 60 m long, performed along the spillway axis.

12 seismic refraction profiles (SS 7 - SS 18), each 60 m long, performed along the bottom discharge axis.

The profiles were executed using the direct and reverse shot method.
Total length of profiles - 1 560 m

3.2 Apparent resistivity profile

A 485 m long apparent resistivity profile was performed along the spillway axis, using $\overline{AB}$ = 45 m and $\overline{MN}$ = 5 m.
Cost geophysical exploration 105×10^3 ESC (1974)

PHASE 4 (1976)

1. Geophysical exploration (Apr 1976)

1.1 Seismic refraction profiles

11 seismic refraction profiles with an average length of 70 m were performed along the dam axis at the left bank.
Total length of profiles - ≈ 770 m

1.2 Apparent resistivity profile

1 apparent resistivity profile, 150 m in length was performed at the left bank, using $\overline{AB}$ = 65 m and $\overline{MN}$ = 5 m.

1.3 Geoelectrical soundings

2 geoelectrical soundings were performed at the left bank, using a max $\overline{AB}$ = 100 m and $\overline{MN}$ = 5 m
Cost of geophysical exploration 30×10^3 ESC (1976)

TUNNEL NAME — ALTO LINDOSO

— GENERAL DATA —

LOCATION	—	LIMA RIVER; 29 KM NE OF PONTE DA BARCA; PORTUGAL
LENGTH	—	5 729 m
DIAMETER	—	9m (internal)
LINING THICKNESS	—	50 cm (average)
OVERBURDEN BETWEEN PORTALS	MAX —	≃ 400 m
	MIN —	≃ 95 m
DATE OF THE PROJECT DESIGN	—	JUL 1977
DESIGN PERIOD	—	1976 - 1977
OWNER	—	ELECTRICIDADE DE PORTUGAL
DESIGNED BY	—	ELECTRICIDADE DE PORTUGAL
CONSTRUCTION COST (project estimate)	—	2 430 x 10^6 ESC
COST OF THE DESIGN	—	N.A.
CONSTRUCTION PERIOD (project estimate)	—	3.5 YEARS
STATUS	—	TO BE CONSTRUCTED

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigations for this tunnel were undertaken in two phases; Phase 1 1976, phase 2 1976 - 1978.

1976 - Basic geological survey by field mapping , carried out in mid 1976.

1977 - 1978 - The second phase of site investigations took place from August 1976 until September 1978. An engineering geological assessment report was produced in June 1977, based on the data available at the time, for the project design which was issued in July 1977.

This tunnel forms part of the Alto Lindoso scheme (see Alto Lindoso dam) and is designed to work under a pressure of between 0.55 and 0.8 MPa.

A previously chosen location for the tunnel (South route) was abandoned for purely geological reasons, although the new location (North route) results in an increase of about 200 m in length. The exixtence of some wide faults, which would be intersected at unfavorable angles, as well as the poor quality of the rock mass to be excavated, led to an investigation of this new route. It is this new route and the respective site investigations which will be dealt with here.

Part of the mechanical characterization tests were not available at the time the design was released; those were completed more than one year later, and so their results could not be taken into account in the geotechnical report produced in 1977.

It was thought, based upon the results of the exploration works and upon the experience from other Portuguese tunnels, that a permanent support would not be needed except in the case of crushed or more weathered zones due to the presence of some faults. It was anticipated that those zones would not be longer than 10% of the total length of the tunnel and a suitable support would be designed according to the conditi - ons encountered during construction. For the remaining 90% of the len-

gth, a lining 0.5 m thick was designed in order to reduce the wall roughness and to prevent any later detachment of blocks. This thickness will be reduced to 0.4 m in particular zones of fresh rock mass with deformability moduli greater than 20 000 MPa.

ROCK MASS CONDITIONS

1. Rock Type

The rock mass to be excavated consists of two types of granite (see Fig. Appendix A23): a medium to coarse grain granite and a muscovite and biotite pseudoporphyritic granite. Some mica - schists will be traversed but they will be within the last 100 m of the tunnel length. At the contact between the granite and the schists a thin band of hornfelds occurs.

2. Structure

Both, the granites and the mica - schists to be excavated are intruded by aplite, pegmatite and quartz veins with a maximum width of 3 m and by 0.3 to 2.5 m wide dolerite dykes, their more frequent attitudes being N (30^{o} - 50^{o}) E subvertical, and N (65^{o} - 80^{o}) subvertical. With respect to faults present at the location, four main sets were assessed, whose attitudes are presented in the table below.

DIRECTION(o)	INCLINATION (o)	FAULT SPACES (m)
N10W TO N20E	70 E TO 75W	0.01 - 1.2
N(30-50)E	70SE TO 70W	0.02 - 2.5
N(60-80)E	70NNW TO 75SSE	0.01 - 3.5
N(35-60)W	70NE TO 70SW	0.01 - 1.0

The infilling is either crushed granite or schist, depending upon the rock type crossed by the faults, with some clay and iron oxides.

In addition to horizontal to subhorizontal sets, the last ones having several directions, the joint set assessment revealed the existence of the following main sets:

A- MEDIUM TO COARSE GRAIN GRANITE

DIRECTION (o)	INCLINATION (o)
N(25-50) E	60NW TO VERT
N(25-50) W	70NE TO VERT
N80E TO E-W	SUBHORIZONTAL

B- PSEUDOPORPHYRITIC GRANITE

DIRECTION (o)	INCLINATION (o)
N(0-30) E	70E TO VERT.TO 70W
N(80-90) E	VERT.
N(30-50) E	70 SW TO VERT.

The direction of the tunnel is approximately NNW - SSE.

3. State of weathering

The state of weathering of the granite at the surface is very irregular. In some outcrops it occurs fresh but in some areas the weathering can reach 40 to 60 m in depth as proved by boreholes. Such alteration is from surface weathering rather than hydrothermal activity.

4. Water

About 80 Lugeon tests were carried out with the borehole investigation. As an overall conclusion, and because of the thickness of rock above the excavation, the problems which can arise due to water inflow into the tunnel will be related to the crossing of particular zones like those of faults.

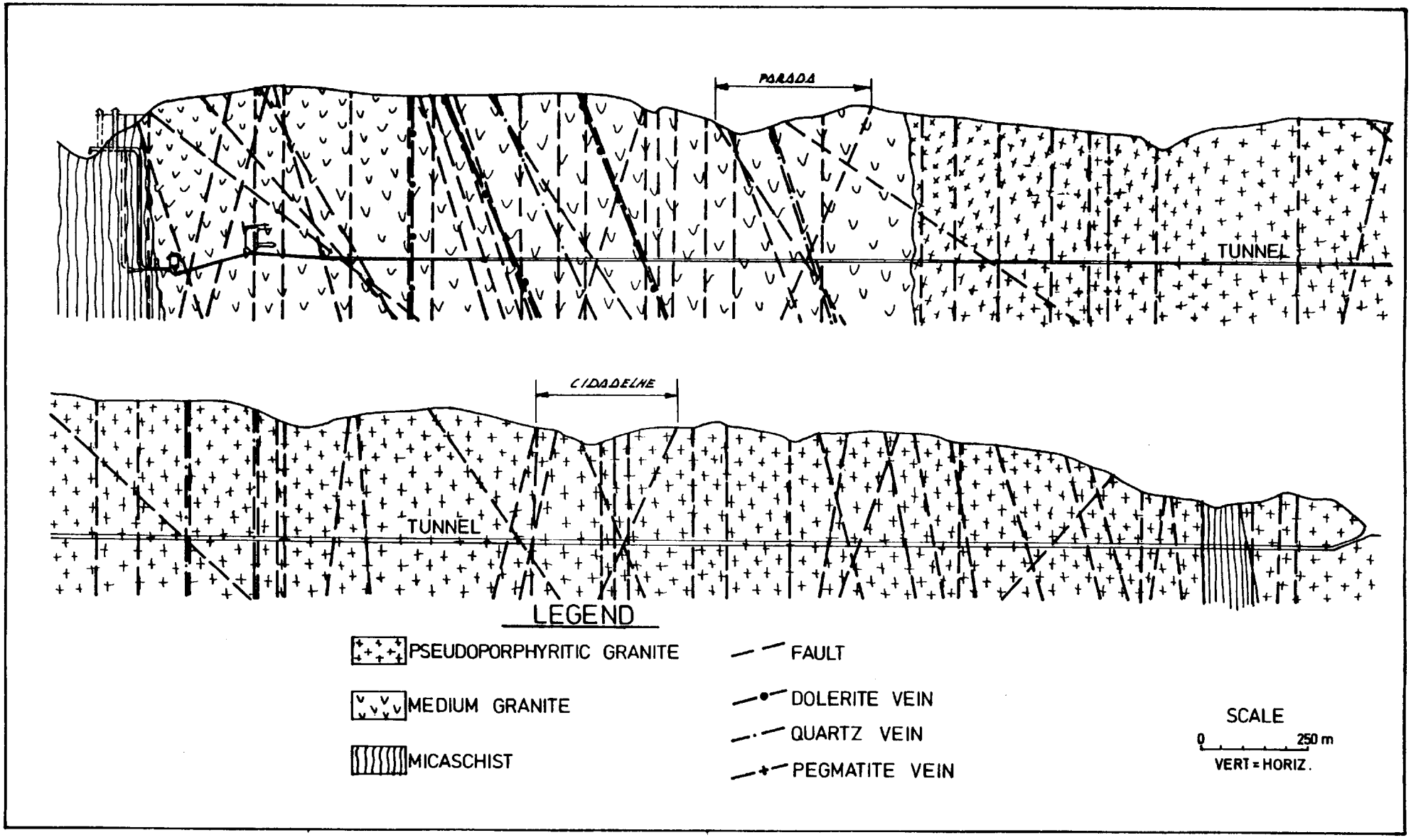


FIG. APPENDIX A23-ALTO LINDOSO TUNNEL

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (1976)

1. Reconnaissance (mid 1976)

PHASE 2

1. Boreholes (Aug 1976 - Jun 1977)

DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{(o)}$	NO OF LUGEON TESTS
R 1	52.4	60	9
R 5	51.0	45	9
R 6	80.0	45	13
R 7	50.1	45	8
R 4	25.0	60	-
R 3	32.3	60	5
RV2	103.1	90	20
R 2	62.2	60	8
RV3	7.4	90	-
RV3A	55.2	90	5
R 8	35.0	45	-
R 9	29.9	60	-

Total no of boreholes	12
Total length	583.6 m
Average length	48.6 m
Total no of Lugeon tests	77
Cost	920 x 10^3 ESC (1977)

2. Trenches (Feb - Mar 1977)

Total no of trenches	28
Total length	340 m
Cost	170 x 10^3 ESC (1977)

3. "In - situ" tests (Mar 1977 - Sep 1978)

3.1 Dilatometer tests

Device used	BHD - LNEC
No of boreholes tested	6
Horizons measured in each borehole	5
Directions measured in each horizon	4
No of loading cycles	3 - 4
Maximum pressure applied	8 or 10 MPa
Total no of horizons measured	30

The tests were performed between the depths 14.0 and 60.0 m.

Results:

ROCK TYPE (LOCAL CLASSIFICATION)	DILATOMETER MODULUS (MPa)		
	MAX	MEDIAN	MIN
GOOD	49 700	17 600	5 100
REASONABLE	21 100	5 000 *	2 200
WEATHERED	8 400	5 500	3 500
MICASCHIST	41 000	20 500 **	6 300

* Apparent anomalous value due to the low value registred in one borehole with a minimum of 2 200 MPa.

** Average

3.2 State of stress "In - situ"

4 tests were performed using the strain tensor gauge (LNEC) along the borehole RV2, at 4 different depths.

Cost of "in - situ" tests 560×10^3 ESC (1977)

4. Laboratory tests (same period as above)

4.1 Uniaxial tests

Total no of tests 58 (53 granite + 5 schist)

Type of samples: cut from rock cores.

Diameter 54 mm and seldom 84 mm

Aspect ratio 2 - 2.5

ASSESSMENT OF E

No of loading cycles 3

Max pressure applied 10 or 15 MPa

Results:

ROCK TYPE (LOCAL CLASSIF)	E (MPa)			NO OF TESTS
	MAX	MIN	AVER	
GOOD	61 200	2 400	32 400	28
REASONABLE	30 200	1 200	9 800	20
WEATHERED	7 000	1 800	3 800	10
MICASCHIST ONLY	88 800	7 000	50 300	-

ASSESSMENT OF σ_{uc}

Tests performed on the above samples, increasing pressure up to failure.

Results:

ROCK TYPE (LOCAL CLASS.)	σ_{uc} (MPa)		
	MAX	MIN	AVER
GOOD	159	46	112
REASONABLE	84	13	42
WEATHERED	21	7	15
MICASCHISTS ONLY	144	21	80

4.2 Ultrasonic wave velocities

(same samples used for uniaxial tests)

Length of samples	120 to 150 mm
No of samples tested	38
Measured velocities	V_p

Results:

Values of V_p for the granite classified as "good" varied between 2 800 and 5 000 m /sec., being almost higher than 3 500 m/sec. For the granite classified as reasonable or weathered those values varied between 1 000 and 2 750 m/sec..

4.3 Point load tests

Type of samples: rock core pieces corresponding to the material used in the uniaxial tests. Aspect ratio > 2.
Total no of samples tested - 33

Results:

The test results showed a good agreement with the results of σ_{uc}, according to the equation $\sigma_{uc} = 24\ I_s$.

4.4 Schmidt Hammer tests

Hammer type	L
Total no of samples tested	38

Results:

ROCK TYPE (LOCAL CLASS.)	REBOUND NUMBER			NO OF SAMPLES
	MAX	MIN	AVER	
GRANITE "GOOD"	52	38	45	21
GRANITE "REASONABLE OR WEATHERED"	40	22	29	12
SCHIST "GOOD"	42	38	40	3
SCHIST "REASONABLE OR WEATHERED"	30	28	29	2

4.5 Other tests

4.5.1 Porosity

Total no of samples tested 18

4.5.2 Bulk density

Total no of samples tested 18

Cost of laboratory tests 110×10^3 ESC (1977)

TUNNEL NAME — CASTELO DO BODE

— GENERAL DATA —

LOCATION	—	FROM THE CASTELO DO BODE DAM SITE TO NEAR NABÃO RIVER; PORTUGAL.
LENGTH	—	5 090 m
DIAMETER	—	3.0m (internal)
LINING THICKNESS	—	35 cm
OVERBURDEN BETWEEN PORTALS	MAX —	90 m
	MIN —	30 m (10m along the last 300 m)
DATE OF THE PROJECT DESIGN	—	MAR 1979
DESIGN PERIOD	—	1975 - 1979
OWNER	—	EPAL - EMPRESA PÚBLICA DAS ÁGUAS DE LISBOA.
DESIGNED BY	—	COBA, SARL - LISBOA
CONSTRUCTION COST (project estimate)	—	590×10^6 ESC
COST OF THE DESIGN	—	8×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	—	N.A.
STATUS	—	SEE GENERAL REMARKS

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

This tunnel was designed for a new water supply system which will take water from the Castelo do Bode Reservoir to Lisbon. The site investigations were undertaken in three phases: phase 1 1975, phase 2 1975-1976, phase 3 1978.

1975 - Basic geological survey by field mapping. A report including a programme for further site investigations was produced in August 1975.

1975 - 1976 - Second phase of site investigations which began in November 1975, included geophysical exploration, boreholes, "in - situ" and laboratory tests. A geotechnical assessment was produced in a report dated September 1976.

1978 - Third phase, which included a detailed study of jointing by rock core analysis and some laboratory tests. Another geophysical exploration programme was performed, as well. A report dealing with the special study of jointing was produced in October 1978. A final geotechnical assessment report was issued as part of the main design report in February 1979.

During the 2nd phase of the S.I. two depths were considered possible for the tunnel axis and so the works were designed to take into consideration both solutions. In the final design, after the 3rd phase of S.I., however, an intermediate depth was selected for hydraulic reasons and so, some of the performed "in - situ" tests fell outside the zone to be excavated. This was thought to be unimportant because the average characteristics of the rock mass had already been satisfactorily assessed.

After the second phase, the consultant engineer suggested a special study of

After the 2nd phase, the consultant engineer suggested a special study of

jointing, hence the reason for a third phase. But, in the meantime, as the position of the last section of the tunnel varied, a new geophysical exploration programme was thought necessary and it was performed.

The tunnel was designed for working under a maximum internal pressure of 0.9 MPa. The rock mass was divided, as a result of the performed studies, into three geotechnical zones:

GEOTECHNICAL ZONE	DILATOMETER MODULUS (MPa)	JOINT FRICTION ANGLE (°)	STATE OF WEATHERING
ZG 1	> 5 000	35	W1-2
ZG 2	2 000-5 000	30	W3
ZG 3	< 2 000	20	W4-5

Different supports were designed for each zone and the stresses on the support were calculated according to both Wickam and Rocha's criteria.

Recently this design was abandoned and a new layout for the water supply scheme was considered which requires a shorter tunnel in the same geological formations.

ROCK MASS CONDITIONS

1. Rock Type

The tunnel would be excavated in a pre - Cambrian metamorphic formation of gneisses and some migmatites . In some of the highest areas these rock types are overlain by Plio - Pleistocene sand and clay sediments but these do not interfere with the excavation (see Fig. Appendix A24).

2. Structure

The rock mass has been subjected to high tectonic forces throughout the geological past.

Despite large scatter presented by joint orientations, four main sets were assessed:

SET	DIRECTION (°)	INCLINATION (°)
A	PARALLEL TO GNEISSOSITY	
B	N (50 - 120) E	50 N TO 50 S
C	N (20 - 100) E	(15 - 75) SW
D	SUBHORIZONTAL	

The general attitude of the gneissose fabric is N (10° - 40°) W, 20° E to subvertical. The tunnel alignment is N 70° E.

Some features follow the gneissose fabric, streams flowing along them in some cases, were detected by the use of aerial photographs. Also detected were some crushed zones parallel to gneissosity; these are possibly related to faults, and have an average thickness of 10 cm reaching a maximum thickness of about 3 m. In some cases these zones are very close to each other.

The joint frequency was found to decrease with depth, being 5.1 joints/m between 0 - 10 m, 2.3 joints/m between 10-30 m and 2 joints/m below 30 m.

Some pegmatite, aplite and quartz veins with a maximum thickness

of 10 cm cross the entire rock mass.

3. State of weathering

The excavation would deal with rock mass zones varying from W 4 - 5 (highly to completely weathered), to W 1 - 2 (fresh to slightly weathered) - see Fig. Appendix A24.

4. Water

Some superficial aquifers, due to infiltration, were detected but they would interfere with the excavation only when it took place near the surface. Percolation through the discontinuities was reported to be more important. An artesian condition was detected in boreholes SCB 3 (flow of 360 1/h at depth of 52.5 m) and SCB 4(flow of 2 100 1/h at depth of 30 m).

The depth of the water table varies from a maximum of 30 m in the zones of highest overburden to very close to the surface beneath some valleys.

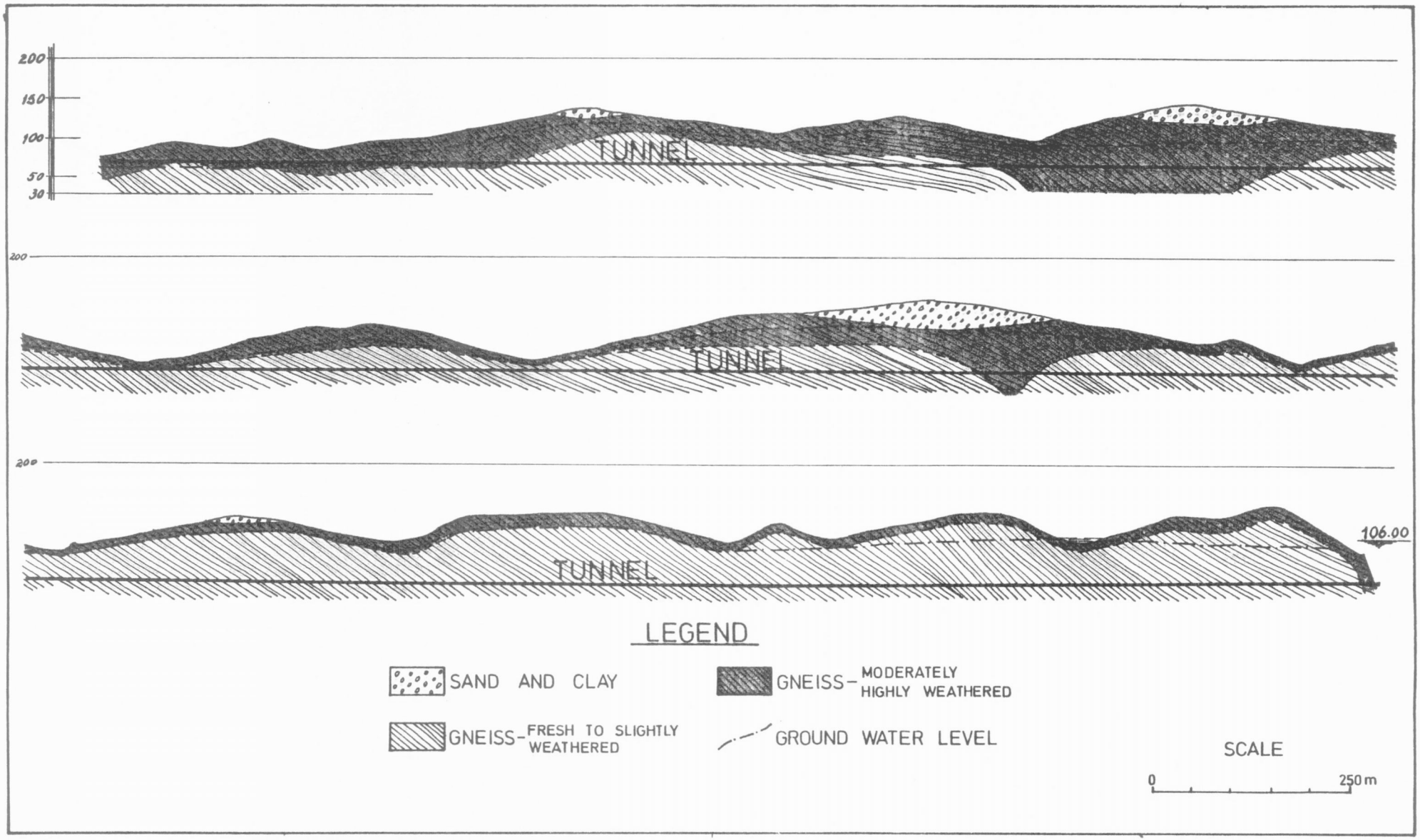


FIG. APPENDIX A24-CASTELO DO BODE TUNNEL

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (1975)

1. Reconnaissance

PHASE 2 (Nov 1975 - Sep 1976)

1. Geophysical exploration (Nov 1975)

Seismic refraction profiles (direct and reverse shots; spacing of geophones - 6 m) in areas where the overburden was less than 50 m.

DESIGNATION	LENGTH (m)	NO OF PROFILE
PS 13 TO PS 19	76	7
PS 21 , PS 22	76	2

More seismic refraction profiles using the same geophone spacing as above but with direct , reverse, and central shots:

DESIGNATION	LENGTH(m)	NO OF PROFILE
PS 101, PS 102,	76	7
PS 109, PS 120,		
PS 113, PS 120,		
PS 121,		
PS 103 TO PS108	120	14
PS 111, PS 112		
PS 114,TO PS119		

Total length of profiles 2 896 m

Cost of geophysical exploration - 75×10^3 ESC (1975)

2. Boreholes (Dec 1975 - Aug 1976)

Two of the boreholes with a diameter of 101 mm. The remaining , NX.

DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{(o)}$	NO OF LUGEON TESTS
SCB 1	110.10	70	2
SCB 2	70.00	70	2
SCB 3	65.05	70	3
SCB 4	30.00	70	3
SCB 5	112.00	90	-
SCB 6	40.00	70	-
SCB 7	30.00	70	1
SCB 8	92.00	90	2
SCB 8A	36.00	90	3
SCB 9	27.00	70	3
SCB 10	48.00	90	-
SCB 11	22.00	70	3
SCB 11A	21.00	70	2

Total no of boreholes	13
Total lengh	703.15 m
Average length	54.10 m
Total no of Lugeon tests	24
Cost	1970 x 10^3 ESC (1976)

3. "In - situ" tests (Jun - Aug 1976)

3.1 Dilatometer tests

Device used	BHD - LNEC
No of boreholes tested	6
Horizons measured in each borehole	3
Directions measured in each horizon	4
No of loading cycles	3
Maximum pressure applied	6 to 9 MPa
Total no of horizons measured	18

The tests were performed between the depths of 6.00 and 67.58 m.

Results:

Average from each borehole

BOREHOLE	DILATOMETER MODULUS E(MPa)	NO OF HORIZONS MEASURED
SCB 3	7 040	3
SCB 4	6 340	3
SCB 7	5 250	3
SCB 8 *	19 780	3
SCB 9	3 290	3
SCB 11	4 560	3

* Tests performed in a selected and particularly fresh zone.

3.2 State of stress "In - situ"

4 tests were performed using the strain tensor gauge (LNEC) at depths of 42.35m, 45.70 m, 97.70 m and 98.95 m.

Cost of "In-situ tests" 620×10^3 ESC)1976)

4. Laboratory tests (Sep 1976)

4.1 Uniaxial tests

Total no of tests 17

Type of samples: cut from rock cores.

Diameter 54 mm and 84 mm

Aspect ratio 2.5 - 3

ASSESSMENT OF E

No of loading cycles 2

Max pressure applied 15 MPa, except in some poor quality samples, where 10 MPa was used.

Results

E (MPa)			ν		
MAX	MIN	AVER	MAX	MIN	AVER
76 500	4 900	27 010	0.23	0.0	0.10

ASSESSMENT OF σ_{uc}

Tests performed in the above samples, increasing pressure up to failure.

Results:

σ_{uc} (MPa)		
MAX	MIN	AVER
83.0	12.6	39.0

4.2 Ultrasonic wave velocities

(samples used for uniaxial tests and some others)

Length of samples 120 - 150 mm

No of samples tested 42

Measured velocities V_p

Results:

V_p (m/ sec .)		
MAX	MIN	AVER
5 500	1 350	3 429

4.3 Point load tests

Type of samples: from rock cores

Aspect ratio > 2 (diametral tests)

Total no of samples tested 20

Results:

I_s (MPa)		
MAX	MIN	AVER
6.07	0.24	1.79

4.4 Schmidt Hammer tests

Hammer type: L

Total no of samples tested: 21

Results:

HARDNESS (R)		
MAX	MIN	AVER
44	14	31

Cost of laboratory tests - 25×10^3 ESC (1976)

PHASE 3 (1978)

1. Detailed study of jointing by rock core analysis

2. Geophysical exploration

Apparent resistivity profiles, performed by using two different $\overline{AB}$ = values in order to investigate two different depths.

DESIGNATION	$\overline{AB}$ (m)	$\overline{MN}$ (m)	APPROX LENGTH(m)
PR1 ; PR5	90; 180	5	150
PR3;PR4;PR6	90; 180	5	240
PR2	90; 180	5	480
PR7	90; 180	5	630

Total length of resistivity profiles 4 260 m

Costs of these profiles 200×10^3 ESC (1978)

3. Laboratory tests (Aug 1978)

3.1 Uniaxial tests

Total no of tests 17

Type of samples: cut from rock cores

Diameter 54 mm and 84 mm

Aspect ratio 2.5

ASSESSMENT OF E

No of loading cycles 3

Max pressure applied 5 - 8 MPa

(less than 1/3 of the estimated unconfined compressive strength)

Results:

E (MPa)			ν		
MAX	MIN	AVER	MAX	MIN	AVER
88 900	1 200	19 300	0.22	0	0.07

ASSESSMENT OF σ_{uc}

Tests performed on the above samples, increasing pressure up to failure.

Results:

σ_{uc} (MPa)		
MAX	MIN	AVER
100	7.1	31.4

3.2 Joint shear tests

Total no of samples tested 17

Type of samples: prepared from jointed NX rock cores

Area of shear 30 - 75 cm

No of tests per sample 4

Results:

c (MPa)			ø (o)		
MAX	MIN	AVER	MAX	MIN	AVER
0.48	0.02	0.18	43.0	25.0	34.7

Cost of these laboratory tests 50×10^3 ESC (1978)

TUNNEL NAME — FUNCHO - BENACIATE

— GENERAL DATA —

LOCATION	— NEAR SILVES; ALGARVE; PORTUGAL
LENGTH	— 1 410 m
DIAMETER	— 2.9m (internal)
LINING THICKNESS	— 25 cm
OVERBURDEN BETWEEN PORTALS MAX	— 50 - 89 m
OVERBURDEN BETWEEN PORTALS MIN	—
DATE OF THE PROJECT DESIGN	— NOV 1980
DESIGN PERIOD	— 1976 - 1980
OWNER	— DIRECÇÃO GERAL DOS RECURSOS E APROVEITAMENTOS HIDRAULICOS - LISBOA
DESIGNED BY	— COBA, SARL - LISBOA
CONSTRUCTION COST (project estimate)	— 300 x 10^6 ESC
COST OF THE DESIGN	— 5.5 x 10^6 ESC
CONSTRUCTION PERIOD (project estimate)	— 1.5 YEAR
STATUS	— TO BE CONSTRUCTED

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

This project deals with 3 contiguous tunnels which form part of a water conduit from the Funcho Reservoir to near Benaciate. In a first proposal, the construction of one tunnel linking the two sites was considered but for reasons of economy a combined undertaking, comprising a channel 10 290 m in length and three small tunnels, was selected. These tunnels are referred to here as initial (460 m), intermediate (150 m)and final (800 m).

The site investigation were carried out in 2 phases: phase 1 1976 - 77, phase 2 1977 - 78.

1976 - 77 - Basic geological survey by field mapping. A geotechnical assessment report was produced in March 1977.

1977 - 78 - Second phase of S.I. comprising boreholes, geophysical exploration, "in - situ" and laboratory tests. A final geotechnical report was produced together with the design in November 1980.

The first geotechnical report proposed a second phase of S.I.. For several reasons, a project design was produced in 1977 which focused only upon the recommendations made in that report. In the meantime, the project was to be redesigned, for hydraulic reasons, and the new 1980 design took into account results from the second phase of S.I..

In the first geotechnical report, average values for the mechanical characteristics of the rock masses were suggested, based on the results of tests performed for a neighbouring dam site (Funcho). Here, the geology is basically similar to that of the site crossed by the tunnels. The table below presents the values suggested in the 1977 geotechnical report. The suggested deformability moduli of the rock mass can be compared with the results of the tests performed during the second phase of S.I., which are presented later.

	ROCK TYPE	E (MPa)	c(JOINTS) (MPa)	∅(JOINTS) (o)
VALUES FROM THE TESTS AT FUNCHO DAM SITE	GREYWACKE	20 000	0.4	35
	SHALE	10 000	0.3	25
VALUES SUGGESTED FOR THE FUNCHO--BENACIATE TUNNELS	GREYWACKE AND SHALE (INITIAL TUNNEL)	15 000	0.3	30
	GREYWACKE AND SHALE (INTERM.TUNNEL)	8 000 TO 10 000	0.3	25

For the limestone and dolerite associated with the final tunnel and not present at Funcho dam site, the following values were suggested:

ROCK TYPE	E (MPa)	c(JOINTS) (MPa)	∅(JOINTS) (o)
DOLERITE	15 000	0.5	35
LIMESTONE	20 000	0.5	40

The suggested values above, were used in the 1977 design, except that for the limestone , in which c = 1 MPa and $\emptyset = 40^{o} - 45^{0}$ were used.

In the 1977 design, two linings of thickness 25 cm and 35 cm were designed, the last one to be used near the portals and in poorer zones of the rock mass. Two different reinforcements were designed for the se two zones.

Based upon the results of the second phase of S.I., the rock mass was divided into 3 geotechnical zones and in the 1980 design a thickness of 35 cm for the final support was designed for the zones near the portals and a thickness 25 cm was designed for the remainder of the zones. The studies of the second phase also allowed the design of lighter reinforcements than the ones of the 1977 design. The support reinforcements were designed taking into account different geotechnical classifications for tunnels, based on the results of the geotechnical zoning.

ROCK MASS CONDITIONS

1. Rock Type

The initial and intermediate tunnels will be excavated through Carboniferous shales and greywackes of flysch facies. The major part of the final tunnel is through limestones. A 190 m wide dolerite dyke is also to be excavated. The upper contact between the dolerite and the limestone is of a breccia beneath a marly clay. The first 60 m of the final tunnel goes through calcareous marls intersected by two dolerite veins 5 m and 12 m thick (see Fig. Appendix A26).

2. Structure

Measured schistosity lies around N(40^{o}W - 5^{o}E); (45^{o} - 65^{o})E in the case of the initial tunnel and around N(20^{o} - 30^{o})W; (40^{o} - 60^{o})E in the intermediate one. The axis of the initial and the intermediate tunnel makes angles of 30^{o} - 50^{o} and about 40^{o} respectively with the schistosity.

The attitude of the major formation in the final tunnel lies between N (45^{o} - 55^{o}) E; (15^{o} - 30^{o}) S.

Shales and greywackes present high deformations showing a succession of isoclines and they are crossed by several crushed zones and quartz veins.

By analysing aerial photographs, some tectonic alignments with a general orientation of NW - SE were noticed, but no major faults were confirmed during mapping because of the presence of superficial deposits.

3. State of weathering

According to the performed studies, the initial and intermediate tunnels are through a rock mass that is highly to completely weathered (W 4 - 5), down to 10 m from the surface. Between this depth and a maximum of 30 m, it is moderately weathered (W 3) becoming fresh to slightly weathered (W 1 - 2) from 30 m downwards. Crushed zones, at depth, are characteristically highly to

completely weathered (W 4 - 5).

The rock mass of the final tunnel presents a W 4 - 5 state of weathering down to a maximum depth of 15 m, which is followed by a W 3 or W 2 rock mass. An exception is made for the breccia where the weathering is deeper.

4. Water

Lugeon tests were carried out together with the boreholes, using mainly pressure steps of 0.5; 1; 0.5, MPa. It was concluded that except for the zones near the portals as well as for the limestones of the final tunnel, the absorptions were lower than 1 Lugeon, at the depths to be excavated. For the limestones there was a total loss of water during the tests.

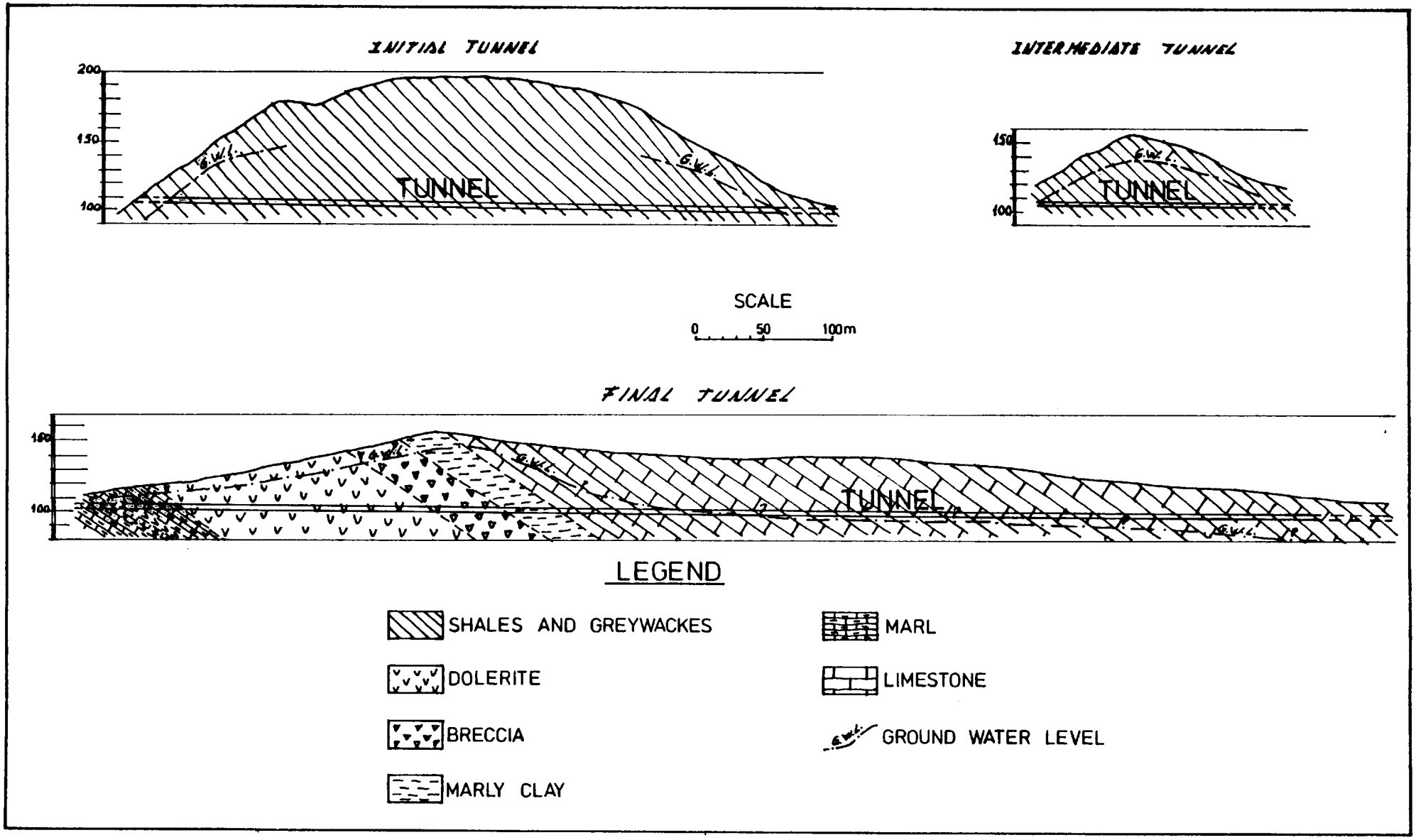


FIG. APPENDIX A25-FUNCHO-BENACIATE TUNNELS

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (Dec 1976)

1. Reconnaissance

PHASE 2 (Jun 1977 - Oct 1978)

1. Boreholes (Jun - Nov 1977)

	DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{(o)}$	NO OF LUGEON TESTS
INITIAL TUNNEL	ST 1	23.2	90	3
	ST 2	50.2	90	3
	ST 3	50.2	60	3
	ST 4	27.3	60	3
INTERMEDIATE TUNNEL	ST 5	23.3	70	3
	ST 5A	30.1	90	3
	ST 6	55.4	70	5
	ST 7	34.3	70	6
FINAL TUNNEL	ST 8	25.3	90	3
	ST 9	45.1	90	3
	ST 10	63.5	90	3
	ST 11	50.5	90	3
	ST 12	23.5	90	3
	ST 13	58.1	90	3
	ST 14	23.4	90	3

Total no of boreholes	15
Total length	583.4 m
Average length	38.9 m
Total no of Lugeon tests	50
Cost	1 050 x 10^3 ESC (1977)

2. Geophysical exploration (Aug 1977)

Seismic refraction profiles (direct and reverse shot method), mainly near the tunnel's portal zones:

DESIGNATION		LENGTH (m)
INITIAL TUNNEL	SP 0	120
	PS 1	120
INTERM. TUNNEL	PS 7	76
	PS 8	76

DESIGNATION		LENGTH (m)
FINAL TUNNEL	PS 23	65
	PS 24	120
	PS 25	120
	PS 26	65

Total no of profiles 8

Total length of profiles 762 m

Cost 30×10^3 ESC (1977)

3. "In - situ" tests (Oct 77 - Apr 78)

3.1 Dilatometer tests

Device used BHD - LNEC

No of boreholes tested 12 (initial tunnel - 3; intermediate tunnel - 3; final tunnel - 6.

Horizons measured in each borehole 3 to 6

Directions measured in each horizon 4

No of loading cycles 1 to 4

Total no of horizons measured 52

The tests were performed between the following depths: initial tunnel - 9 to 43 m ; interm. tunnel - 21.80 to 52.05 m; final tunnel - 12.90 to 48.60m.

Results:

TUNNEL/ROCK TYPE		DILATOM.MODULUS E (MPa)			NO Of HORIZONS MEASURED
		MAX	MIN	AVER	
INITIAL + INTERM.	GREYWACKE	50 000	21 400	36 300	6
	GREYWACKY SHALE	17 200	2 600	7 100	22
FINAL	DOLERITE	23 600	3 900	9 900	8
	LIMESTONE	5 000	1 300	3 000	12
	BRECCIA	11 200	3 700	6 000	4

Cost of "in - situ" tests 560×10^3 ESC (1977)

4. Laboratory tests (same period as above)

4.1 Uniaxial tests

Total no of tests 31

Type of samples: cut from rock cores

Diameter 54 mm

Aspect ratio 2 - 3

ASSESSMENT OF E

No of loading cycles 3

Max pressure applied Almost 4 MPa

Results:

ROCK TYPE	E (MPa)			ν			NO OF SAMPLES
	MAX	MIN	AVER	MAX	MIN	AVER	
GREYWACKE	77 800	74 100	76 000	-	-	0.27 *	2
GREYWACKY SHALE	7 250	4 300	2 070	-	-	-	10
DOLERITE	80 000	10 500	42 400	0.33	0.16	0.24	7
LIMESTONE	82 200	9 400	64 300	0.36	0.15	0.26	7
BRECCIA	11 600	1 800	5 700	0.14	0.05	0.10	5

* Only 1 sample

ASSESSMENT OF σ_{uc}

Tests performed on the above samples, increasing pressure up to failure.

Results:

ROCK TYPE	σ_{uc} (MPa)			NO OF TESTS
	MAX	MIN	AVER	
GREYWACKE	222	117	170	2
GREYWACKY SHALE	87	9	55	9
DOLERITE	146	24	81	7
LIMESTONE	101	14	72	7
BRECCIA	42	13	27	5

4.2 Point load tests

Type of sample: from rock cores

Aspect ratio ≈ 1 (axial tests)

Total no of samples tested 19

Results:

ROCK TYPE	I_s (MPa)			NO OF TESTS
	MAX	MIN	AVER	
GREYWACKE	12.5	4.4	8.5	2
GREYWACKY SHALE	3.7	1.6	2.6	4
DOLERITE	6.4	2.5	4.3	5
LIMESTONE	4.0	2.0	2.8	4
BRECCIA	3.3	0.8	1.7	4

4.3 Ultrasonic wave velocities

(Almost of the samples used for the uniaxial tests)

Length of samples 110.8 - 142.8 mm

No of samples tested 29

Measured velocity V_p

Results:

ROCK TYPE	V_p (m/sec)			NO OF TESTS
	MAX	MIN	AVER	
GREYWACKE			5 550	1
GREYWACKY SHALE	4 690	2 500	3 569	8
DOLERITE	5 670	3 780	4 568	8
LIMESTONE	5 860	3 130	5 430	7
BRECCIA	3 390	1 720	2 400	5

4.4 Schmidt hammer tests

Hammer type: L

Total no of samples tested 3

Results:

ROCK TYPE	HARDNESS (R)	NO OF SAMPLES
GREYWACKY SHALE	37	1
LIMESTONE	27	1
DOLERITE	29	1

Cost of laboratory tests 40×10^3 ESC (1977)

TUNNEL NAME — SABUGAL - MEIMOA

— GENERAL DATA —

LOCATION	— LINKS SABUGAL AND MEIMOA RESERVOIRS; GUARDA; PORTUGAL
LENGTH	— 4 073 m
DIAMETER	— 3.0 m
LINING THICKNESS	— 30 cm
OVERBURDEN BETWEEN PORTALS	MAX - MORE THAN 100 m BETWEEN KM 2.7 AND 3.2 MIN - ≈30 m
DATE OF THE PROJECT DESIGN	— AUG 1978
DESIGN PERIOD	— 1973 - 1978
OWNER	— DIRECÇÃO GERAL DOS RECURSOS E APROVEITAMENTOS HIDRAULICOS - LISBOA
DESIGNED BY	— COBA, SARL - LISBOA
CONSTRUCTION COST (project estimate)	— 160×10^6 ESC
COST OF THE DESIGN	— 3.2×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	— 30 MONTHS
STATUS	— TO BE CONSTRUCTED

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

The site investigations for the Sabugal - Meimoa tunnel were performed in 2 phases: phase 1 1973; phase 2 1974.

1973 - Basic geological survey by field mapping which led to the estabishment of a pro - gramme of exploration works.

1974 - During this second phase of site investi gations only boreholes were performed. A final geotechnical assessment report was produced together with the project desi- gn in August 1978.

This tunnel forms part of a scheme mainly for irrigation purposes which comprise four dams - Sabugal, Valhelhas, Meimoa and Capinha. The pro- ject design was issued at the same time as the one of Sabugal dam. In addition to linking Sabugal and Meimoa reservoirs, this tunnel will sup ply water for a power plant with aninstalled power of 17.5 Mw which will be increased to 35 Mw in a second phase.

Some dilatometer tests for assessing the rock mass deformability were proposed by the consultants but not accepted by the owner.

At the time when the boreholes were sunk the tunnel axis was not yet well defined and so, some of them were performed out of final ali- gnment, and due to lack of convenient topographic data two boreholes did not reach the depth to be excavated.

An estimated value of 10 000 MPa as the deformability modulus for the rock mass was suggested in the geotechnical report. This value was ba- sed on results obtained from identical formations at the site of Aguei- ra dam in 1974.

For the structural design of the tunnel's final support the following parameters were considered for the rock mass:

$$c = 4.4 \text{ MPa}$$
$$\phi = 33^{\circ}$$
$$E = 10\,000 \text{ MPa}$$
$$\nu = 0.2$$

A stronger support was designed for zones near the portals, but the lining thickness was 30 cm throughout.

ROCK MASS CONDITIONS

1. Rock Type

The rock mass to be excavated consists of schists and greywackes (see Fig. Appendix A26) which belong to the Portuguese Ante - Ordovician Complex. Those rocks are overlain by a thin layer of residual soils.

2. Structure

The general bedding attitude is around N (30^{o} - 40^{o}) W, 75^{o} NE , with local variations. The main direction of the tunnel (see Fig. Appendix A26) is near N 12^{o} E. Frequent veins of quartz cut across the rock mass in many directions and some crushed zones corresponding to some of them and to some faults of minor importance were detected by boreholes. Some dolerite dykes are present as well.

The rock mass exhibits strong deformation, with close and recumbent folds.

Three major joint sets were assessed:

DIRECTION (o)	INCLINATION (o)
N (10-30) E	65NW TO SUBVERTICAL
N (8-90) E	SUBVERTICAL
N (60-70) E	(40-50) S

3. State of weathering

The rock mass was classified as completely weathered down to depths of 2 to 6 m beneath the surface. Below that depth it changes to moderately to highly weathered down to a maximum depth of 25 m. At the depth of excavation the rock mass was classified as fresh to slightly weathered.

4. Water

The water table depth was measured in all the performed boreholes and it lies between 4 m and 13.5 m below the surface. Thus, the height of water over the tunnel line can reach about 100 m in some places. The excavation through some fractured zones in conection with the surface of the rock mass may bring water in under high pressures.

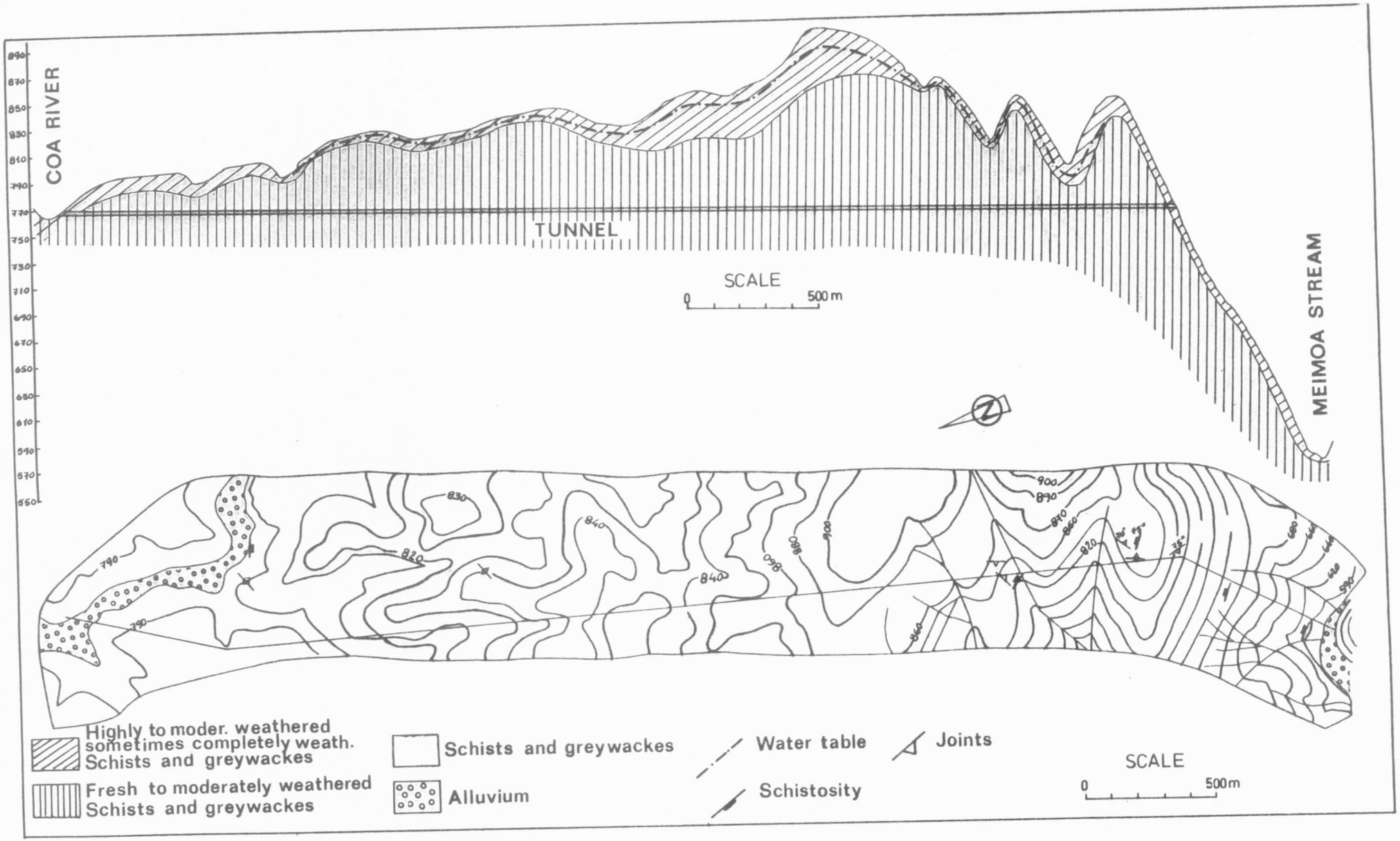


FIG. APPENDIX A26-SABUGAL-MEIMOA TUNNEL

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (1973)

1. Reconnaissance (Mid 1973)

PHASE 2 (1974)

1. Boreholes (May - Jul 1974)

DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{(0)}$	NO OF LUGEON TESTS
ST 2	27.36	60	2
ST 3	55.00	60	3
ST 4	70.00	60	1
ST 5	75.88	60	2
ST 6	75.11	75	3
ST 7	50.20	90	2
ST 8	25.00	60	-

Total no of boreholes	7
Total length	378.55 m
Average length	54.08 m
Total no of Lugeon tests	13
Cost	370 x 10^3 ESC (1974)

TUNNEL NAME — SADO - MORGAVEL

— GENERAL DATA —

LOCATION	— SINES; PORTUGAL	
LENGTH	—	13 159 m
DIAMETER	—	2.9m (internal)
LINING THICKNESS	—	(see inside)
OVERBURDEN BETWEEN PORTALS	MAX —	130 m
	MIN —	10 - 30 m
DATE OF THE PROJECT DESIGN	—	OCT 1974
DESIGN PERIOD	—	1973 - 1974
OWNER	—	GABINETE DA ÁREA DE SINES - LISBOA
DESIGNED BY	—	HIDROPROJECTO, SARL - LISBOA
CONSTRUCTION COST (project estimate)	—	330×10^6 ESC
COST OF THE DESIGN (approx.)	—	4×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	—	22 MONTHS
STATUS	—	UNDER CONSTRUCTION

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

This tunnel forms part of a water supply scheme for the Sines industrial complex. Part of the water necessary will be taken from the Morgavel dam already constructed near Sines, which is to be linked to the Sado river by an aqueduct formed in part by a channel and in part by this tunnel.

The site investigations for the Sado -Morgavel tunnel were undertaken in 3 phases: phase 1 1973; phase 2 1973 - 1974; phase 3 1974.

1973 - First investigations by means of a field mapping, after which the first programme of exploration works was proposed.

1973 - 1974 - A second phase of S.I. which consisted of geophysical exploration and boreholes. Based on the results of this phase a new phase was programmed.

1974 - Third phase of site investigations comprising integral sampling, "in - situ" and laboratory tests. A final geotechnical assessment report was issued in September 1974, also forming part of the main design report completed in October 1974.

When the S.I. began, the only topographic map available was one of 1 : 25 000 which was used to establish the specifications for the exploration works. When the topographic maps of 1: 10 000 and 1: 2 000 became available some differences emerged in the longitudinal profile, variations in the tunnel overburden reaching near 10 m in some zones. This meant that some of the boreholes that had been drilled had not reached tunnel level.

The "in - situ" tests proved very difficult to perform due to the poor condition of the rock mass. The initial programme established 24 tests inside 8 boreholes (3 tests in each). Afterwards the test loca-

tions were selected according to the possibility of their achievement and so the results reflected the characteristics of the better zones encountered. In the end, 26 tests were carried out in 6 boreholes, the number of tests in each varying from 1 to 9. The number of unsuccessful tests reached 80.

The ratio of laboratory deformability modulus to dilatometer modulus was assesses to be around 20.

For support design purposes, the geotechnical report considered two differents zones.

Z1 - Plio - Pleistocene formations, and completely to moderately weathered shales and greywackes.

Z2 - Fresh shales and greywackes.

In the design, four zones were considered for which the following mechanical characteristics were adopted:

F 1	Plio-Pleistocene materials	$\phi = 33^o$
F 2	Completely weathered Shale and Greywacke	$\phi = 27^o$ $\sigma_{uc} = 0.6$ MPa c = 0.1-0.15 MPa
F 3	Highly to moderately weathered Shale and Greywacke	$\sigma_{uc} = 8$ MPa
F 4	Fresh Shale and Greywacke	$\sigma_{uc} = 35$ MPa

The design considered a final support of reinforced concrete, about 38 cm thick, for the zones F1 and F2. For the other two zones several types of support were suggested which would be selected in accordance with the encountered situations.

The aspect relating to support design had to be studied again during the construction phase due to the poorer and varied conditions encountered.

ROCK MASS CONDITIONS

1. Rock Type

The rock mass to be excavated is a Carboniferous complex of mainly shales and greywackes. These formations are overlain by Plio - - Pleistocene sediments, sand and clay, which will be excavated in some zones, especially where the overburden is thin.

2. Structure

The rock mass presents strong deformation features of very tight to isoclinal folds, with orientation about NW - SE direction and the bedding is in general subvertical. The tunnel axis has been adapted to be close to orthogonal to the strike of the bedding.

Several interbedded quartz veins are present throughout the rock mass. They appear badly crushed, this revealing the magnitude of the forces wich acted on the rock mass.

Some faults, suggested from aerial photographic interpretation , were confirmed by the field works.

3. State of weathering

The rock mass (see Fig. Appendix A27) almost completely weathered near the surface, is followed at depth by a zone classified as moderately to highly weathered. At greater depths, the rock mass was classified as fresh.

4. Water

From the hydrogeological point of view 3 situations were considered. A first unit, of Pliocene clay - sand sediments, to which an equivalent permeability of between 10^{-3} and 10^{-5} cm.sec^{-1} was assigned. A second unit of Pliocene clay layers and completely weathered shales and greywackes which behave as impervious materials. A third unit of fresh rock mass. Important percolation takes place through fractures, faults and crushed zones.

The location of the water table is shown in Fig. Appendix A27.According to the geotechnical report it was expected that several

narrow crushed zones parallel to foliation, as well as several crushed quartz veins, would occur. These features and some fractures which are present would allow water inflow towards the excavation. The same was expected to happen due to other fractures not coincident with the foliation and with some existing faults to be crossed by the excavation.

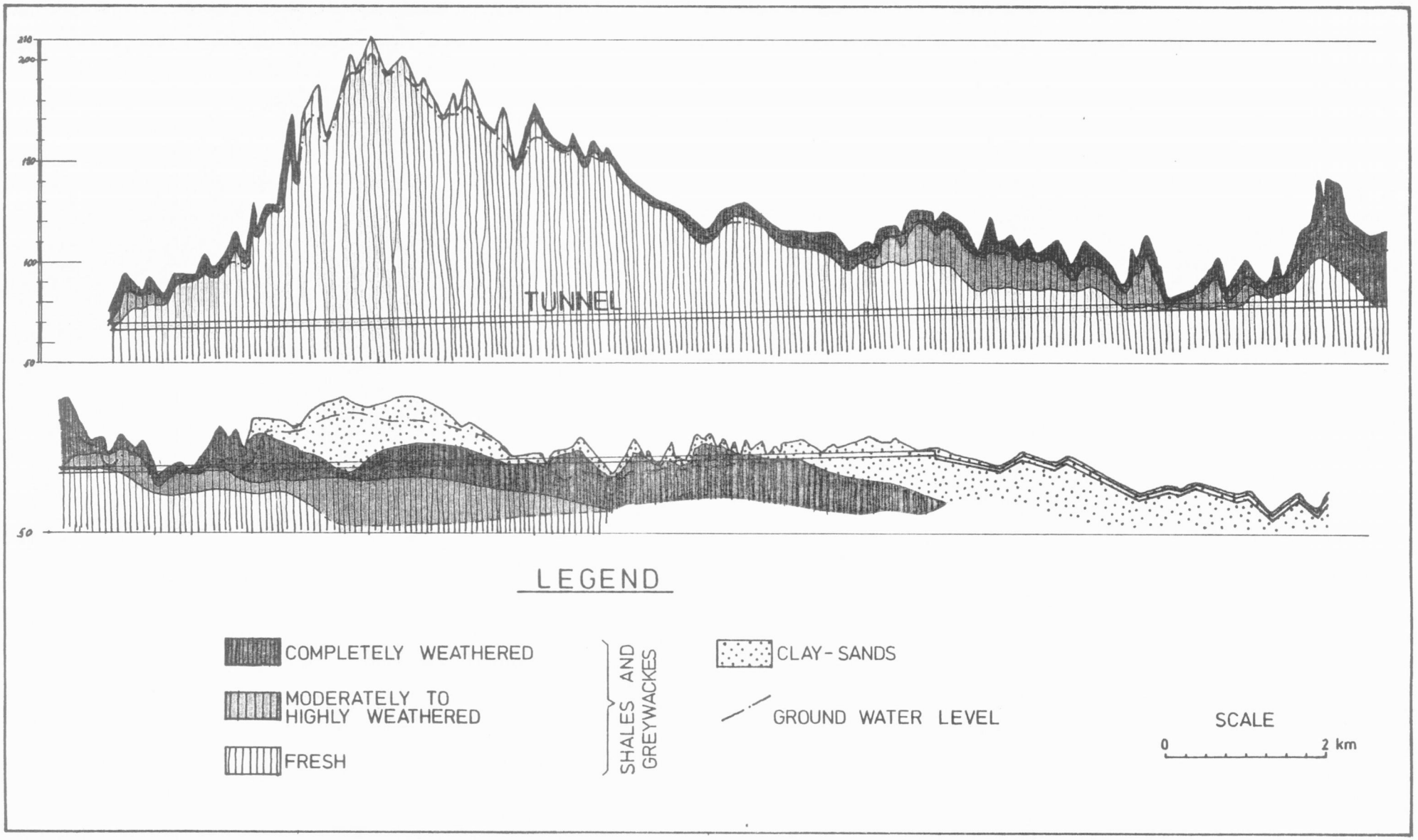


FIG. APPENDIX A27-SADO-MORGAVEL TUNNEL

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (January 1973)

1. Reconnaissance

Phase 2 (1973)

1. Geophysical exploration (mid 1973)

Seismic refraction profiles (direct and reverse shooting method)

DESIGNATION	LENGTH (m)	NO OF PROFILES
PS 1 TO PS 79	65 TO 300	79

Total no of profiles ≃ 9 700 m

Cost 210×10^3 ESC (1974)

2. Percussion drills (Sep 1973 - Apr 1974)

DESIGNATION	LENGTH (m)
SP 21	15.00
SP 22	15.16
SP 23	15.20

3. Boreholes (Same period as above)

DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ$_{(o)}$	NO OF LUGEON TESTS
SR 1	30.0	60	-
SR 2	30.0	90	-
SR 3	30.0	60	3
SR 4	60.0	90	3
SR 4A	80.0	60	3
SR 4B	75.0	90	3
SR 4C	105.0	90	3
SR 5	68.0	90	3
SR 6	43.0	90	3
SR 7	38.0	90	3
SR 8	36.0	90	3
SR 9	31.0	90	3
SR 10	25.0	90	3
SR 11	30.0	60	3
SR 12	25.0	90	3
SR 13	35.0	60	3
SR 14	50.0	90	3
SR 15	25.0	90	3
SR 16	30.0	90	3
SR 17	40.0	90	3
SR 17A	45.0	90	3
SR 18	59.0	90	2
SR 19	53.0	90	3
SR 20	35.0	90	3

Total no (boreholes + percussion drills) 27

Total length (boreholes and perc. drills) 1 123.36 m

Average length 41.6 m

Total no of Lugeon tests 65

Cost (boreholes + perc. drills) 1 000 x 10^3 ESC (1974)

PHASE 3 (1974)

1. Integral sampling

Although the performance of integral sampling along the last 12 - - 15 m of 8 boreholes had been programmed, the method was abandoned after 3 trials because good results were not achieved.

Total length of integral sampling - 108 m

Cost 210 x 10^3 ESC (1974)

2. "In - situ" tests (Feb - mid 1974)

2.1 Dilatometer tests

Device used	BHD - LNEC
No of boreholes tested	6
Horizons measured in each borehole	1 to 9
Directions measured in each horizon	almost 4
Maximum pressure applied	variable
Total no of horizons measured	26

The tests were performed between the depths of 6.5 m and 58.5 m.

Results:

(all measurements)

DILATOMETER MODUL (MPa)		
MAX	MIN	AVER
13 130	140	2 140

The more weathered zones of the rock mass gave values of between 200 and 1 000 MPa while the better zones exhibited dilatometer moduli of between 1 000 and 12 000 MPa. The coefficient of anisotropy within the measured directions reached the value of 7.

Cost of "in - situ" tests 250×10^3 ESC (1974)

3. Laboratory tests (same period as above)

3.1 Uniaxial tests

Total no of tests	20
Type of samples: cut from rock cores	
Diameter	52 - 71 mm
Aspect ratio	2 - 2.5

ASSESSMENT OF E

No of loading cycles	3 - 4
Max pressure applied	6 MPa (2 MPa in 4 samples)
Angle between schistosity and samples axis	50° - 90°

Results:

ROCK TYPE	E (MPa)			NO OF SAMPLES
	MAX	MIN	AVER	
FRESH TO SLIGHTLY WEATHERED SHALE	64 025	2 834	41 535	12
HIGHLY WEATHERED SHALE	3 460	546	1 747	6

ASSESSMENT OF σ_{uc}

Tests performed on the above samples, increasing pressure up to failure.

Results:

ROCK TYPE	σ_{uc} (MPa)			NO OF SAMPLES
	MAX	MIN	AVER	
FRESH TO SLIGHTLY WEATHERED SHALE	12	49.7	29.9	13
HIGHLY WEATHERED SHALE	1.8	13.7	7.4	7

3.2 Ultrasonic wave velocities

(same samples used for uniaxial tests)

Length of samples — see uniaxial tests

No of samples tested — 19

Measured velocities — V_p, V_s (in 5 samples)

Results:

ROCK TYPE	V_p (m/sec)			NO OF SAMPLES
	MAX	MIN	AVER	
FRESH TO SLIGHTLY WEATHERED SHALE	3 880	4 870	4 487	11
HIGHLY WEATHERED SHALE	1000.	1975.	1447	8

ROCK TYPE	V_s (m/sec)			NO OF SAMPLES
	MAX	MIN	AVER	
FRESH TO SLIGHTLY WEATHERED SHALE	2 560	3 212	2 886	2
HIGHLY WEATHERED SHALE	700	1060	880	3

Cost of laboratory tests 40×10^3 ESC (1974)

TUNNEL NAME — SANTA CLARA - MONTE DA ROCHA

— GENERAL DATA —

LOCATION	—	LINKS S.TA CLARA AND M.TE DA ROCHA RESERVOIRS; ALENTEJO; PORTUGAL
LENGTH	—	9 860 m
DIAMETER	—	2.7m (internal)
LINING THICKNESS	—	22.5 cm
OVERBURDEN BETWEEN PORTALS — MAX	—	100 m
OVERBURDEN BETWEEN PORTALS — MIN	—	20 - 30 m
DATE OF THE PROJECT DESIGN	—	DEC 1979
DESIGN PERIOD	—	1978 - 1979
OWNER	—	DIRECÇÃO GERAL DOS RECURSOS E APROVEITAMENTOS HIDRAULICOS - LISBOA
DESIGNED BY	—	HIDROPROJECTO, SARL - LISBOA
CONSTRUCTION COST (project estimate)	—	965×10^6 ESC
COST OF THE DESIGN	—	5×10^6 ESC
CONSTRUCTION PERIOD (project estimate)	—	36 MONTHS
STATUS	—	TO BE CONSTRUCTED

HISTORY OF SITE INVESTIGATIONS AND GENERAL REMARKS

For this tunnel, which links Santa Clara and Monte da Rocha Reservoirs as part of the water supply system for the industrial complex of Sines, three phases of site investigations were undertaken: phase 1 1977; phase 2 1978; phase 3 1979.

1977 - Basic geological survey by field mapping and geophysical exploration . An engineering geological report was produced in April 1977.

1978 - Second phase of site investigations which consisted of geophysical exploration, boreholes, "in - situ" and laboratory tests. A new engineering geological report was issued in November 1978.

1979 - This third phase consisted solely of some geophysical exploration works, finished in December 1979. The report concerning these works was issued in January 1980.

The first engineering geological report proposed a programme for the subsequent phase of site investigations. On this, minor modifications were made by the owner in addition to thewithdrawal of some geophysical exploration works.

In December 1978, after the second engineering geological report had been issued,the owner agreed to perform a further small site investigation phase in order to clarify some aspects which arose during the preceding works, mainly related to the presence of a suspected fault with similar orientation to the tunnel. This was refered to as a third programme of investigation.

The tunnel was designed to work under an internal pressure of 0.2 MPa. Two different deformability moduli were attributed to the rock mass , based on the performed tests:

a) 3 000 MPa for the moderately weathered zones and

b) 7 000 MPa for the fresh to slightly weathered zones.

The lining thickness designed was 22.5 cm except in the zones near the portals where 27 cm is to be used. Two types of reinforcement were established, according to the assumed deformability modulus of the rock mass, the more resistant to be used in the lower deformability modulus zones.

ROCK MASS CONDITIONS

1. Rock Type

The excavation will deal with a Carboniferous complex of interbedded shales and greywackes (see Fig. Appendix A28), the predominance of each rock type varying along the tunnel axis.

2. Structure

The rock mass presents intense tectonic deformation which is evidenced by the presence of successive isoclinal folds, generally very tight with axis orientated around NNW - SSE. Due to folding, bedding direction shows some variations locally.

The assessment of jointing established four main sets. These are shown below in order of their decreasing occurrence.

DIRECTION (o)	INCLINATION (o)
N 70 W TO N 85 W	70 - 90 NE
N 55 E TO N 80 E	60 - 90 SE
N 20 E TO N 35 E	70 - 85 SE
N 10 E TO N 30 E	60 - 85 NW

The fracturing intercept is F4 (6 - 20 cm), F 4 -5 near the surface and F3 (20 - 60 cm) at deeper zones, and at depths concerned with the excavation the joints show no infilling with smooth and fresh walls.

Some faults were detected or confirmed during site investigations along the entire length of the tunnel, their main orientation being a) faults with the same direction as the bedding and similar assumed inclination; b) faults normal to bedding; c) faults subparallel to the tunnel axis.

Several crushed quartz veins occur crossing the rock mass, mainly about 10 cm thick, with a maximum of 40 cm measured in the rock cores. They can occur filling either major discontinuities or branched fissures.

3. State of weathering

The rock mass is highly to completely weathered W (4- 5) from the surface to a maximum depth of 20 m. Beneath this depth it is moderately weathered (W 3). The excavation is to be concerned with mainly fresh to slightly weathered rock mass (W 1 - 2) except in two zones where W 3 is present as shown by boreholes (see Fig. Appendix A28). In the majority of the boreholes the percentage of recovery was high, frequently 100%.

4. Water

To assist in the assessment of "permeability" conditions, Lugeon tests were performed using pressure steps of 0.5; 1; 0.5 MPa and 0.25; 0.5; 1; 0.5; 0.25 MPa for depths greater and lower than 30 m respectively.

The results gave absorptions greater than 20 l/min.m for the rock mass included in zone I (see Fig. Appendix A28); between 10 and 20 l/min.m for the zone II and 2 l/min.m or even lower than 1 l / /min.m for the zone III.

The geotechnical report refers to the possibility of the tunnel crossing some structural features as crushed quartz veins, not detected during the investigations, which can constitute important aquifers.

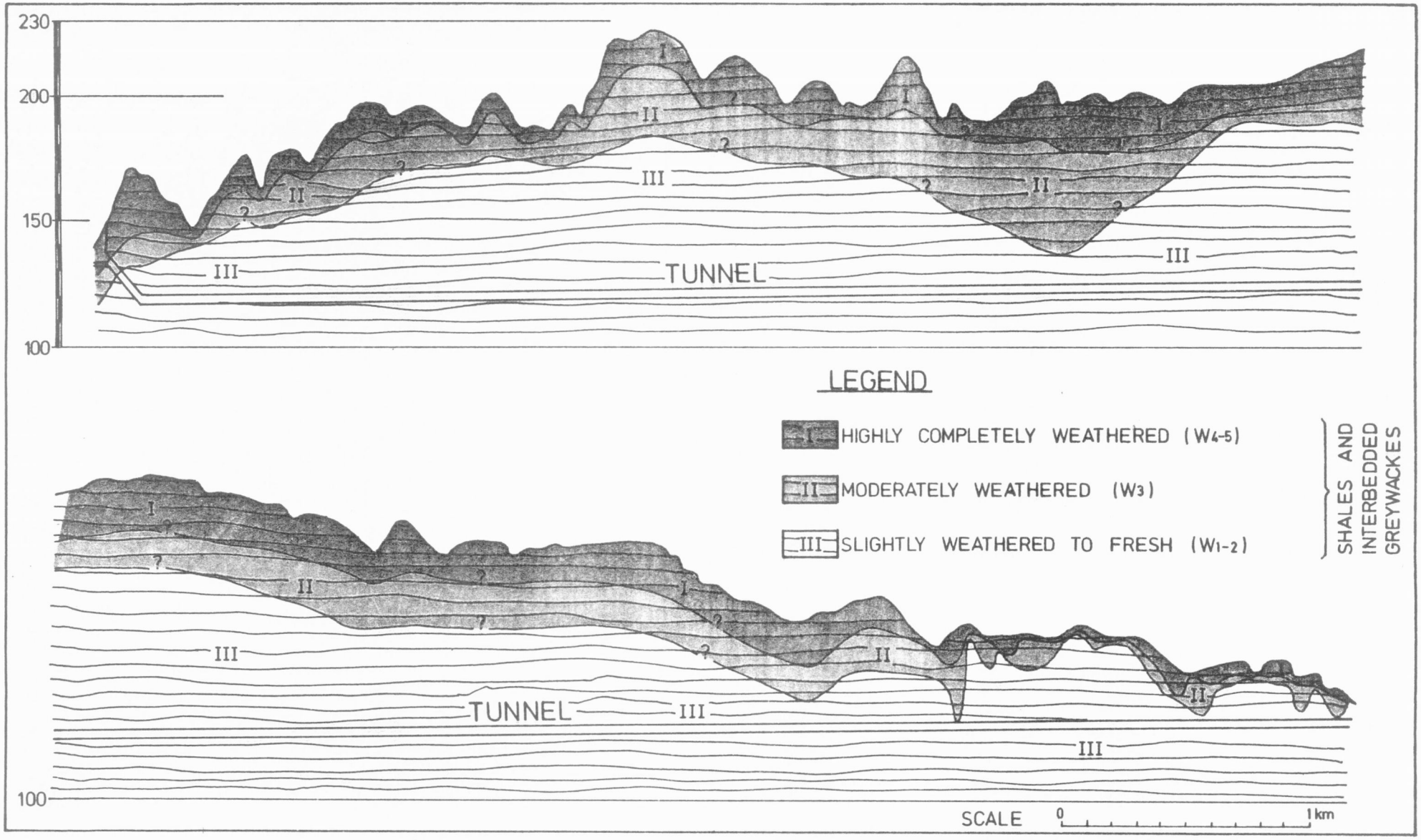


FIG. APPENDIX A28-STA CLARA-MTE DA ROCHA TUNNEL

SITE INVESTIGATIONS - DETAILS AND COSTS OF THE PERFORMED WORKS

PHASE 1 (Mar - Apr 1977)

1. Reconnaissance

2. Geophysical exploration (Mar 1977)

Seismic refraction profiles (direct, reverse and central shot method . Spacing of geophones - 6 m).

DESIGNATION	LENGTH (m)	NO. OF PROFILES
PS1 TO PS15	125	15

Total length of profiles 1 875 m

Costs of these profiles 70×10^3 ESC (1977)

PHASE 2 (1978)

1. Geophysical exploration (Mar 1978)

1.1 Geoelectrical soundings

57 geoelectrical soundings, 30 m apart, along a 1 700 m long tunnel section,

DESIGNATION	MAX $\overline{AB}$ (m)	$\overline{MN}$ (m)	NO. OF SOUNDINGS
SEO TO SE56	200	5	57

1.2 Apparent resistivity profile

One profile was performed along the above referred section, for three different investigation depths, hence using three different $\overline{AB}$'s.

DESIGNATION	$\overline{AB}$ (m)	$\overline{MN}$ (m)	NO. OF PROFILES
-	25,55,105	5	1

Total length of resistivity profiles 5 100 m

Cost of phase 2 geoph. exploration 180 x 10^3 ESC (1978)

2. Boreholes (Apr - Aug 1978)

DESIGNATION	LENGTH (m)	INCLINATION TO THE HORIZ (°)	NO OF LUGEON TESTS
S 1	98.0	83	16
S 2	64.0	20	4
S 3	35.0	20	1
S 4	80.0	20	3
S 5	85.7	20	3
S 6	85.0	20	3
S 8	90.0	20	2
S 9	88.4	20	6
S 10	50.1	20	3
S 11	50.0	10	1
S 11A	19.7	10	-
S 12	45.0	20	2
S 13	28.0	20	2
S 14	20.0	90	-
S 15	20.2	90	-

Total no of boreholes 15

Total length 859.1 m

Average length 57.3 m

Total no of Lugeon tests 46

Cost 2 270 x 10^3 ESC (1978)

3. "In - situ" tests (Jun - Aug 1978)

3.1 Dilatometer tests

Device used BHD - LNEC

No of boreholes tested 4

Horizons measured in each borehole 4 - 7

Directions measured in each horizon 4

No of loading cycles 3

Maximum pressure applied almost 8 MPa but 5 - 6 MPa were used in 5 cases.

Total no of horizons measured 20

The tests were perfomed between the depths of 21.10 and 87.25 m.

Results

A - Average from each borehole

BOREHOLE	DILATOMETER MODULUS E (MPa)	NO OF HORIZONS MEASURED
S2	21 000	7
S5	21 750	4
S8	29 800	5
S13	7 750	4

B - Average of all measurements

ROCK TYPE	DILATOMETER MODULUS (MPa)			NO OF HORIZONS MEASURED
	MAX	MIN	AVER	
GREYWACKY SHALE (W3)	—	—	5.000	1
SHALE / GREYWACKE (W1-2)	43 000	7 000	21 500	19

3.2 State of stress "in - situ"

2 tests were performed using the strain tensor gauge (LNEC) at depths of 71.55 m and 67.55 m. A third attempted test failed.

Cost of "in - situ" tests 700×10^3 ESC (1978)

4. Laboratory test (same period as above)

4.1 Uniaxial tests

Total no of tests 12

Type of samples: cut from rock cores

Diameter 54 mm and 86 mm (2 samples)

Aspect ratio 2 - 2.5

ASSESSMENT OF E

No of loading cycles 2 - 3

Max pressure applied almost 8 MPa (20 MPa in one case and 6 MPa in two other cases)

Results:

ROCK TYPE	E (MPa)			NO OF TESTS
	MAX	MIN	AVER	
GREYWACKE	52 000	7 000	26 000	5
SHALES, SOMETIMES WITH GREYWACKE	28 000	5 000	18 000	6

ASSESSMENT OF σ_{uc}

Tests performed in the above samples, increasing pressure up to failure.

Results:

ROCK TYPE	σ_{uc} (MPa)			NO OF TESTS
	MAX	MIN	AVER	
GREYWACKE	83	13	50	5
SHALES, SOMETIMES WITH GREYWACKE	75	14	37	6

4.2 Ultrasonic wave velocities

(same samples used for uniaxial tests)

Length of samples 120 - 150 mm

No of samples tested 5

Measured velocities V_p and V_s

Results:

ROCK TYPE	V_p (m/sec)			V_s (m/sec)			NO OF TESTS
	MAX	MIN	AVER	MAX	MIN	AVER	
GREYWACKE	5 280	3 070	4 175	3 080	2 330	2 705	2
MINLY SHALE	3 325	2 040	2 782	1 740	1 670	1 705	3*

* V_s measured only in 2 samples

4.3 Other tests

4.3.1 Porosity

No of samples tested 6

4.3.2 Bulk density

No of samples tested 6

Cost of laboratory tests 35×10^3 ESC (1978)

PHASE 3 (1979)

1. Geophysical exploration (late 1979)

10 apparent resistivity profiles with an average length of 156 m were performed. These profiles were made using $\overline{AB}$'s of 185 m and 55 m in order to investigate two depths. The $\overline{MN}$ used was 5 m. An apparent resistivity map was produced using the results of the profiles with $\overline{AB}$ equals to 185 m.

Cost of phase 3 geoph. exploration 55×10^3 ESC (1979)



FIG. APPENDIX A 29

SEISMIC INTENSITY

MAP OF PORTUGAL (1901 - 1972)

INTERNATIONAL SCALE

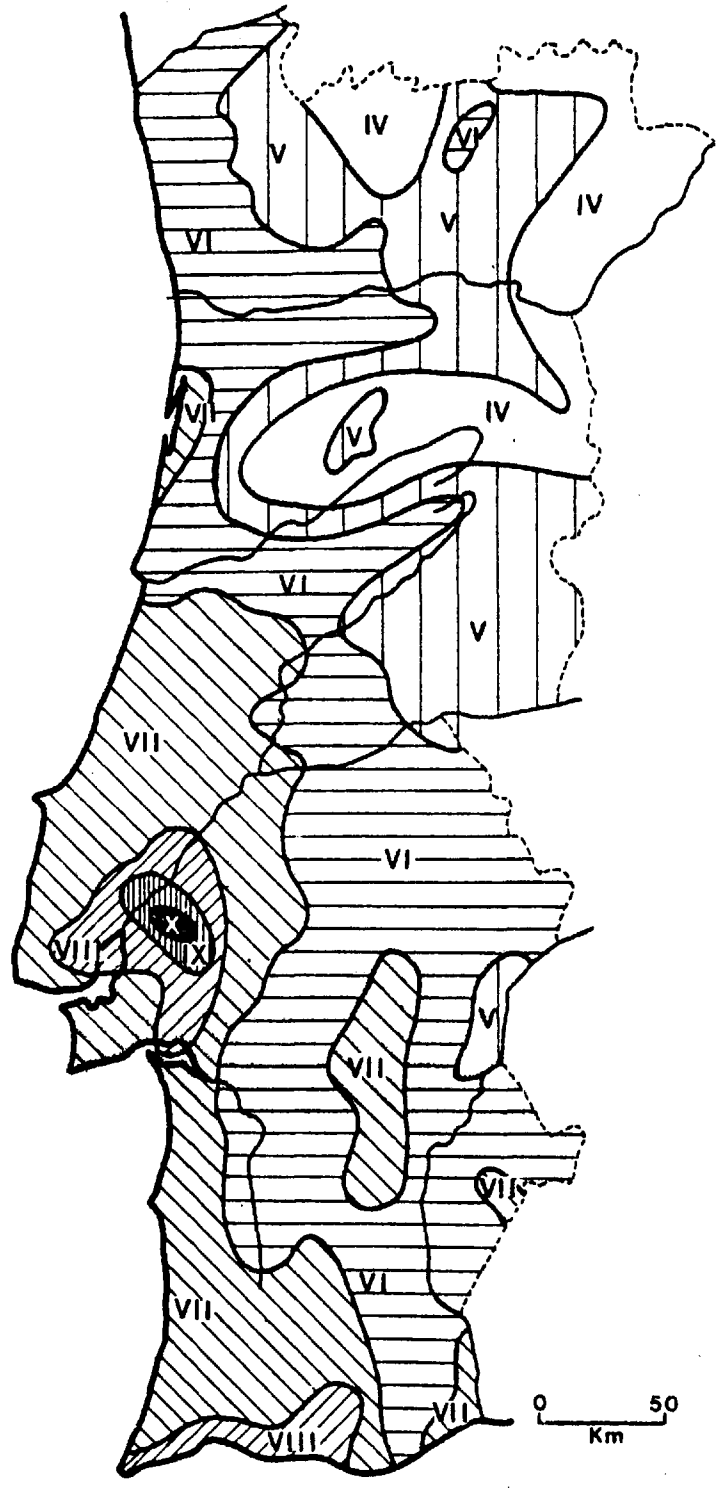


FIG. APPENDIX A30

AVERAGE ANNUAL PRECIPITATION IN THE PORTUGUESE HYDROGRAPHIC BASINS BETWEEN 1954-55/1977-78

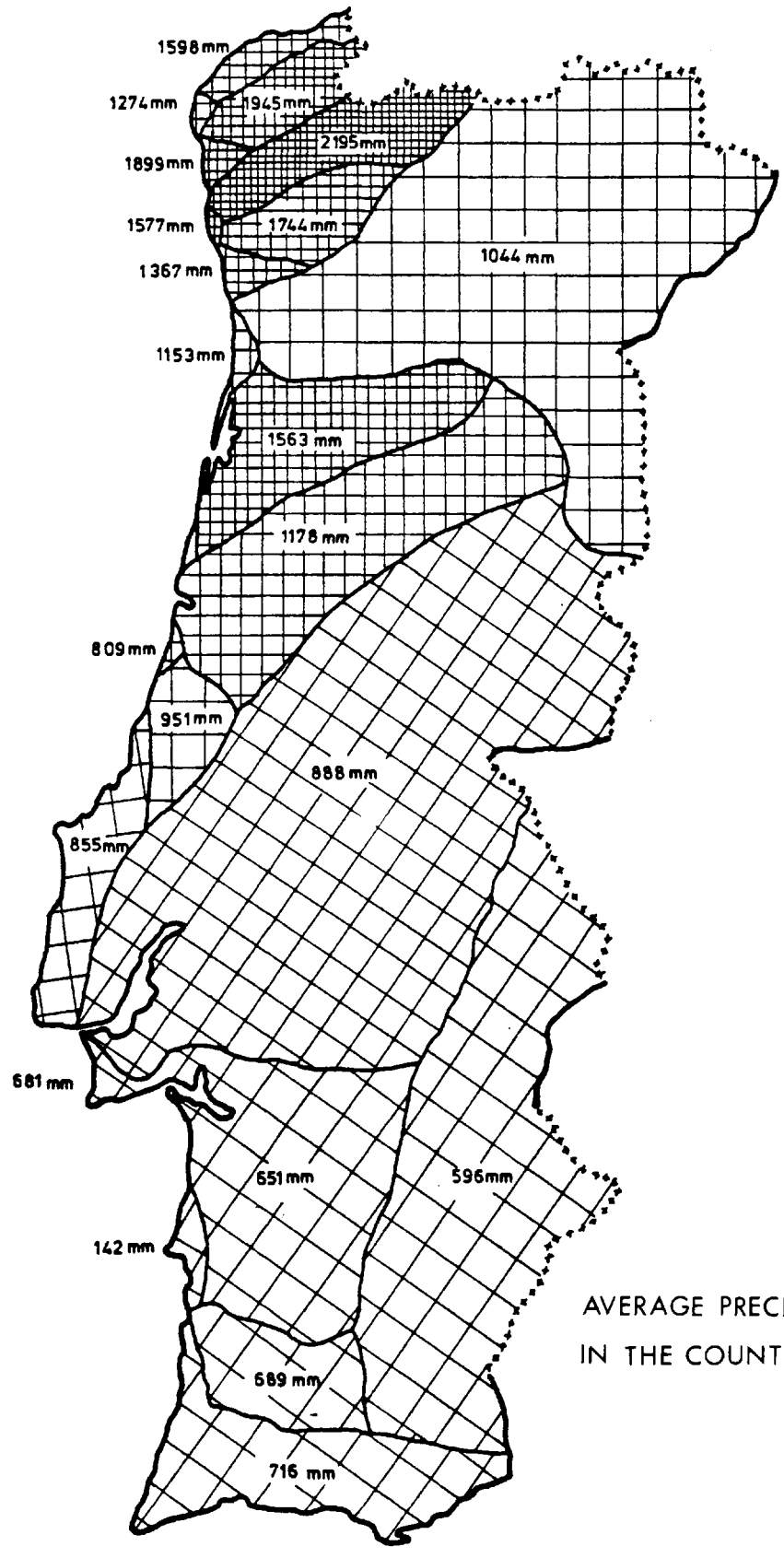


FIG. APPENDIX A31

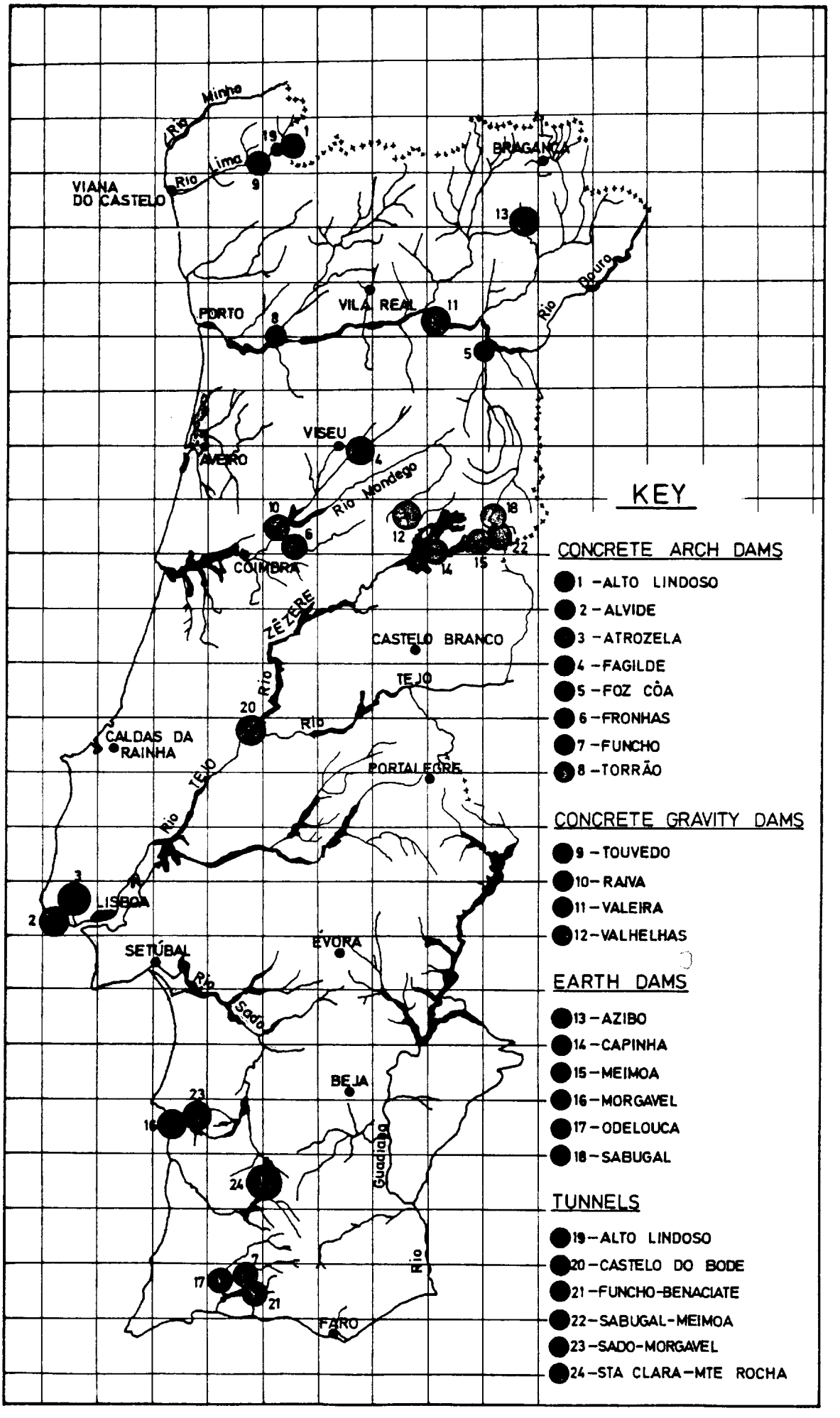


FIG. APPENDIX A32

APPENDIX B

THE BENEFITS

ALTO LINDOSO DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	3	6	8	10
Phase 1 - Reconnaissance and gathering existing information	10 (9.0)	35 (31.5)	10 (8.5)	15 (12.8)
Phase 2 - 9 boreholes (427.85 m)	20 (18.0)	5 (4.5)	20 (17.0)	10 (8.5)
6 galleries (294 m)	25 (22.5)	20 (18.0)	25 (21.3)	30 (25.5)
Phase 3 - 6 boreholes (250.25 m)	15 (13.5)	5 (4.5)	15 (12.8)	5 (4.3)
Phase 4 - 5 boreholes (239.7 m)	10 (9.0)	5 (4.5)	10 (8.5)	5 (4.3)
12 trenches (145 m)	5 (4.5)	15 (13.5)	5 (4.3)	25 (21.3)
Laboratory tests: uniaxial tests (36 samples) joint shear tests (33 samples) rock shear tests (12 samples)	15 (13.5)	15 (13.5)	15 (12.8)	10 (8.5)
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	90	90	85	85

ALVIDE DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	8	9		
Phase 1 - Reconnaissance and gathering existing information	10 (9.5)	5 (4.8)		
Phase 2 - 1 trench (95 m)	2 (1.9)	2 (1.9)		
6 boreholes (200.5 m)	50 (47.5)	43 (40.9)		
Phase 3 - 1 borehole (20 m)	3 (2.9)	2 (1.9)		
1 gallery (40 m)	17 (16.2)	18 (17.1)		
Phase 4 - 3 boreholes (80.2 m)	5 (4.8)	5 (4.8)		
Geophysical exploration:				
7 seismic refraction profiles				
2 seismic fans	5 (4.8)	10 (9.5)		
"In-situ" tests:				
dilatometer tests (14 tests)	5 (4.8)	10 (9.5)		
Laboratory tests:				
uniaxial tests (18 samples)				
joint shear tests (15 samples)	3 (2.9)	5 (4.8)		

(continued)

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	8	9		
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	95	95		

ATROZELA DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	8	9		
Phase 1 - 4 boreholes (185 m)	35 (33.3)	35 (33.3)		
Phase 2 - Reconnaissance and gathering existing information	20 (19.0)	15 (14.3)		
Phase 3 - 5 boreholes (227.1 m) + 8 percussion drills (345 m)	42 (39.9)	45 (42.8)		
2 trenches (74 m)	3 (2.9)	5 (4.8)		
% of knowledge at completion of site investigaions in relation to that considered desirable (100%)	95	95		

FAGILDE DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	6			
- works performed before:				
1 trench (5 m)	5 (4.0)			
Phase 1 - Reconnaissance and gathering existing information	25 (20.0)			
Phase 2 - 4 boreholes (108.9 m)	70 (56.0)			
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	80			

FOZ-COA DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	3	6	8	10
Phase 1 - Reconnaissance an gathering existing information	5 (4.8)	15 (13.5)	15 (4.8)	15 (14.3)
Phase 2 - 6 trenches (1 189 m)	5 (4.8)	10 (9.0)	5 (4.8)	5 (4.8)
Phase 3 - Geophysical exploartion:				
11 apparent resistivity profiles and apparent resistivity map				
33 seismic refraction profiles	5 (4.8)	10 (9.0)	10 (9.5)	5 (4.8)
9 trenches (1 700 m)	10 (9.5)	15 (13.5)	10 (9.5)	10 (9.5)
10 galleries (601 m)	15 (14.3)	10 (9.0)	15 (14.3)	15 (14.3)
51 boreholes (2 839.9 m) + 11 boreholes (547.6 m)	25 (23.8)	15 (13.5)	25 (23.8)	25 (23.8)
Geophysical exploration: cross-hole method	10 (9.5)	10 (9.0)	10 (9.5)	10 (9.5)
"In-situ" tests: flat jack tests (24 tests) joint shear tests (3 blocks)	15 (14.3)	10 (9.0)	15 (14.3)	10 (9.5)

(continued)

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	3	6	8	10
Laboratory tests:				
uniaxial tests (48 samples)				
rock shear tests (25 samples)				
joint shear tests (41 samples)	10	5	5	5
	(9.5)	(4.5)	(4.8)	(4.8)
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	95	90	95	95

FRONHAS DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	3	6	8	10
- works performed before:				
14 boreholes (281.6 m)				
4 galleries (80 m)	15 (12.8)	15 (12.8)	15 (13.5)	15 (3.8)
Phase 1 - Reconnaissance and gathering existing information	5 (4.3)	25 (21.3)	5 (4.5)	10 (7.5)
Phase 2 - Geophysical exploration:				
11 apparent resistivity profiles and apparent resistivity map				
12 seismic refraction profiles				
4 seismic fans	10 (8.5)	10 (8.5)	10 (9.0)	10 (7.5)
9 trenches (980 m)	10 (8.5)	10 (8.5)	10 (9.0)	15 (11.3)
8 galleries (209 m)	20 (17.0)	10 (8.5)	15 (13.5)	20 (15.0)
18 boreholes (807.7 m)	20 (17.0)	10 (8.5)	25 (22.5)	25 (18.8)

(continued)

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	3	6	8	10
"In-situ" tests:				
dilatometer tests (41 tests)				
flat jack tests (12 tests)				
joint shear tests (4 blocks)	15	15	15	10
	(12.8)	(12.8)	(13.5)	(7.5)
Laboratory tests:				
uniaxial tests (48 samples)				
joint shear tests (33 samples)	5	5	5	5
	(4.3)	(4.3)	(4.5)	(3.8)
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	85	85	90	75

FUNCHO DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	5	8		
Phase 1 - Reconnaissance and gathering existing information	5 (4.5)	7 (6.7)		
Phase 2 - 6 trenches (283 m)	10 (9.0)	5 (4.8)		
9 boreholes (436.6 m)	20 (18.0)	25 (23.8)		
Phase 3 - 1 gallery (30 m)	5 (4.5)	8 (7.6)		
1 trench (53 m)	2 (1.8)	2 (1.9)		
Phase 4 - 3 galleries (104.7 m)	25 (22.5)	25 (23.8)		
"In-situ" tests: flat jack tests (5 tests) dilatometer tests (24 tests)	20 (18.0)	20 (19.0)		
Laboratory tests:				

(continued)

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	5	8		
uniaxial tests (24 samples)				
rock shear tests (11 samples)				
joint shear tests (18 samples)	13	8		
	(11.7)	(7.6)		
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	90	95		

TORRÃO DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	3	6	8	10
Phase 1 - Reconnaissance and gathering existing information	5 (4.5)	15 (13.5)	10 (8.0)	18 (13.5)
Phase 2 - 10 trenches (1 673 m)	20 (18.0)	25 (22.5)	5 (4.0)	18 (13.5)
19 galleries (401.5 m)	20 (18.0)	15 (13.5)	20 (16.0)	22 (16.5)
Geophysical exploration: 10 apparent resistivity profiles at two depths and two apparent resistivity maps				
4 geoelectrical soundings	10 (9.0)	5 (4.5)	15 (12.0)	6 (4.5)
19 boreholes (792.1 m)	15 (13.5)	10 (9.0)	25 (20.0)	12 (9.0)
"In-situ" tests: dilatometer tests (28 tests) rock shear tests (9 blocks) joint shear tests (3 blocks)	15 (13.5)	15 (13.5)	15 (12.0)	12 (9.0)

(continued)

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	3	6	8	10
Laboratory tests:				
uniaxial tests (50 samples)				
rock shear tests (13 samples)				
joint shear tests (6 samples)	10	5	5	6
	(9.0)	(4.5)	(4.0)	(4.5)
Geophysical exploration:				
4 seismic refraction profiles				
17 seismic fans	5	10	5	6
	(4.5)	(9.0)	(4.0)	(4.5)
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	90	90	80	75

JAMBA-IA-MINA DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	8	9		
Phase 1 - Reconnaissance and gathering existing information	15 (12.0)	15 (12.8)		
Phase 2 - 17 boreholes (639.5 m)	65 (52.0)	60 (51.0)		
Geophysical exploration: 6 apparent resistivity profiles and apparent resistivity map 6 geoelectrical soundings 79 seismic refraction profiles	20 (16.0)	25 (21.3)		
% of knowledge at completion of site investigations in the relation to that considered desirable (100%)	80	85		

JAMBA-IA-OMA DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	5	8		
Phase 1 - Reconnaissance and gathering existing information	10 (8.0)	10 (8.5)		
Phase 2 - 15 boreholes (600.8 m) + 4 boreholes (108.4 m)	55 (44.0)	50 (42.5)		
Phase 3 - Geophysical exploration: 5 seismic refraction profiles + 8 seismic refraction profiles	20 (16.0)	20 (17.0)		
2 boreholes (42.55 m)	15 (12.0)	20 (17.0)		
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	80	85		

TOUVEDO DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	3	4	6	10
Phase 1 - Reconnaissance and gathering existing information	25 (20.0)	30 (24.0)	50 (42.5)	20 (15.0)
Phase 2 - 8 trenches (543 m)	20 (16.0)	20 (16.0)	25 (21.3)	30 (22.5)
23 boreholes (1 032 m)	55 (44.0)	50 (40.0)	25 (21.3)	50 (37.5)
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	80	80	85	75

RAIVA DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	6	7	8	
Phase 1 - Reconnaissance and gathering existing information	35 (29.8)	5 (2.5)	15 (11.3)	
Phase 2 - 62 short (.5 m to 3.8 m long) percussion drills	25 (21.3)	5 (2.5)	5 (3.8)	
Phase 3 - 2 trenches (110 m)	10 (8.5)	20 (10.0)	10 (7.5)	
Special study of jointing	5 (4.3)	5 (2.5)	5 (3.8)	
Geophysical exploration: 3 seismic refraction profiles 11 apparent resistivity profiles	15 (12.8)	15 (7.5)	15 (11.3)	
Phase 4 - 19 boreholes (562.26 m)	10 (8.5)	50 (25.0)	50 (37.5)	
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	85	50	75	

VALEIRA DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	6	7	8	
- Works performed before:				
2 boreholes (22.98 m)	40 (38.0)	5 (4.0)	5 (3.5)	
Phase 1 - Reconnaissance and gathering existing information	5 (4.8)	5 (4.0)	10 (7.0)	
Phase 2 - 4 boreholes (204.76 m)	10 (9.5)	10 (8.0)	10 (7.0)	
Phase 3 - 4 galleries (149 m)	15 (14.3)	20 (16.0)	25 (17.5)	
9 boreholes (440.5 m) + 18 boreholes (833.2 m)	20 (19.0)	50 (40.0)	40 (28.0)	
2 trenches (250 m)	10 (9.5)	10 (8.0)	10 (7.0)	
% of knowledge at completion of site investigations in relation to that considered desirable	95	80	70	

VALHELHAS DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	3	6	8	10
Phase 1 - Reconnaissance and gathering existing information	10 (8.5)	35 (29.8)	10 (8.5)	15 (12.8)
Phase 2 - 5 trenches (510 m)	10 (8.5)	10 (8.5)	5 (4.3)	15 (12.8)
Special study of jointing	5 (4.3)	5 (4.3)	5 (4.3)	5 (4.3)
Geophysical exploration: 25 seismic refraction profiles 12 apparent resistivity profiles and apparent resistivity map 10 geoelectrical soundings	20 (17.0)	- (-)	20 (17.0)	20 (17.0)
39 boreholes (1 202.81 m)	40 (34.0)	30 (25.5)	40 (34.0)	35 (29.8)
Integral sampling (45 m)	5 (4.3)	5 (4.3)	10 (8.5)	5 (4.3)
Laboratory tests: uniaxial tests (18 samples)				

(continued)

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	3	6	8	10
joint shear tests (18 samples)	10 (8.5)	15 (12.8)	10 (8.5)	5 (4.3)
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	85	85	85	85

AZIBO DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	5	8	9	
Phase 1 - Reconnaissance and gathering existing information	10 (7.5)	20 (16.0)	15 (12.8)	
Phase 2 - 1 trench (370 m)	15 (11.3)	10 (8.0)	10 (8.5)	
16 boreholes (606.5 m) + 2 boreholes (40 m)	40 (30.0)	45 (36.0)	30 (25.5)	
Phase 3 - 8 boreholes (115.7 m)	20 (15.0)	10 (8.0)	25 (21.3)	
1 trench (430 m)	10 (7.5)	5 (4.0)	15 (12.8)	
Phase 4 - 2 trenches (220 m)	5 (3.8)	10 (8.0)	5 (4.3)	
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	75	80	85	

CAPINHA DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	8			
Phase 1 - Reconnaissance and gathering existing information	10 (8.5)			
Phase 2 - 1 trench (75 m)	30 (25.5)			
1 borehole (45 m)	10 (8.5)			
Phase 3 - 5 boreholes (124.9 m)	30 (25.5)			
1 trench (60 m)	10 (8.5)			
2 boreholes (24.95 m)	10 (8.5)			
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	85			

MEIMOA DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	8	9		
Phase 1 - Reconnaissance and gathering existing information	10 (7.5)	15 (11.3)		
Phase 2 - 7 trenches (491 m)	15 (11.3)	10 (7.5)		
20 boreholes (502.5 m)	40 (30.0)	35 (26.3)		
Phase 3 - 7 trenches (644 m)	10 (7.5)	8 (6.0)		
3 auger borings (23.49 m)	5 (3.8)	2 (1.5)		
12 boreholes (487.11 m)	20 (15.0)	30 (22.5)		
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	75	75		

MORGAVEL DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	2	8		
Phase 1 - Reconnaissance and gathering existing information	33 (31.4)	26 (24.7)		
Phase 2 - 13 boreholes (417.47 m) + 10 percussion drills (120.46 m)	50 (47.5)	54 (51.3)		
3 boreholes (42. 39 m) + 3 percussion drills (23.84 m)	11 (10.5)	13 (12.4)		
Phase 3 - Geophysical exploration: 5 seismic refraction profiles	6 (5.7)	7 (6.7)		
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	95	95		

ODELOUCA DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	5	8		
Phase 1 - Reconnaissance and gathering existing information	5 (4.5)	10 (9.0)		
Phase 2 - 14 trenches (302 m)	8 (7.2)	10 (9.0)		
18 boreholes (819.7 m)	25 (22.5)	25 (22.5)		
Geophysical exploration: 19 seismic refraction profiles 10 geoelectrical soundings 1 apparent resistivity profile	5 (4.5)	5 (4.5)		
Phase 3 - 2 galleries (100 m)	28 (25.2)	25 (22.5)		
Geophysical exploration: 6 apparent resistivity profiles and apparent resistivity map 10 seismic refrection profiles	10 (9.0)	10 (9.0)		

(continued)

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	5	8		
2 shafts (12 m and 14 m deep)	7 (6.3)	5 (4.5)		
Phase 4 - 12 boreholes (357.37 m)	10 (9.0)	10 (9.0)		
1 trench (240 m)	2 (1.8)	- -		
% of knowledge at completion of site investigations in relation to that considered desirable (100 %)	90	90		

SABUGAL DAM

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	8	9		
Phase 1 - Reconnaissance and gathering existing information	10 (8.5)	10 (9.0)		
Phase 2 - 1 trench (395 m)	10 (8.5)	10 (9.0)		
10 boreholes (544.5 m)	30 (25.5)	40 (36.0)		
Phase 3 - 3 boreholes (208 m)	10 (8.5)	10 (9.0)		
5 boreholes (116.8 m)	15 (12.8)	14 (12.6)		
Geophysical exploration:				
6 seismic refraction profiles				
20 seismic refraction profiles				
1 apparent resistivity profile	15 (12.8)	11 (9.9)		
Phase 4 - Gephysical exploration:				
11 seismic refraction profiles				

(continued)

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	8	9		
1 apparent resistivity profile				
2 geoelectrical soundings	10	5		
	(8.5)	(4.5)		
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	85	90		

ALTO LINDOSO TUNNEL

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	1	6	8	10
Phase 1 - Reconnaissance and gathering existing information	20 (17.0)	35 (31.5)	20 (16.0)	20 (12.0)
Phase 2 - 12 boreholes (583.6 m)	30 (25.5)	15 (13.5)	50 (40.0)	40 (24.0)
28 trenches (340 m)	10 (8.5)	25 (22.5)	5 (4.0)	20 (12.0)
"In-situ" tests: dilatometer tests (30 tests) state of stress (4 tests)	20 (17.0)	15 (13.5)	15 (12.0)	10 (6.0)
Laboratory tests: uniaxial tests (58 samples) ultrasonic velocity (38 samples) point load tests (33 samples) Schmidt hammer (38 samples) porosity (18 samples) bulk density (18 samples)	20 (17.0)	10 (9.0)	10 (8.0)	10 (6.0)

(continued)

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	1	6	8	10
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	85	90	80	60

CASTELO DO BODE

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	5	8	9	
Phase 1 - Reconnaissance and gathering existing information	5 (4.8)	10 (9.0)	5 (4.5)	
Phase 2 - Geophysical exploration:				
9 seismic refraction profiles 21 seismic refraction profiles	6 (5.7)	10 (9.0)	10 (9.0)	
13 boreholes	30 (28.5)	37 (33.3)	30 (27.0)	
"In-situ" tests: dilatometer tests (18 tests) state of stress (4 tests)	20 (19.0)	10 (9.0)	10 (9.0)	
Laboratory tests: uniaxial tests (17 samples) ultrasonic velocity (42 samples) point load tests (20 samples) Schmidt hammer (21 samples)	10 (9.5)	8 (7.2)	10 (9.0)	
Phase 3 - Special study of jointing				

(continued)

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	5	8	9	
(on rock cores)	15 (14.3)	10 (9.0)	15 (13.5)	
Geophysical exploration: 7 apparent resistivity profiles at two depths	4 (3.8)	5 (4.5)	5 (4.5)	
Laboratory tests: uniaxial tests (17 samples) joint shear tests (17 samples)	10 (9.5)	10 (9.0)	15 (13.5)	
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	95	90	90	

FUNCHO-BENACIATE TUNNEL

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	5	8		
Phase 1 - Reconnaissance and gathering existing information	10 (9.0)	10 (8.5)		
Phase 2 - 15 boreholes (583.4 m)	45 (40.5)	50 (42.5)		
Geophysical exploration: 8 seismic refraction profiles	10 (9.0)	10 (8.5)		
"In-situ" tests: dilatometer tests (52 samples)	25 (22.5)	20 (17.0)		
Laboratory tests: uniaxial tests (31 samples) point load tests (19 samples) ultrasonic velocity (29 samples) Schmidt hammer (3 samples)	10 (9.0)	10 (8.5)		
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	90	85		

SABUGAL-MEIMOA TUNNEL

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	5	8	9	
Phase 1 - Reconnaissance and gathering existing information	20 (10.0)	20 (10.0)	30 (15.0)	
Phase 2 - 7 boreholes (378.55 m)	80 (40.0)	80 (40.0)	70 (35.0)	
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	50	50	50	

SADO-MORGAVEL TUNNEL

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	2	8		
Phase 1 - Reconnaissance and gathering existing information	25 (17.5)	15 (13.5)		
Phase 2 - Geophysical exploration: 79 seismic refraction profiles	20 (14.0)	20 (18.0)		
Phase 3 - 3 percussion drills (45.36 m) + 24 boreholes (1 078 m)	35 (24.5)	45 (40.5)		
Integral sampling (108 m)	- (-)	- (-)		
Phase 4 - "In-situ" tests: dilatometer tests (26 tests)	15 (10.5)	15 (13.5)		
Laboratory tests: uniaxial tests (20 samples) ultrasonic velocity (19 samples)	5 (3.5)	5 (4.5)		

(continued)

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	2	8		
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	70	90		

STA CLARA-MTE DA ROCHA TUNNEL

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	2	8		
Phase 1 - Reconnaissance and gathering existing information	15 (12.0)	20 (19.0)		
Phase 2 - Geophysical exploration: 15 seismic refraction profiles	20 (16.0)	20 (19.0)		
Phase 3 - Geophysical exploration: 57 geoelectrical soundings 1 apparent resistivity profiles at three depths	10 (8.0)	5 (4.8)		
15 boreholes (859.1 m)	30 (24.0)	35 (33.3)		
"In-situ" tests: dilatometer tests (20 tests) state of stress (2 tests)	15 (12.0)	13 (12.4)		
Laboratory tests: uniaxial tests (12 samples) ultrasonic velocity (5 samples)				

(continued)

	% OF KNOWLEDGE FROM EACH TYPE OF WORK			
	2	8		
porosity (6 samples)				
bulk density (6 samples)	5	5		
	(4.0)	(4.8)		
Phase 4 - Geophysical exploration:				
10 apparent resistivity profiles at two depths and apparent resistivity map	5	2		
	(4.0)	(1.9)		
% of knowledge at completion of site investigations in relation to that considered desirable (100%)	80	95		

APPENDIX C

THE COSTS

ALTO LINDOSO DAM	cost when performed ($\times 10^3$ esc)	cost at 1980 prices ($\times 10^3$ esc)
construction (budjet)	1 370 x 10^3 (1977)	2 420 x 10^3
eng. geological staff	400 (1977)	700
9 BOREHOLES	300 (1968)	1 790
6 GALLERIES *	830 (1969)	4 580
6 BOREHOLES	190 (1971)	940
5 BOREHOLES	400 (1977)	710
12 TRENCHES	120 (1977)	210
LABORATORY TESTS	250 (1977)	440
exploration + testing works		8 670
total for site investigations (exploration + testing + eng. geological staff)		9 370
cost of site investigations / cost of construction		0.4 %

ALVIDE DAM	cost when performed ($\times 10^3$ esc)	cost at 1980 prices ($\times 10^3$ esc)
construction (budjet)	10 x 10^3 (1972)	45 x 10^3
eng. geological staff	100 (1972)	450
1 TRENCH	40 (1971	210
6 BOREHOLES	150 (1971)	720
1 BOREHOLE	30 (1972)	120
1 GALLERY	380 (1972)	1 700
3 BOREHOLES	140 (1976)	310
GEOPHYSICAL EXPLORATION	30 (1977)	50
"IN-SITU" TESTS	210 (1977)	370
LABORATORY TESTS	40 (1977)	70
exploration + testing works		3 550
total for site investigations (exploration + testing + eng. geological staff)		4 000
cost of site investigations / cost of construction		8.9 %

* Cost assumed based on actual prices

ATROZELA DAM	cost when performed ($x10^3$esc)	cost at 1980 prices ($x10^3$esc)
construction (budjet)	29×10^3 (1977)	50×10^3
eng. geological staff	250 (1977)	440
4 BOREHOLES	150 (1972)	680
5 BOREHOLES	190 (1975)	510
8 PERCUSSION DRILLS	200 (1975)	530
2 TRENCHES	8 (1975)	20
exploration + testing works		1 740
total for site investigations (exploration + testing + eng. geological staff)		2 180
cost of site investigations / cost of construction		4.4 %

FAGILDE DAM	cost when performed ($x10^3$esc)	cost at 1980 prices ($x10^3$esc)
construction (budjet)	42×10^3 (1979)	50×10^3
eng. geological staff	40 (1979)	50
1 TRENCH	2 (1973)	8
4 BOREHOLES	100 (1973)	410
exploration + testing works		418
total for site investigations (exploration + testing + eng. geological staff)		468
cost of site investigations / cost of construction		0.9 %

FOZ - CÔA DAM	cost when performed ($\times 10^3$ esc)	cost at 1980 prices ($\times 10^3$ esc)
construction (budjet)	6 000 x 10^3 (1980)	6 000 x 10^3
eng. geological staff	1 000 (1980)	1 000
6 TRENCHES	1 690 (1977)	2 980
GEOPHYSICAL EXPLORATION	300 (1977)	530
9 TRENCHES	3 660 (1979)	4 260
10 GALLERIES	8 030 (1979)	9 360
62 BOREHOLES	5 770 (1979)	6 730
GEOPHYSICAL EXPLORATION	550 (1979)	640
"IN-SITU" TESTS	4 570 (1979)	5 330
LABORATORY TESTS	360 (1979)	420
exploration + testing works		30 250
total for site investigations (exploration + testing + eng. geological staff)		31 250
cost of site investigations / cost of construction		0.5 %

FRONHAS DAM	cost when performed ($\times 10^3$ esc)	cost at 1980 prices ($\times 10^3$ esc)
construction (budjet)	500 x 10^3 (1976)	1 130 x 10^3
eng. geological staff	400 (1976)	900
14 BOREHOLES (1938)*	390 (1976)	880
4 GALLERIES (1938)*	440 (1976)	990
GEOPHYSICAL EXPLORATION	170 (1975)	450
9 TRENCHES	750 (1976)	1 690
8 GALLERIES	1 150 (1976)	2 590
18 BOREHOLES	850 (1976)	1 910
"IN-SITU" TESTS	2 150 (1976)	4 840
LABORATORY TESTS	140 (1976)	320
exploration + testing works		13 670
total for site investigations (exploration + testing + eng. geological staff)		14 570
cost of site investigations / cost of construction		1.3 %

* Cost assumed based on actual prices

FUNCHO DAM	cost when performed ($\times 10^3$ esc)	cost at 1980 prices ($\times 10^3$ esc)
construction (budjet)	170×10^3 (1978)	240×10^3
eng. geological staff	520 (1978)	750
1 TRENCH	60 (1971)	310
9 BOREHOLES	300 (1972)	1 360
1 GALLERY	200 (1974)	630
1 TRENCH	20 (1975)	50
4 GALLERIES	980 (1976)	2 210
"IN-SITU" TESTS	1 050 (1976)	2 360
LABORATORY TESTS	85 (1976)	190
exploration + testing works		7 110
total for site investigations (exploration + testing + eng. geological staff)		7 860
cost of site investigations / cost of construction		3.3 %

TORRÃO DAM	cost when performed ($\times 10^3$ esc)	cost at 1980 prices ($\times 10^3$ esc)
construction (budjet)	360×10^3 (1969)	$2\,000 \times 10^3$
eng. geological staff	270 (1969)	1 500
10 TRENCHES	720 (1969)	3 990
19 GALLERIES	1 730 (1969)	9 590
GEOPHYSICAL EXPLORATION	180 (1969)	1 000
19 BOREHOLES	460 (1969)	2 550
"IN-SITU" TESTS	1 200 (1969)	6 650
LABORATORY TESTS	100 (1969)	550
GEOPHYSICAL EXPLORATION	110 (1970)	580
exploration + testing works		24 910
total for site investigations (exploration + testing + eng. geological staff)		26 410
cost of site investigations / cost of construction		1.3 %

JAMBA-IA-MINA DAM	cost when performed ($\times 10^3$ esc)	cost at 1980 prices ($\times 10^3$ esc)
construction (budjet)	300×10^3 (1974)	960×10^3
eng. geological staff	290 (1974)	930
17 BOREHOLES	1 050 (1971)	5 170
GEOPHYSICAL EXPLORATION	280 (1971)	1 380
exploration + testing works		6 550
total for site investigations (exploration + testing + eng. geological staff)		7 480
cost of site investigations / cost of construction		0.8 %

JAMBA-IA-OMA DAM	cost when performed ($\times 10^3$ esc)	cost at 1980 prices ($\times 10^3$ esc)
construction (budjet)	350×10^3 (1976)	790×10^3
eng. geological staff	420 (1976)	950
19 BOREHOLES	970 (1971)	4 750
GEOPHYSICAL EXPLORATION	180 (1975)	470
2 BOREHOLES	150 (1975)	400
exploration + testing works		5 620
total for site investigations (exploration + testing + eng. geological staff)		6 570
cost of site investigations / cost of construction		0.8 %

TOUVEDO DAM	cost when performed ($\times 10^3$ esc)	cost at 1980 prices ($\times 10^3$ esc)
construction (budjet)	490 x 10^3 (1978)	710 x 10^3
eng. geological staff	350 (1978)	500
8 TRENCHES	230 (1977)	410
23 BOREHOLES	1 430 (1977)	2 530
exploration + testing works		2 940
total for site investigations (exploration + testing + eng. geological staff)		3 440
cost of site investigations / cost of construction		0.5 %

RAIVA DAM	cost when performed ($\times 10^3$ esc)	cost at 1980 prices ($\times 10^3$ esc)
construction (budjet)	200 x 10^3 (1974)	640 x 10^3
eng. geological staff	190 (1974)	600
62 PERCUSSION DRILLS	50 (1972)	230
2 TRENCHES	20 (1973)	90
SPECIAL STUDY OF JOINTING	20 (1973)	80
GEOPHYSICAL EXPLORATION	80 (1973)	330
19 BOREHOLES	370 (1973)	1 180
exploration + testing works		1 910
total for site investigations (exploration + testing + eng. geological staff)		2 510
cost of site investigations / cost of construction		0.4 %

VALEIRA DAM	cost when performed ($\times 10^3$ esc)	cost at 1980 prices ($\times 10^3$ esc)
construction (budjet)	500 x 10^3 (1969)	2 800 x 10^3
eng. geological staff	250 (1969)	1 400
2 BOREHOLES (1948) *	50 (1969)	280
4 BOREHOLES	220 (1964)	1 460
4 GALLERIES *	380 (1967?)	2 320
9 BOREHOLES	260 (1967)	1 590
18 BOREHOLES	410 (1968)	2 460
2 TRENCHES	50 (1968)	290
exploration + testing works		8 400
total for site investigations (exploration + testing + eng. geological staff)		9 800
cost of site investigations / cost of construction		0.4 %

VALHELHAS DAM	cost when performed ($\times 10^3$ esc)	cost at 1980 prices ($\times 10^3$ esc)
construction (budjet)	280 x 10^3 (1973)	1 150 x 10^3
eng. geological staff	340 (1973)	1 400
5 TRENCHES	200 (1973)	820
SPECIAL STUDY OF JOINTING	20 (1973)	80
GEOPHYSICAL EXPLORATION	65 (1973)	270
39 BOREHOLES	700 (1973)	2 850
INTEGRAL SAMPLING	110 (1973)	430
LABORATORY TESTS	35 (1973)	140
exploration + testing works		4 590
total for site investigations (exploration + testing + eng. geological staff)		5 990
cost of site investigations / cost of construction		0.5 %

* Cost assumed based on actual prices

AZIBO DAM	cost when performed ($\times 10^3$ esc)	cost at 1980 prices ($\times 10^3$ esc)
construction (budjet)	230×10^3 (1976)	520×10^3
eng. geological staff	700 (1976)	1 580
1 TRENCH	80 (1971)	390
18 BOREHOLES	430 (1972)	1 950
8 BOREHOLES	320 (1976)	710
1 TRENCH	100 (1976)	230
2 TRENCHES	30 (1978)	45
exploration + testing works		3 325
total for site investigations (exploration + testing + eng. geological staff)		4 905
cost of site investigations / cost of construction		0.9 %

CAPINHA DAM	cost when performed ($\times 10^3$ esc)	cost at 1980 prices ($\times 10^3$ esc)
construction (budjet)	27×10^3 (1977)	50×10^3
eng. geological staff	160 (1977)	280
1 TRENCH	15 (1974)	50
1 BOREHOLE	75 (1974)	240
5 BOREHOLES	250 (1977)	440
1 TRENCH	20 (1977)	35
2 BOREHOLES	100 (1977)	180
exploration + testing works		945
total for site investigations (exploration + testing + eng. geological staff)		1 225
cost of site investigations / cost of construction		2.5 %

MEIMOA DAM	cost when performed ($\times 10^3$ esc)	cost at 1980 prices ($\times 10^3$ esc)
construction (budjet)	290 x 10^3 (1974)	930 x 10^3
eng. geological staff	500 (1974)	1 600
7 TRENCHES	240 (1971)	1 180
20 BOREHOLES	330 (1971)	1 630
7 TRENCHES	2 260 (1974)	7 260
3 AUGER BORIGNS + 12 BOREHOLES	460 (1974)	1 480
exploration + testing works		11 550
total for site investigations (exploration + testing + eng. geological staff)		13 150
cost of site investigations / cost of construction		1.4 %

MORGAVEL DAM	cost when performed ($\times 10^3$ esc)	cost at 1980 prices ($\times 10^3$ esc)
construction (budjet)	200 x 10^3 (1975)	530 x 10^3
eng. geological staff	70 (1975)	190
16 BOREHOLES + 13 PERCUSSION DRILLS	500 (1973)	2 050
GEOPHYSICAL EXPLORATION	25 (1975)	65
exploration + testing works		2 115
total for site investigations (exploration + testing + eng. geological staff)		2 305
cost of site investigations / cost of construction		0.4 %

ODELOUCA DAM	cost when performed ($\times 10^3$ esc)	cost at 1980 prices ($\times 10^3$ esc)
construction (budjet)	620 x 10^3 (1977)	1 100 x 10^3
eng. geological staff	500 (1977)	880
14 TRENCHES	65 (1972)	290
18 BOREHOLES	320 (1972)	1 460
GEOPHYSICAL EXPLORATION	110 (1972)	500
2 GALLERIES	940 (1976)	2 110
GEOPHYSICAL EXPLORATION	60 (1976)	140
2 SHAFTS	580 (1976)	1 310
12 BOREHOLES	600 (1977)	1 060
1 TRENCH	60 (1977)	110
exploration + testing works		6 980
total for site investigations (exploration + testing + eng. geological staff)		7 860
cost of site investigations / cost of construction		0.7 %

SABUGAL DAM	cost when performed ($\times 10^3$ esc)	cost at 1980 prices ($\times 10^3$ esc)
construction (budjet)	250 x 10^3 (1978)	360 x 10^3
eng. geological staff	420 (1978)	610
1 TRENCH	230 (1972)	1 020
10 BOREHOLES	360 (1972)	1 610
8 BOREHOLES	270 (1974)	870
GEOPHYSICAL EXPLORATION	105 (1974)	340
GEOPHYSICAL EXPLORATION	30 (1976)	70
exploration + testing works		3 910
total for site investigations (exploration + testing + eng. geological staff)		4 520
cost of site investigations / cost of construction		1.3 %

ALTO LINDOSO TUNNEL	cost when performed ($\times 10^3$esc)	cost at 1980 prices ($\times 10^3$esc)
construction (budjet)	2 430 x 10^3 (1977)	4 300 x 10^3
eng. geological staff	600 (1977)	1 000
12 BOREHOLES	920 (1977)	1 630
28 TRENCHES	170 (1977)	290
"IN-SITU" TESTS	560 (1977)	990
LABORATORY TESTS	110 (1977)	190
exploration + testing works		3 100
total for site investigations (exploration + testing + eng. geological staff)		4 100
cost of site investigations / cost of construction		0.1 %

CASTELO DO BODE TUNNEL	cost when performed ($\times 10^3$esc)	cost at 1980 prices ($\times 10^3$esc)
construction (budjet)	590 x 10^3 (1979)	690 x 10^3
eng. geological staff	1 900 (1979)	2 220
GEOPHYSICAL EXPLORATION	75 (1975)	200
13 BOREHOLES	1 970 (1976)	4 430
"IN-SITU" TESTS	620 (1976)	1 400
LABORATORY TESTS	25 (1976)	55
GEOPHYSICAL EXPLORATION	200 (1978)	290
LABORATORY TESTS	50 (1978)	70
exploration + testing works		6 445
total for site investigations (exploration + testing + eng. geological staff)		8 665
cost of site investigations / cost of construction		1.3 %

FUNCHO-BENACIATE TUNNEL	cost when performed ($\times 10^3$esc)	cost at 1980 prices ($\times 10^3$esc)
construction (budjet)	300 x 10^3 (1980)	300 x 10^3
eng. geological staff	700 (1980)	700
15 BOREHOLES	1 050 (1977)	1 860
GEOPHYSICAL EXPLORATION	30 (1977)	55
"IN-SITU" TESTS	560 (1977)	990
LABORATORY TESTS	40 (1977)	70
exploration + testing works		2 975
total for site investigations (exploration + testing + eng. geological staff)		3 675
cost of site investigations / cost of construction		1.2 %

SABUGAL-MEIMOA TUNNEL	cost when performed ($\times 10^3$esc)	cost at 1980 prices ($\times 10^3$esc)
construction (budjet)	160 x 10^3 (1978)	230 x 10^3
eng. geological staff	260 (1978)	380
7 BOREHOLES	370 (1974)	1 170
exploration + testing works		1 170
total for site investigations (exploration + testing + eng. geological staff)		1 550
cost of site investigations / cost of construction		0.7 %

SADO-MORGAVEL TUNNEL	cost when performed ($\times 10^3$esc)	cost at 1980 prices ($\times 10^3$esc)
construction (budjet)	330 x 10^3 (1974)	1 060 x 10^3
eng. geological staff	250 (1974)	800
GEOPHYSICAL EXPLORATION	210 (1974)	670
3 PERCUSSION DRILLS + 24 BOREHOLES	1 000 (1974)	3 210
INTEGRAL SAMPLING	210 (1974)	660
"IN-SITU" TESTS	250 (1974)	800
LABORATORY TESTS	40 (1974)	130
exploration + testing works		5 470
total for site investigations (exploration + testing + eng. geological staff)		6 270
cost of site investigations / cost of construction		0.6 %

STA CLARA-MTE DA ROCHA TUNNEL	cost when performed ($\times 10^3$esc)	cost at 1980 prices ($\times 10^3$esc)
construction (budjet)	965 x 10^3 (1979)	1 130 x 10^3
eng. geological staff	350 (1979)	410
GEOPHYSICAL EXPLORATION	70 (1977)	125
GEOPHYSICAL EXPLORATION	180 (1978)	250
15 BOREHOLES	2 270 (1978)	3 290
"IN-SITU" TESTS	700 (1978)	1 010
LABORATORY TESTS	35 (1978)	50
GEOPHYSICAL EXPLORATION	55)1979)	65
exploration + testing works		4 790
total for site investigations (exploration + testing + eng. geological staff)		5 200
cost of site investigations / cost of construction		0.5 %