HELLENIC DEMOCRACY

MINISTRY OF INFRASTRUCTURE, TRANSPORTATION AND NETWORKS
GENERAL SECRETARIAT FOR PUBLIC WORKS

SYKIA DAM

PANEL OF EXPERTS
REPORT

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Prof. Louis Berga   Bernard Tardieu

…………….……………………………..   ………..……………………………

Prof. George Koukis   Dr. Nikolaos Moutafis

Athena

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1. INTRODUCTION

Sykia project, comprising an embankment dam and its appurtenant works, has been under construction on the Acheloos River for several years, under various contracts. Sykia dam, which is situated at the confluence of the Acheloos and the Kouboutjainnitikos rivers, is part of the works for the partial diversion of the upper Acheloos River towards the Thessaly plane.

Works on the site commenced in mid ‘80s with the excavation of two diversion tunnels, one on the Acheloos River and one in-between the Acheloos and the Koubourjannitikos Rivers.

The diversion of the rivers through the diversion tunnels was implemented approx. 10 years later, in 1997, by the construction of three cofferdams, under a separate contract. Within the framework of this contract, a large volume of excavations was accomplished, mainly on the right abutment. The contract was later terminated and the works were interrupted until 2005, when a new contract was signed for the construction of the dam and the remaining appurtenant works.

It is noted that since 1997 and until today (2010), Koubourjannitikos River flows through the diversion tunnel that connects the tributary to the main river, while the combined waters of both Acheloos and Koubourjannitikos rivers flow through the Acheloos Diversion Tunnel.

Construction of the dam and the remaining appurtenant structures continued until a recent order by the Supreme Court of Greece, in accordance to its decision No. 141/2010, requested “the immediate interruption of all works related to, or aiming at, the construction of the partial diversion of the upper Acheloos River towards the Thessaly plane, and the abstention from any physical act aiming at the completion and operation of the works related to the river diversion”.

In accordance to the Supreme Court’s order, the Contractor was instructed on the 10th March 2010, to interrupt immediately all works. It is noted that the Supreme Court’s order includes and covers any works on the Messochora project.

The abrupt interruption of the works at Sykia dam, has left the gross majority of the project structures unfinished and some in full operation, e.g. the diversion tunnels and the cofferdams. The fact that some of those structures that are presently in full operation, have been designed to operate for a limited period of time, suffering from a gradual and continuous deterioration, has created a grave concern to the relevant departments of the Ministry of Infrastructure, Transportation and Networks (MITN), if those structures cease to operate as designed and planned, or if they fail. Monitoring of the project conditions is also an important and tedious task that has to be dealt with.

In order to investigate the hazards (and possibly the potential damages), imposed by the abrupt (and of indefinite duration) interruption of project construction, on the dam related works that have been constructed, on the local infrastructure and on the social activities in the downstream areas, the MITN has formed a 4 member Panel of Experts (PoE), by the undersigned Greek and foreign specialists, members of the International Commission on Large Dams.

2. OBJECT OF PANEL OF EXPERTS

The object of the PoE, as specified in the relevant individual contract documents between the MITN and the panel members, is that the panel members after have been informed on:

a) The approved final design of the project,

b) The modifications and adjustments to the approved final design to suit the in situ conditions,

c) All the works that have been constructed within the framework of the present and the previous construction contracts,

and after they have visited the project site and have investigated the existing conditions, will prepare a joint report dealing indicatively and not exclusively, with the following:

- Estimation of potential hazards that may cause damages to the accomplished or constructed works related to Sykia dam and the appurtenant works, by the abrupt and of indefinite duration interruption of the works and the importance of those hazards.
- Estimation of potential hazards that may cause damages to the areas downstream of the Sykia dam site, by the abrupt and of indefinite duration interruption of the works and the importance of those hazards.
- Proposals to eliminate or mitigate the hazards that may cause damages to the works constructed and to the downstream areas, the necessity to implement those proposals and the estimated construction schedule.

3. AVAILABLE DATA

3.1 Reports

The MITN gave to the PoE hard copies of the following documents and reports:

1) ΔΕΗ/ΔΑΥΕ – ΦΡΑΓΜΑ ΣΥΚΙΑΣ - ΕΚΘΕΣΗ ΕΠΙ ΤΗΣ ΑΝΤΙΜΕΤΩΠΙΣΗΣ ΔΙΑΡΡΟΩΝ ΚΑΤΑ ΤΗΝ ΕΚΣΚΑΦΗ ΤΗΣ ΤΑΦΡΟΥ ΠΥΡΗΝΑ – ΑΘΗΝΑ – ΝΟΕΜΒΡΙΟΣ 2007.
2) Υ.ΠΕ.ΧΩ.Δ.Ε. – ΑΠΟΠΕΡΑΤΩΣΗ ΦΡΑΓΜΑΤΟΣ ΣΥΚΙΑΣ – ΤΕΧΝΙΚΗ ΠΕΡΙΓΡΑΦΗ – Κ.Ε.1900.Γ – Νοέμβριος 2004.
3) Απόφαση Υπουργείου Υποδομών, Μεταφορών και Δικτύων, οικ.1273/Μ.Σ.890/26 Μαΐου 2010 για «Συγκρότηση ομάδας Εμπειρογνωμόνων ως Τεχνικών Συμβούλων της ΕΥΔΕ/ΟΣΥΕ, για τη διερεύνηση και επίλυση τεχνικών προβλημάτων σε θέματα ασφάλειας, αντιμετώπισης και αποτροπής κινδύνου στα έργα μερικής εκτροπής του άνω ρου του ποτ. Αχελώου προς Θεσσαλία».
4) Απόφαση Επιτροπής Αναστολών του Συμβουλίου της Επικρατείας: Αριθμός 141/2010 που εκδόθηκε στις 10 Φεβρουαρίου 2010.

3.2 Drawings

The MITN gave to the PoE hard copies of the following drawings:

Drawings for the construction of the Diversion Tunnels:

3000130-C12-02/1985	3000130-C12-07a/1987	3000130-C12-16/1987
3000130-C12-04/1985	3000130-C12-09/1985	3000130-C12-19/1987
3000130-C12-05/1985	3000130-C12-10/1987	
3000130-C12-06/1985	3000130-C12-11/1987	

Drawings for the construction of the Dam:

1404-CTR-02-01A/2002	1404-CTR-12-13A/2002	1404-CTR-12-131D/2002
1404-CTR-02-02A/2002	1404-CTR-12-14A/2002	1404-CTR-12-132F/2002
1404-CTR-02-03A/2002	1404-CTR-12-14B/2002	1404-CTR-12-132G/2002
1404-CTR-06-01A/2002	1404-CTR-12-15A/2002	1404-CTR-12-132H/2002
1404-CTR-06-02A/2002	1404-CTR-12-16A/2002	1404-CTR-12-132I/2002

1404-CTR-06-03A/2002	1404-CTR-12-17A/2002	1404-CTR-12-132K/2002
1404-CTR-06-04A/2002	1404-CTR-12-18A/2002	1404-CTR-12-132L/2002
1404-CTR-06-06A/2002	1404-CTR-12-19A/2002	1404-CTR-12-134B/2002
1404-CTR-06-15A/2002	1404-CTR-12-20A/2002	1404-CTR-12-134C/2003
1404-CTR-06-16A/2002	1404-CTR-12-21A/2002	1404-CTR-12-151B/2002
1404-CTR-06-17A/2002	1404-CTR-12-40A/2002	1404-CTR-12-201A/2002
1404-CTR-06-18A/2002	1404-CTR-12-41A/2002	1404-CTR-12-202A/2002
1404-CTR-06-19A/2002	1404-CTR-12-42A/2002	1404-CTR-12-203A/2002
1404-CTR-06-20A/2002	1404-CTR-12-51/2002	1404-CTR-12-204A/2002
1404-CTR-07-05A/2002	1404-CTR-12-52/2002	1404-CTR-12-205/2002
1404-CTR-07-06A/2002	1404-CTR-12-53/2002	1404-CTR-12-206/2002
1404-CTR-08-01B/2002	1404-CTR-12-54/2002	1404-CTR-12-207/2002
1404-CTR-08-03B/2002	1404-CTR-12-55/2002	1404-CTR-12-221/2003
1404-CTR-08-133/002	1404-CTR-12-56/2002	1404-CTR-12-222/2003
1404-CTR-08-135/002	1404-CTR-12-60A/2002	1404-CTR-12-223/2003
1404-CTR-11-01A/2002	1404-CTR-12-61A/2002	1404-CTR-12-224/2003
1404-CTR-11-02A/2002	1404-CTR-12-62A/2002	1404-CTR-12-225/2003
1404-CTR-11-03A/2002	1404-CTR-12-63A/2002	1404-CTR-12-226/2003
1404-CTR-11-04A/2002	1404-CTR-12-64A/2002	1404-CTR-14-01A/2002
1404-CTR-11-05A/2002	1404-CTR-12-65A/2002	1404-CTR-14-02A/2002
1404-CTR-12-04A/2002	1404-CTR-12-66A/2002	1404-CTR-15-01/2003
1404-CTR-12-05A/2002	1404-CTR-12-121A/2002	1404-CTR-15-02/2003
1404-CTR-12-07A/2002	1404-CTR-12-122A/2002	1404-CTR-15-03/2003
1404-CTR-12-08A/2002	1404-CTR-12-123A/2002	1404-CTR-15-04/2003
1404-CTR-12-09B/2002	1404-CTR-12-124A/2002	1404-CTR-15-05/2003
1404-CTR-12-10A/2002	1404-CTR-12-126A/2002	1404-CTR-15-06/2003
1404-CTR-12-11A/2002	1404-CTR-12-127A/2002	
1404-CTR-12-12A/2002	1404-CTR-12-131C/2002	

Drawings prepared by the Contractor:

1. ORIZ_1/22-07-2010: General plan of Sykia Dam
2. TOM_1/22-07-2010: Sections 1 and 2 of Sykia Dam

3.3 Abbreviations

MITN	Ministry of Infrastructure, Transportation and Networks
PoE	Panel of Experts
PPC	Public Power Corporation of Greece
DAYE	Department for Development of Hydroelectric projects
DT-1	Diversion Tunnel No. 1 on the Acheloos River
DT-2	Diversion Tunnel No. 2 on the Koubourjiannitikos River
MUC	Main Upstream Cofferdam
U/S	Upstream
D/S	Downstream
I.D.	Internal Diamenter
hm^3	≡ 1 million cubic meters
Fig. …..	Figure No. …..
Photo. ….	Photograph No. ……
H:V	Horizontal to Vertical

4. EXISTING SITUATION

4.1 Introduction

Sykia project has been designed as both a hydroelectric project and as a storage reservoir for the transfer of waters from the Acheloos basin (in western Greece) to the Thessaly plane in the east. The parts of the project that are related to water storage and water transfer to the Thessaly plane are under the auspices of the MITN, while the structures related to hydroelectric energy production are under the auspices of the Public Power Corporation (PPC) of Greece.

The works related to the water storage are an embankment dam with central core, an overflow spillway with a plunge pool, three diversion tunnels and cofferdams, grouting and drainage tunnels, grouting and drainage works, access roads and haul roads, excavations, reservoir works, etc.

The embankment dam is located a short distance downstream of the confluence of Acheloos and Koubourjiannitikos rivers. The dam axis is curved upstream to fit better to the abutment contours.

Construction of the dam necessitates diversion of both rivers to bypass the dam site. This is achieved by a) diverting Koubourjiannitikos River towards the Acheloos valley and b) diverting the combined waters of Acheloos and Koubourjiannitikos rivers from the dam site.

Of those works related to water storage, the PoE has reviewed solely and exclusively those that are or maybe affected by the abrupt interruption of the works. Those works are summarized in the following list:

a) Diversion tunnels
b) Cofferdams
c) Dam excavations and dam shells
d) Spillway
e) Stockpiled and dumped materials

Construction of above works up to their abrupt interruption in March 2010, has been achieved through three different discrete main phases (and different construction contracts):

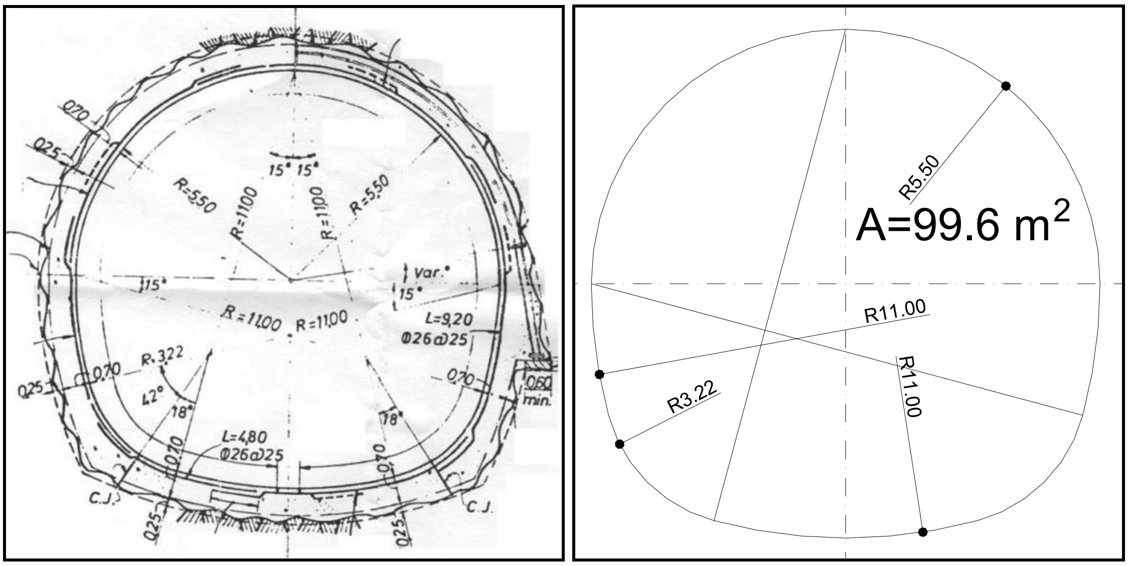
- Construction of Acheloos and Koubourjiannitikos diversion tunnels, in the late 80's.
- Dam excavations and construction of the cofferdams, in the late 90's
- Construction of dam shells, spillway and additional diversion tunnel, since 2005.

The works that have been reviewed by the PoE are briefly presented in the following paragraphs.

4.2 Diversion Tunnels

4.2.1 Acheloos Diversion Tunnel – Diversion Tunnel No. 1

The Acheloos Diversion Tunnel (indicated on the contract drawings as Diversion Tunnel No. 1 – henceforth referred to as DT-1), is located on the right bank of the Acheloos river and is of a horseshoe section (see Fig. 4-1 a) of 99.6 m^2 cross sectional area (see Fig. 4-1 b). The invert elevation at the intake structure is +414.00 and at the outlet +406.00. It has a length of ~959 m, with a longitudinal inclination upstream of the permanent plug 6.5‰ and downstream of the plug 10.43‰.



(a) (b)

Fig. 4-1 Typical cross section of the Acheloos Diversion Tunnel (DT - 1)

The tunnel is lined over its whole length with 0.70 m thick reinforced concrete, as shown on Fig. 4-1. Excavation lines and concrete reinforcement may vary locally along the tunnel, depending on the rockmass conditions revealed during the excavation and the support measures applied.

The tunnel was excavated and concrete lined in the late 80's. The inlet and outlet portals were also constructed during that period, but the works were not used for a period of approximately 10 years, till 1997, when the Acheloos River was diverted through the tunnel and the cofferdams were constructed.

The DT-1 inlet portal is connected with the inlet of the Bottom Outlet (see Photo. 4-1 a). The open excavations at the inlet portal have been extensive and the slopes are supported by pre-stressed anchors, as shown in Photo. 4-1 b. The pre-stressed anchors have not been maintained since their installation in the mid 80's.

(a) (b)

Photo. 4-1 – Acheloos Diversion Tunnel Inlet (a) and portal excavation support measures (b)

The tunnel outlet portal is shown in Photo. 4-2.

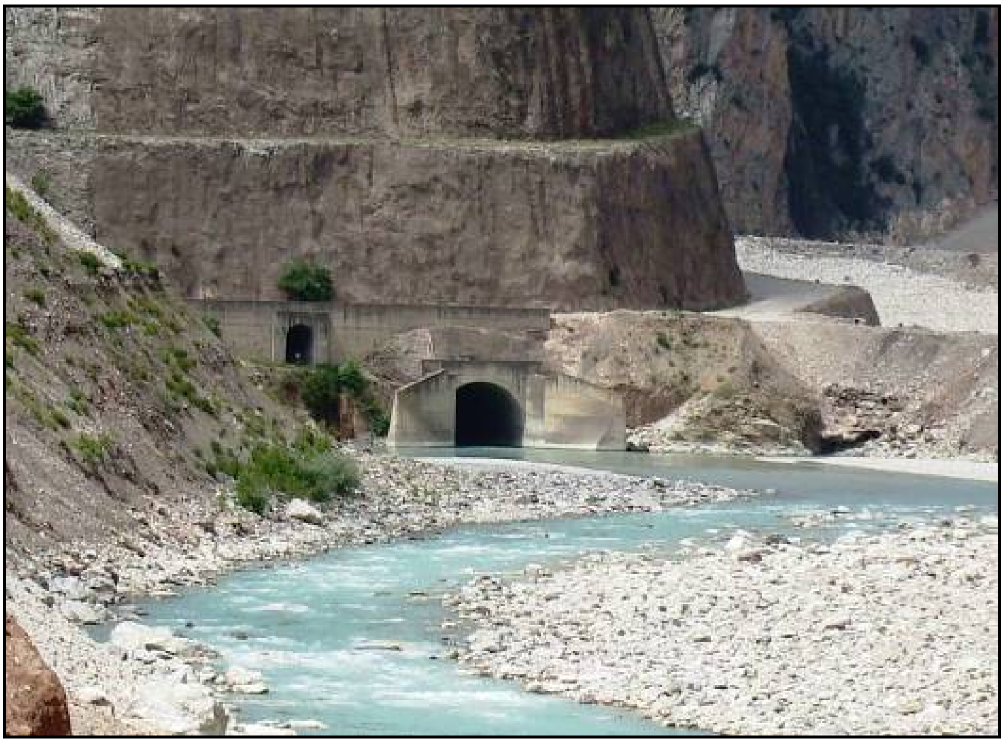


Photo. 4-2 - Acheloos Diversion Tunnel – Outlet Portal

The rating curve of Acheloos Diversion Tunnel (DT - 1) is shown in Fig. 4-2.

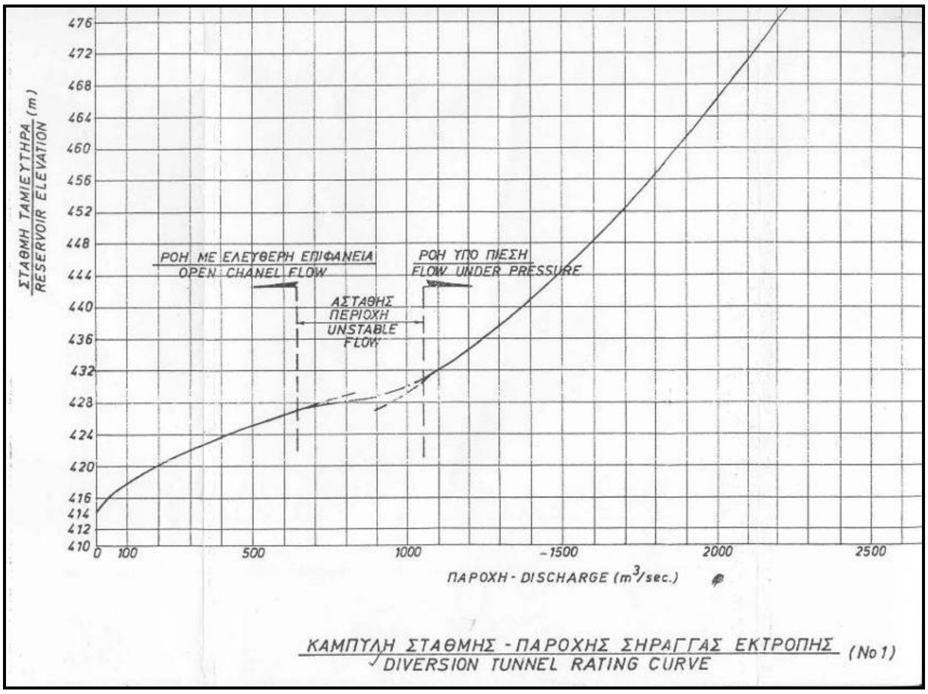


Fig. 4-2. Acheloos Diversion Tunnel rating curve

(Diagram obtained from diversion tunnels design drawings)

4.2.2 Koubourjiannitikos Diversion Tunnel – Diversion Tunnel No. 2

The Koubourjiannitikos Diversion Tunnel (indicated on the drawings as Diversion Tunnel No. 2, henceforth referred to as DT-2), is located on the right bank of the Koubourjiannitikos river and is of a horseshoe section of 82.3 m^2 cross sectional area (see Fig. 4-3).

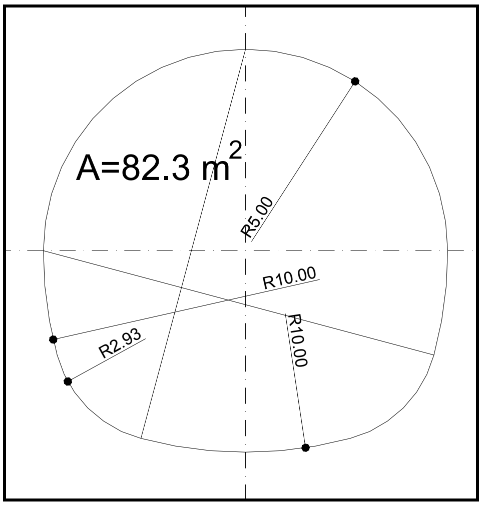


Fig. 4-3. Koubourjiannitikos diversion tunnel cross sectional area

Invert elevation at the intake structure is +417.96 and at the outlet +415.00. It has a length of ~602 m, with a longitudinal inclination 4.9 ‰.

The tunnel is lined in the lower half, with reinforced concrete and lined over the whole perimeter at the portals. The lining thickness at the portals is 0.60 m. Reinforcement may vary locally, depending on the rockmass conditions, but in general the rockmass was reported to be good and the tunnel was only partly lined.

Floods have partly filled the tunnel with sediments. During the visit of the PoE to the site, the inlet portal was half filled with sediments (see Photo. 4-3), but the outlet portal was almost free of sediments (see Photo. 4-4).

(a)

(b)

Photo. 4-3 – Koubourjiannitikos Diversion Tunnel – Inlet portal half filled with river sediments

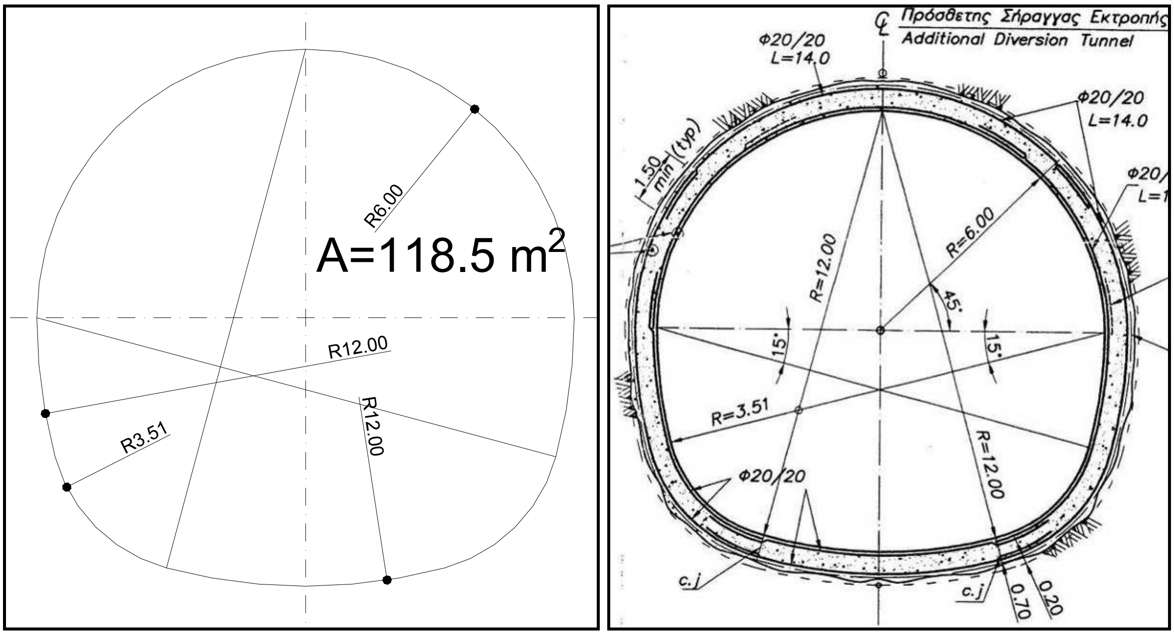
Photo. 4-4 - Koubourjiannitikos Diversion Tunnel – Outlet portal

4.2.3 Additional Diversion Tunnel

In 2003, following a period of heavy rains, a landslide occurred on the right bank, downstream of the dam site, blocking the river and flooding the area between the slide and dam site (see § 5.3).

In an aim to avoid future blockage of the river by re-activation of the said slide or otherwise, an additional diversion tunnel has been excavated, totally by-passing the slide area. The tunnel was excavated and concrete lined by the dam contractor.

The additional diversion tunnel is located on the left bank of the Acheloos river and is of a horseshoe section, with a cross sectional area of 118.5 m^2 (see Fig. 4-4). It has a length of 429.85 m and a longitudinal inclination of 7‰.



(a) (b)

Fig. 4-4. Additional Diversion Tunnel – (a) Typical cross section, (b) Concrete lining

The inlet portal (see Photo. 4-5 a) is located in-between the toe of the dam shell and the main D/S cofferdam.

The elevation of the invert at the inlet postal is +410.00 and at the outlet portal +400.00. The outlet portal is shown in Photo. 4-5 (b).

(a)

(b)

Photo. 4-5 – Additional Diversion Tunnel – (a) Inlet portal, (b) Outlet portal

4.3 Cofferdams

4.3.1 Introduction

River diversion from the dam site required the construction of two Main Upstream Cofferdams (MUC), one on each river and a main downstream cofferdam on the Acheloos river. The term 'main' differentiates those three cofferdams from other auxiliary cofferdams implemented for the construction of the works,

4.3.2 MUC on the Acheloos River

The MUC on the Acheloos River is an embankment dam with a central impervious core. The cofferdam shells have been constructed with rockfill from the diversion tunnel excavations. River alluvia may have been used locally. The cofferdam crest has been formed at elevation +458.50. It may be assumed that the crest of the core is one meter below, i.e at elevation +457.50. A typical section of the MUC is shown in Fig. 4-5. Unconfirmed information indicated that the faces of the cofferdam were constructed at a slope 2:1 (H:V), while the core faces at a slope 1:10 (H:V).

The cofferdam is founded on the river alluvia, following removal of the uppermost unsuitable materials. The alluvia underneath the cofferdam were treated with one line of jet grouting holes, to reduce their permeability. The jet grouting holes extended to the underlying bedrock.

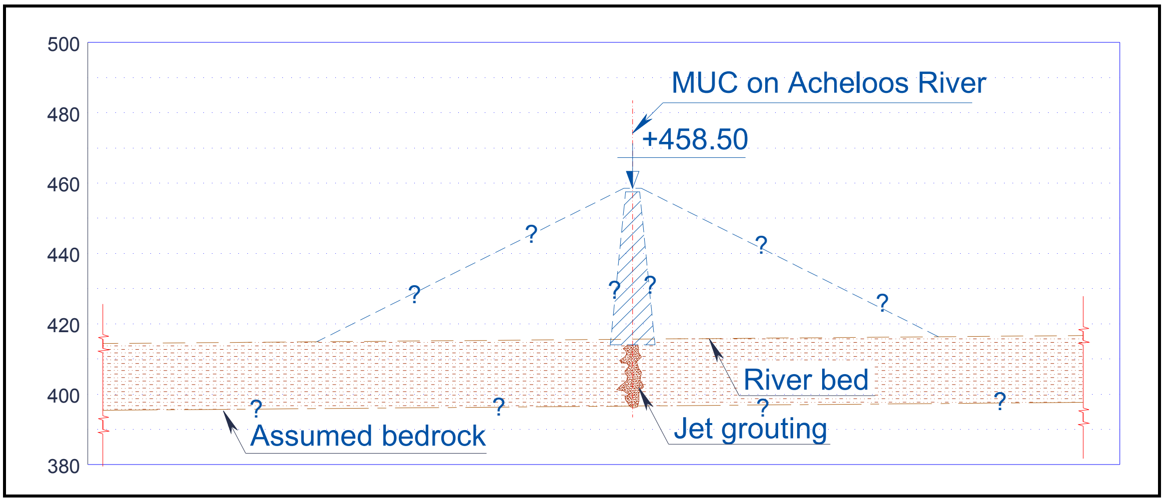


Fig. 4-5 – Typical Section of MUC on the Acheloos River

4.3.3 MUC on the Koubourjiannitikos River

The MUC on the Koubourjiannitikos River is also an embankment dam with a central impervious core. The cofferdam shells have been constructed with rockfill from the diversion tunnel excavations. The cofferdam crest has been formed at elevation +462.00. A typical section of the MUC is shown in Fig. 4-6. Unconfirmed information indicated that the faces of the cofferdam were constructed at a slope 2:1 (H:V), while the core faces at a slope 1:10 (H:V).

The MUC is founded on the river alluvia that were treated with one line of jet grouting holes, to reduce their permeability. Additional grouting was performed applying the tube a manchette method.

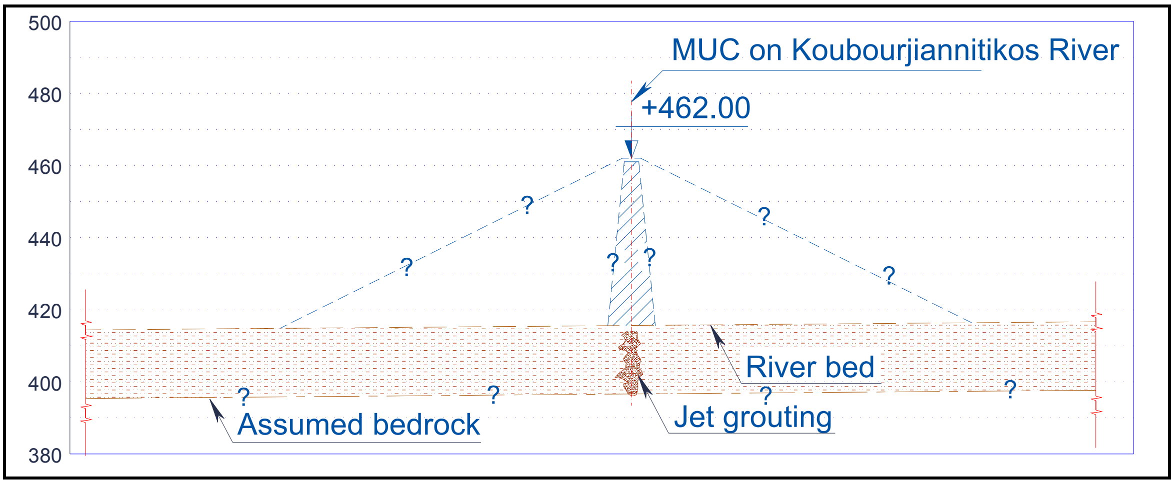


Fig. 4-6 - Typical Section of MUC on the Koubourjiannitikos River

4.3.4 Main D/S cofferdam

The main D/S cofferdam has a crest at elevation +423.60. Unconfirmed information indicates that the cofferdam has a central impervious core and shells made from rock excavation materials and river alluvia. Views of the U/S and D/S faces are shown in Photo. 4-6.

(a)

(b)

Photo. 4-6 – Downstream Cofferdam – (a) U/S view – (b) D/S view

Part of the cofferdam was eroded away during the 2009 flood (see Photo. 6-2) and was repaired at a later stage, as shown in Photo. 4-6 b.

4.4 Dam Shells

4.4.1 Introduction

Parts of both the U/S and D/S dam shells have been constructed by the dam contractor, in an aim to minimize the need of stockpiling excavation materials. Unconfirmed information raises the volume of the shell materials placed in the dam to be of the order of 1.5 hm^3.

4.4.2 Upstream Dam Shell

The design of the dam foresees that both MUCs shall be incorporated in the upstream shell of the dam. So, the dam contractor has placed considerable volume of shell materials mainly on the downstream face of the Acheloos MUC as shown in Fig. 4-7 (see also Photo. 4-7).

A typical cross section of the MUC and the shell materials on its downstream slope is shown in Fig. 4-7.

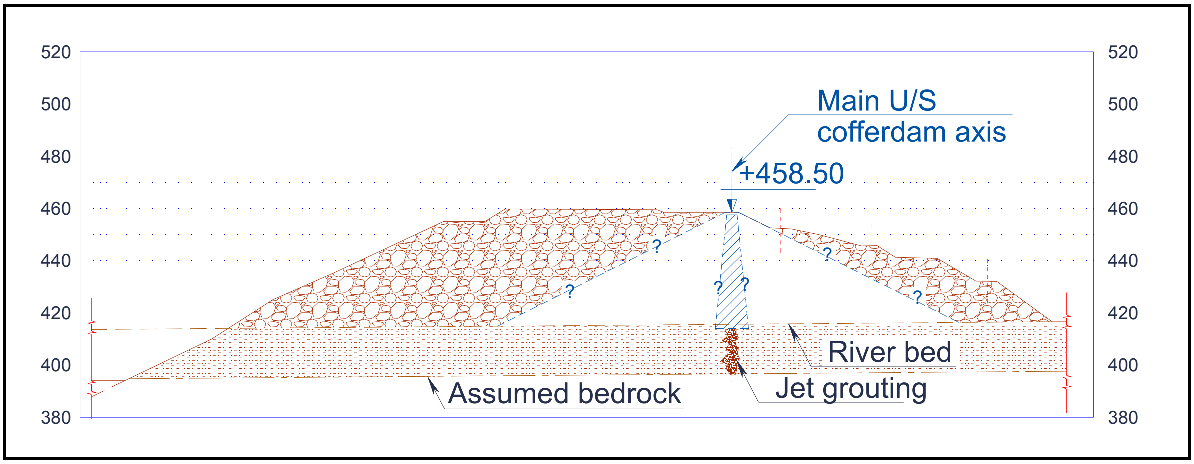


Fig. 4-7 - Typical Cross Section of Main Upstream Cofferdam on the Acheloos River

Photo. 4-7 - Upstream Dam Shell – View from D/S

A typical section along the Koubourjiannitikos River, crossing the MUC and the materials stockpiled on either side of the cofferdam is shown in Fig. 4-8. The section has been scanned from a drawing prepared by the dam contractor and given to the PoE during the site visit.

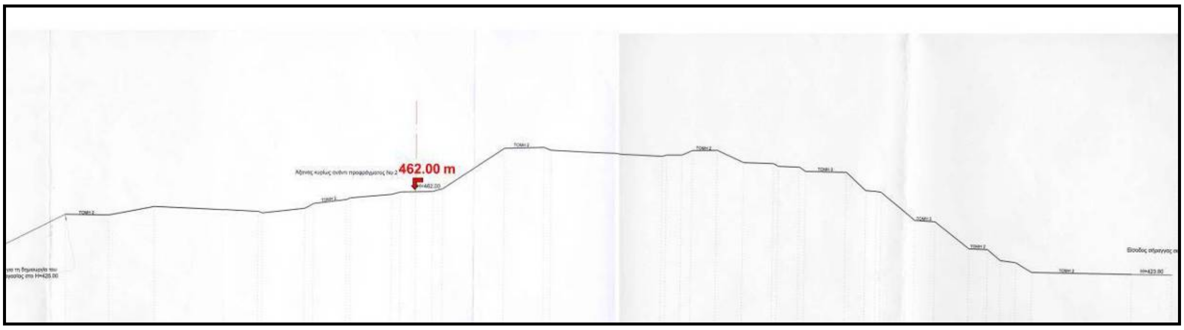


Fig. 4-8 - Cross Section of Koubourjiannitikos MUC and Upstream Dam Shell

4.4.3 Downstream Dam Shell

The dam contractor constructed part of the downstream dam shell, in-between the core trench excavation and the main D/S cofferdam. Views of the D/S shell are shown in Photo. 4-8 (a) and (b).

(a) (b)

Photo. 4-8 – Downstream dam shell – (a): U/S view – (b): D/S view

A typical section of the D/S dam shell along the river course is shown in Fig. 4-6 and

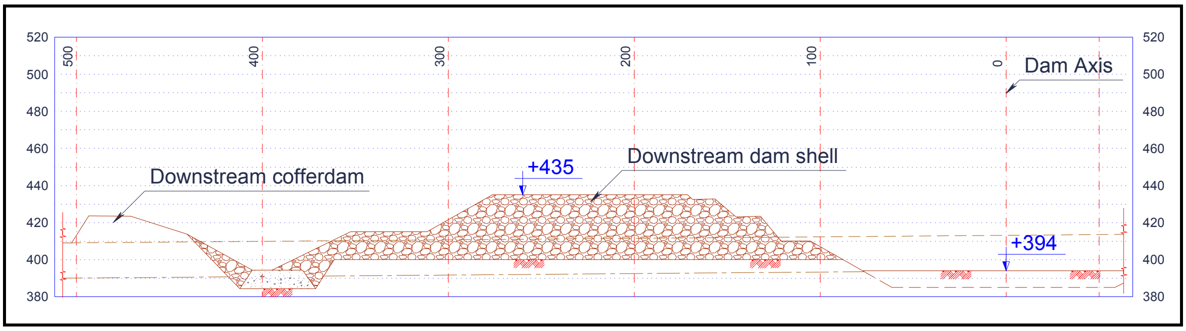


Fig. 4-9 - Downstream dam shell – Section along the river course

4.5 Diaphragm Wall

Prior to MUC construction on Acheloos and Koubourjiannitikos rivers, the alluvia in the valleys were treated with jet grouting to reduce their permeability. Jet grouting holes were drilled in one row along the cofferdam axes. The jet grouting holes were extended to the underlying bedrock.

Water-proofing of the Koubourjiannitikos cofferdam foundation was done by a combination of jet grouting and tube a manchette grouting.

Details of the grouting procedures followed and their results were not available to the PoE.

Nevertheless, during excavations in the river bed, significant filtrations were observed into the excavation areas, which indicated percolation of water through the alluvial under the MUCs (as shown diagrammatically in Fig. 4-8) and the lack of functionality of the water-tightening methods used (i.e. jet grouting and tube a manchette).

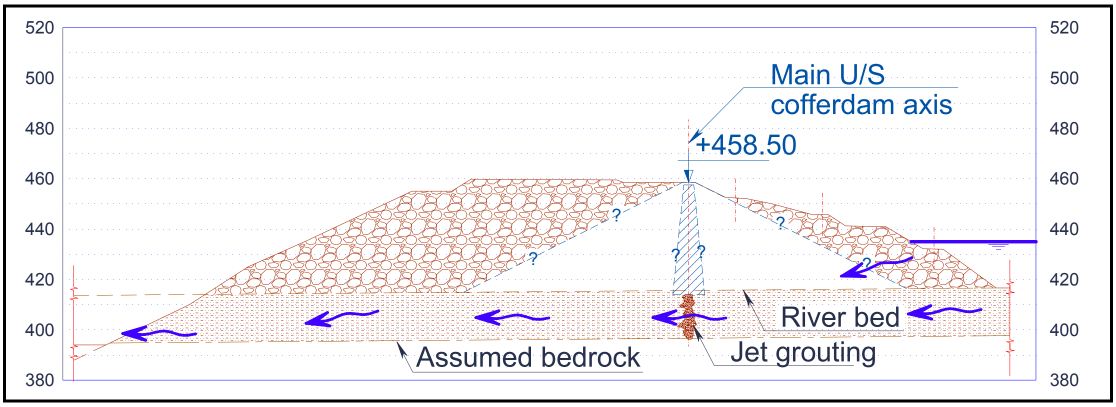


Fig. 4-10 - MUC – Filtrationd through river alluvia

The main downstream cofferdam behaved differently from the upstream cofferdam and there was no water filtering through its foundation or through the dam body. DAYE/PPC attributes that difference to the two lines of jet grouting holes that were constructed along the axis of the D/S cofferdam.

Water seeping through the alluvia was a problem:

a) for the construction of the dam and

b) because it could produce internal erosion of the alluvial of the foundation and piping through the body of the cofferdam that could result in cofferdam failure.

To deal with this problem various treatment methods were examined and analyzed in an aim to eliminate (or minimize) water circulation and the filtrations. DAYE/PPC examined various alternatives and produced a report in Nov. 2007, titled "Sykia Dam - Report on the treatment of leakages during core trench excavation".

Based on technical and economic evaluation of the various alternatives, it was finally decided to construct a positive concrete diaphragm wall. Originally the diaphragm wall was located upstream of the cofferdam axis, but due to extremely high bentonite slurry takes, the trench walls could not be stabilized. It was finally decided to construct the wall downstream of the cofferdam axis, from a berm at elevation +425, as shown Fig. 4-10. At the time this decision was taken, it seemed a proper decision, since construction of the dam core was eminent and it would be raised above elevation +425 in a relatively short period of time.

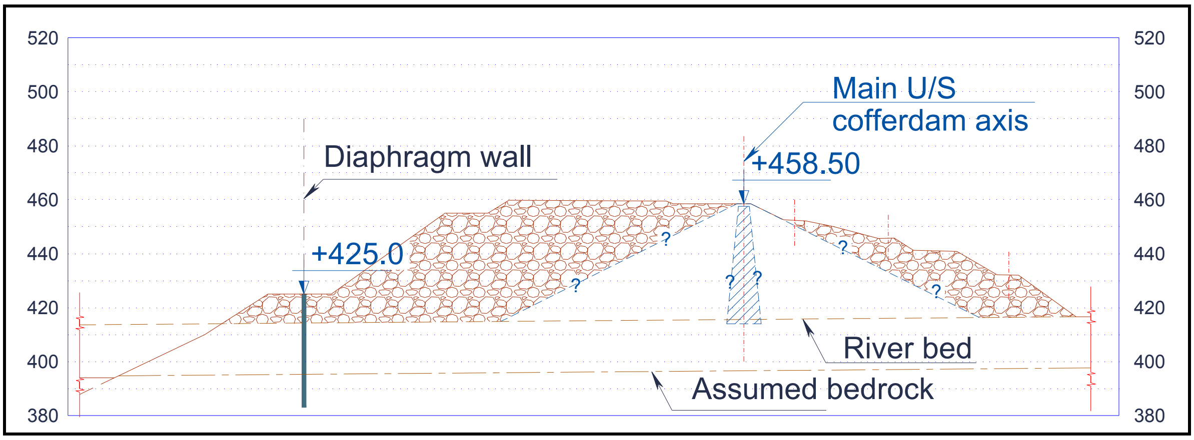


Fig. 4-11 – Section of U/S Dam Shell and Location of Diaphragm Wall

The diaphragm wall has a maximum height of 42 m and it penetrates ~2 m into the foundation rock. The total surface area of the wall is 5.670 m^2. The berm excavated on the D/S face of the dam shell is shown in the Photo. 4-9.

(a)

(b)

Photo. 4-9 – Berm on the D/S face of the dam shell and top of diaphragm Wall

4.6 Spillway

The spillway is located on the left abutment and consists of an approach channel, a gated ogee, three inclined chutes, an flip bucket and a plunge pool. The excavations of the approach channel have been completed, but no concrete structures have been constructed.

The excavations of the spillway chute and a large part of the concrete structures habe been completed, as can be seen in Photo. 4-10 a. The U/S end of the chutes is shown in Photo. 4-10 b.

There have been no works in the flip bucket area and the plunge pool.

(a)

(b)

Photo. 4-10 – Spillway – (a): Three Channel Chute, (b): Upstream View of Channels

4.7 Stockpiled Materials

Considerable volume of materials suitable for dam shell construction have been stockpiled on the Koubourjiannitikos MUC (see Photo. 4-11 a) and on the right bank downstream of the dam site (see Photo. 4-11 b). Unconfirmed information raises the volume of the stockpiled materials to ~500,000 m^3.


(a) (b)

Photo. 4-11 – Stockpiled materials – (a) on the Koubourjiannitikos MUC, (b) Downstream of the dam site, on the right bank

4.8 Dumped Materials

Materials unsuitable to be incorporated into the dam have been dumped on both banks downstream of the dam site. The materials dumped on the left bank are shown in Photo. 4-12.


(a) (b)

Photo. 4-12 - Dumped Materials – (a): D/S on the right bank, (b) D/S on both banks

4.9 Volume of Materials at Dam Site

The total volume of materials placed in the cofferdams and the dam shells, materials stockpiled and materials dumped at the dam site is roughly estimated at about 2 - 3 hm^3.

5. BASIC GEOLOGICAL DATA

5.1 Regional Geology

5.1.1 Geomorphology

The Sykia site is situated within the Pindos mountains, having a NNW to SSE structural trend and regional dip to the east. These are mainly composed of limestone, chert and shales of the Pindos geotectonic zone (Olonos –Pindos zone in the Geological map of Greece). Imbricate thrust units have been displaced to the west and override an area of predominantly flysch sedimentation of the Gavrovo geotectonic zone (Gavrovo zone in the Geological map of Greece).

The drainage pattern has been developed on the regional structure. Thus the Acheloos river is usually structurally controlled flowing south, parallel to the structure, or roughly west – east along joint or fault lines. Thrust masses have been eroded by the rivers to form steep V – shaped gorges, in contrast to the gentle and easily erodible flysch topography.

The present topography apparently began to develop during the Pleistocene when glaciers occupied the northern areas of the Pindos mountain. Subsequent erosion and later episodes of mountain uplift have resulted in stream rejuvenation and the formation of deep gorges. The final episode of uplift appears to have occurred after the rivers had established their grade and caused the river to be cut to a depth of about 20m below its present grade as is shown by the depth of gravel deposits in the Acheloos river.

Upstream from the Sykia dam site (river el. ~+410) and for a distance of ~10 km, (reaching el. ~+460m), Acheloos R flows along a sinuous course in-between two major land massifs, which form the right and left banks of the river valley. At the dam site, a major tributary, Koubourjiannitikos R, flows into the Acheloos R from the left bank (see Fig. 5-1).

It is pointed out that along this narrow valley section considerable amount of loose talus debris occur.

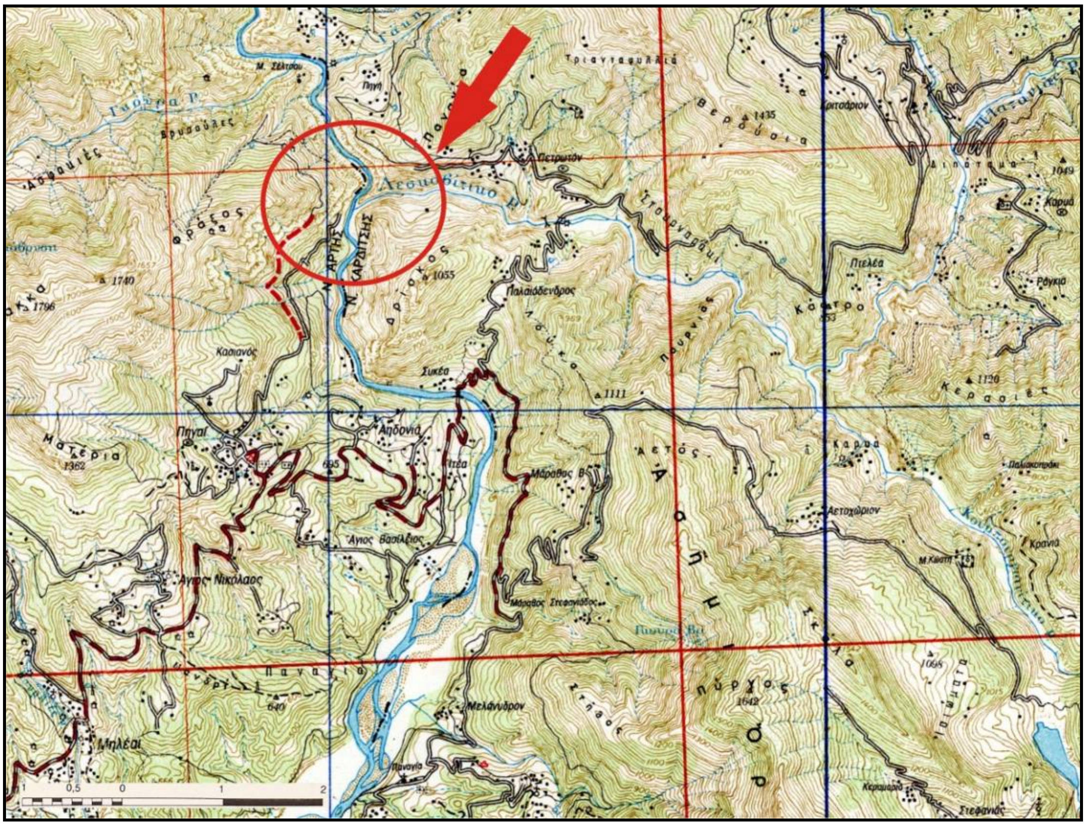


Fig. 5-1. Topografic map of damsite and its downstream areas

(copied from 1:50.000 scale map)

5.1.2 Stratigraphy

In more detail a cross section of the area, from east to west, indicates a sequence of rock formations of Olonos –Pindos zone with internal thrusts, which overthrusts on the flysch of Gavrovo zone (see Geological map of Greece, Mirofilon sheet, 1:50000 at scale as well as the E-W cross section - Fig. 5-2 and Fig. 5-3).

From the above formations those of more interest for the Sykia damsite are the Upper Cretaceous limestones and the underlying Middle Jurassic – lower Cretaceous formations (Radiolarite series).

Cretaceous limestone (No7, in the legend of Fig. 5-2*)*

They consist principally of pink and grey pelagic limestone occasionally interbedded with red chert and shales. The contacts with both the older Middle Jurassic – Lower Cretaceous formations (radiolarite series) and the younger flysch are gradational. The thickness of this zone is variable.

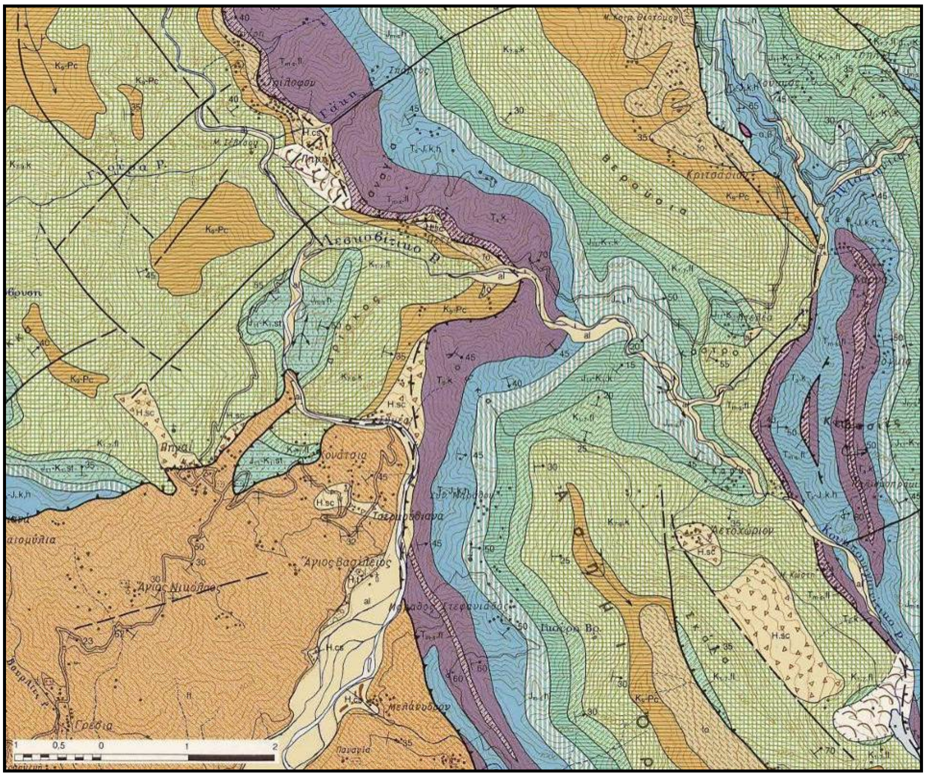
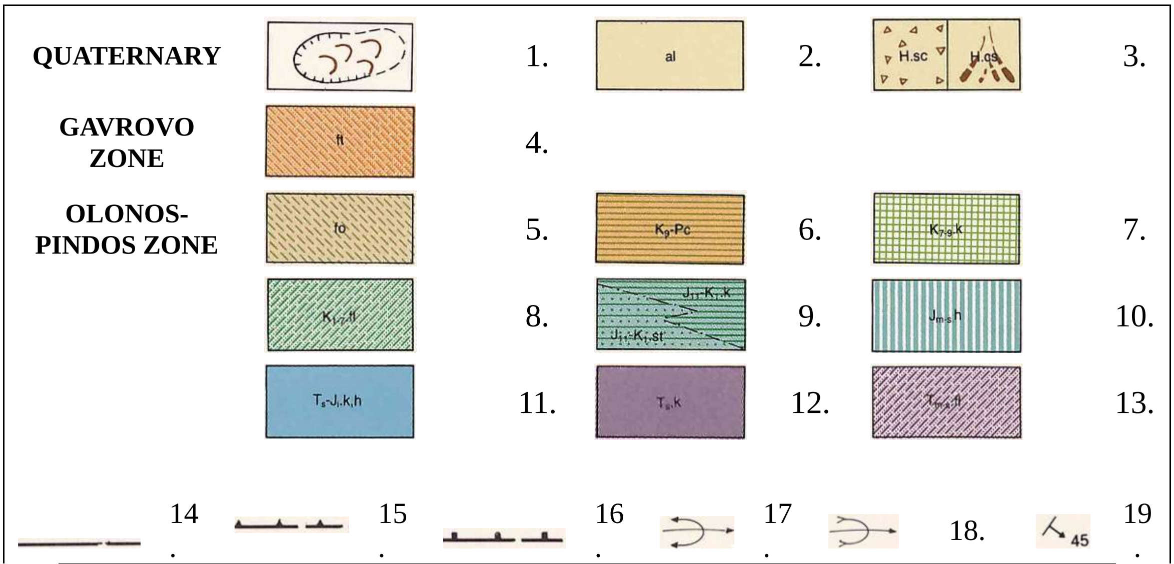


Fig. 5-2. Area geological map - IGME (Mirofillon sheet. 1993) 1:50.000.

LEGEND



1. Landslides and creeping	2. Allouvial deposits	3.Scree and talus cones
4. Flysch indivited	5. Flysch	6. Transition beds
7. Upper Cretaceous	8. First flysch	9. Limestones with
10. Multicoloured cherts	11. Limestones and cherts	12. Upper Triassic
13. Clastic Formation	14. Fault	15. Overthrust
16. Thrust	17. Recumbent anticlinal	18. Recumbent synclinal
19. Strike and dip of beds		

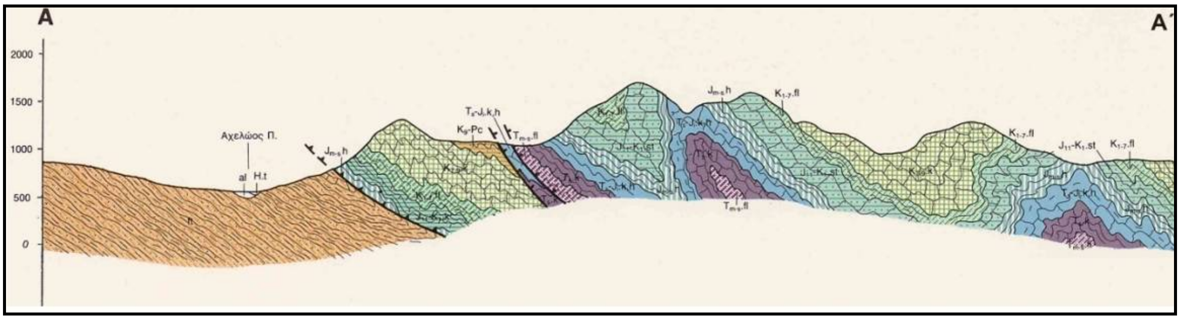


Fig. 5-3. Geological section of the wider area, from east to west.

The Cretaceous limestones are generally observed as thin to medium bedded (5 to 50 cm, on average 20 cm) grey or pink lithographic limestone. In the lower part of the formation, near the contact with the radiolarite series, thinly bedded red limestones, red shales and argillites are interbedded with pink limestone. In the middle of the formation nodules, or occasional lenses and thin beds of red chert are interbedded with pink and grey limestones. Near the transition zone between the Cretaceous limestones and flysch formation, nodules and lenses of black chert are abundant.

The limestone, although extremely folded, is generally not as fractured as the Jurassic limestone which has a greater abundance of silica and chert beds. Zones of thickly bedded Cretaceous limestone are prone to solution and development of karst. In general Pindos limestone shows advanced karstic erosion near the surface but not in depth. Particularly, for the Sykia site, the karst is limited, the limestone is strong and its permeability moderate, since the thinly bedded limestones are intercalated with red siliceous layers, dipping generally 60˚-70˚ upstream. Finally, the thickness of the limestone is over 200m.

Middle Jurassic – Lower Cretaceous formations (Radiolarite series) (No 8, 9 and 10, in the legend of Fig. 5-2*)*

These consist of argillite, shales, limestones and cherts. The cherts are medium bedded (5 to 30 cm, on average 10 cm) and vary from dark brown or red to purple or green. Since these rocks are almost entirely silica, they are hard and brittle and, as a result, highly jointed.

The limestone is thin to medium bedded (5 to 30 cm, on average 10 cm) grey or red – grey. Interbedded with the limestone are thin (1 to 3 cm) beds of red shale. Nodules and interbeds of chert are also present.

Locally the limestone may be extensively folded. Where chert is abundant the rock may be highly fractured and broken, while where the pure limestone and interbedded argillite predominate, there may be little fracturing.

The upper horizons of this series (First Flysch in the Geological map) consist of alternations of thin layers of red marls, cherts, marly limestones and clayey marly schists. In the upper members occur coarse–grained sandstones.

5.1.3 Structural geology

Regionally the dam site and the reservoir area are within the structural province of Pindos mountains.

The oldest sediments were deposited in a north – south geosynclinal basin that existed during the Upper Triassic, Jurassic and Cretaceous. This phase of marine sedimentation probably ended during Eocene as orogenic processes began with thrusting of the Pindos sediments. Thrusting continued during Oligocene, forming thus the Pindos mountains which are characterized by north – south ridges.

The stratigraphic sequence is thickened by the repetition due to isoclinal folding. Layers of sheared clay have been formed along the bedding planes derived from clay–shale by the compressive forces during the folding.

Concerning Sykia reservoir area is bounded by regional thrust faults on the east and west. The rock formation in the block between the two thrust faults are intensely folded, particularly the Cretaceous limestone formations. Generally the rocks are folded into rather gentle anticlinoria – synclinoria, but within the framework of these large structures the folding is very intense and tight with steeply plunging fold axes, usually plunging to the west.

So, the whole reservoir is in limestone, which is intensely folded. The fold axis is in NNW – SSE direction. The bedrock is mostly tight and with less evidence of karstic development. Faulting is common but of small scale and with small displacements.

The transition upwards from Radiolarite series to Cretaceous limestone is gradual. The limestones immediately above are thinly bedded (beds average 10 cm), intensely folded and broken, although tight.

Permeability tests made at the damsite indicate, in general, low rock permeability.

5.2 Damsite Geology

5.2.1 Rock types

The predominant rock of the damsite area is Cretaceous limestone, extending over practically the whole core trench on both abutments (see Photo. 5-1).

(a) (b)

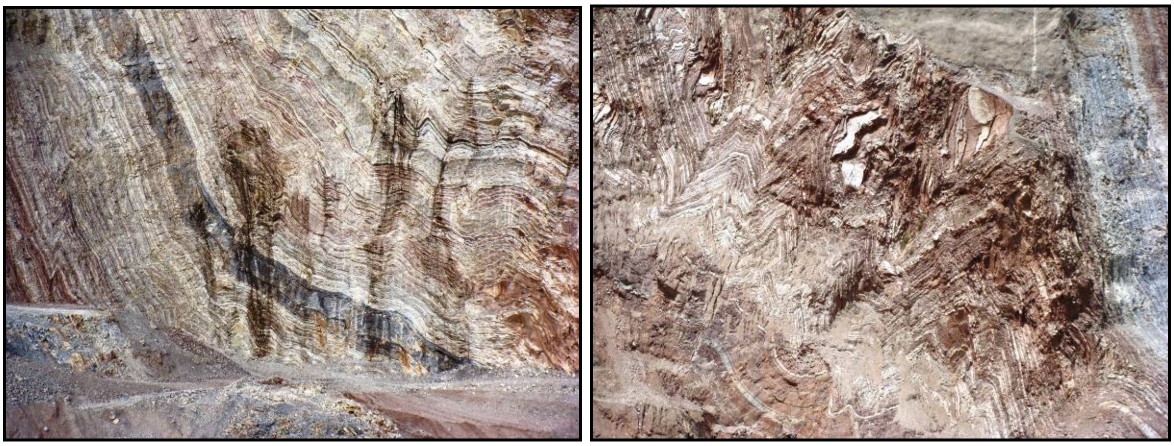
Photo. 5-1 - Upper Cretaceous limestone on (a): right bank and (b) left bank

The limestone is thinly to moderately bedded (beds 2-15 cm thick), the thin beds dominating (2-10 cm range), although beds up to 25 cm thick have been encountered. The limestone occurs in alternating zones of predominantly red (pink to purple) and predominantly grey (light to dark), the latter usually containing little or no red colored limestone, whereas the red colored zones always contain some grey (see Photo. 5-2 a).

Because of the intense folding of the area (see Photo. 5-2 b), even the average thickness of the zones is difficult to determine, but it is believed to range from 15 to 35 m.

On the left part of Photo. 5-1 (a), the green sandstones of the First Flysch are observed, representing the upper horizons of the Radiolarite Series.

These limestones overlie the Radiolarite series of rocks which outcrop downstream of the river junction and which include red limestone, red shale, mudstone, sandstone, red chert and sheared shale. Because of the relative difference in competence and resistance between limestone and Radiolarite series of rocks, the contact is well defined as a topographic flare in the valley below the river junction (see Photo. 5-3 and Photo. 5-4).



(a) (b)

Photo. 5-2 - Upper Cretaceous limestone - Bedding and Folding

Photo. 5-3 - The First flysch with the green sandstones and the underlying members of the Radiolarite Series.

The limestone, grey as well as red, is usually fine grained although in the grey zones coarse grained to lithographic varieties have been encountered. The grey zones also tend to contain occasional thick beds (25 cm or more), whereas the red zones are uniformly thinly bedded. Solution action, although minor, in the form of leached or rugged joints and fractures, is more common in grey than in red limestone.

Photo. 5-4 - Green sandstones of the First flysch (upper horizons of Radiolarite Series) and the transition to the Upper Cretaceous limestones (left bank).

Interbeds of chert and partings of sheared shale occur with regularity throughout the site rock formations. Partings of sheared shale, represent thin beds of shale, now sheared and slicken sided, and occasionally decomposed to clay, the shearing caused by tight folding and slipping of one bed over the other.

Red sheared shale and clay partings, rarely up to 5 cm thick, occur in the red limestone zones, whereas the grey sheared shales and clay seams occur in the grey zones. Similarly, red chert is associated with red limestone, and grey chert with grey limestone (there are exceptions). In the red limestone zones the percentage of sheared shale and clay is higher than in the grey limestone zones.

Chert content is rather low in the red as well as the grey limestone zones, ranging from 5-9%. It occurs in beds rarely exceeding in thickness 15 cm.

5.2.2 Folding and joint trends

The structure of the site is complicated due to the intense, small scale folding and rugged topographic features.

The tetrahedral shaped landmass bounded by the Acheloos to the west and south and Koumporianitikos to the north comprises an easterly plunging anticline (see Geological map of Greece). The Acheloos cuts through its axis, thus exposing its core. The orientation of the bedding at the site is highly variable because of small scale folding, and ranges generally from N85˚W (E-W) to N30˚W, although in folded sections north – easterly strikes are not uncommon. The dips range from vertical to 30˚ in a northerly direction and an east – west to northwest variation of bedding trend has been indicated.

In the outcrops of the area four prominent joint trends have been revealed:

1) Set ˝a˝ - Bedding plane joints, strike N60˚-85˚W (275˚-300˚) dip 45˚-70˚NE.
2) Set ˝b˝ - Joint set strike N15˚-35˚E (015-035) dip 45˚-80˚NW.
3) Set ˝c˝ - Joint set strike N5˚-45˚W (315-355) dip 30˚-50˚SW (occasional steep joints dip up to 80 degrees).
4) Set ˝d˝ - Joint set strike N60˚E, dip 60˚SE.

In set ˝a˝ we have continuous planar joints, filled with clay, often sheared and slickensided (sheared clay up to 5 cm).

In set ˝b˝ continuity 5-10 m, sub-parallel to river, in-filled with calcite. Set 'b' may create unstable conditions on the left abutment.

In set ˝c˝ major joints up to 20m, strongly developed. Minor joints less than 2m continuity. Thrust shear zones often with clay and evidence of movement up to 30cm, spacing 3-4m, minor 0.15-2m, faults spaced at about 10m vertically. It is the most significant joint set relative to the stability of the dam abutments.

In set ˝d˝ we have a poorly defined system, open or calcite filled. Joints are probably stress relief, seen along the gorge wall.

5.2.3 Rock quality of foundations

The quality of the foundation bedrock is controlled by 3 factors:

1) Percent clay in the limestone layers
2) Prominent joint trends
3) Degree of weathering and relaxation.

The folded limestone beds are generally thinly bedded and about 2-10cm thick. The beds have a regular upstream dip of 40˚ to 60˚ to the north. Sheared clay layers occur and the percentage of sheared clay – shale is 1 to 2.5 in the grey limestone and 1.5 to 2.5 in the red limestone. Thus, rock quality is not seriously decreased by the clay layers.

Prominent joint trends are as follows:

Set ˝a˝ - folded bedding plane joints

Set ˝b˝ - stress relief joints parallel to gorge walls

Set ˝c˝ - normal to bedding, joints with downstream dip are significant to stability

Set ˝d˝ - sub-parallel to gorge walls, minor.

Their spacing, continuity, openness and filling are of significance to the rock quality, as described before.

Weathering in the limestone generally only takes place to shallow depths and along joint planes. However, poor zones and open joints on the left abutment have been encountered. Relaxation of stress relief joints, paralleled to cliffs which form the gorge walls, are especially noted at higher elevations on the abutments. However, the ˝b˝ joint set is less well developed than at many sites.

In general rock quality is good as far as it concerns the Upper Cretaceous limestones and the sound rock occurs beyond the effects of the open stress relief joints. But the dam is expected to be founded in its southern part also in the underlying of these limestones Radiolarite series and especially in the formations of First flysch (alternations of thin layers of red marls, cherts, marly limestones and clayey – marly schists as well as sandstones in the upper members). These radiolarite horizons are susceptible to surface weathering and erosional effects.

5.3 Landslide Phenomena

Major landslide phenomena, that may affect the wider damsite area, occur in the area of “Piges” village, located on the right bank, downsteam of the dam site, as shown on Fig. 5-4.

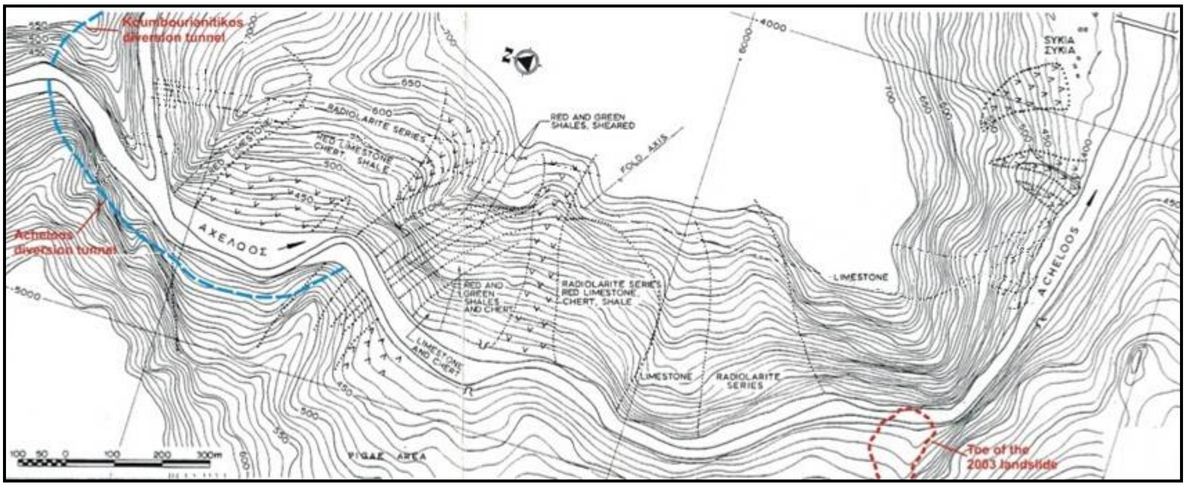


Fig. 5-4 - Map of the damsite's area, also showing the geology (PPC, 1972).

On the 15th of January 2003 a landslide occurred on the right bank, downslope of "Fryxolia Spring". The landslide was a mudflow flow in the flysch of Gavrovo zone including debris, which blocked Acheloos River (see Photo. 5-5 a) and flooded the upstream areas, forming a 25 m deep lake (see Photo. 5-5 b).

(a) (b)

Photo. 5-5 - View of the landslide in 2003 and flooded area.

On 19-1-2003 surface ruptures were identified at the north of the previous landslide in a geologically sensitive zone, where instability phenomena are taking place for the last 200 years, as very slow movements of detached limestone masses and scree materials over the flysch. The above mass movements were re-activated by heavy rainfall in the period of December 2002 and January 2003.

A geological section across the landslide is shown in Fig. 5-5 and a picture of the whole area is shown in Photo. 5-6.

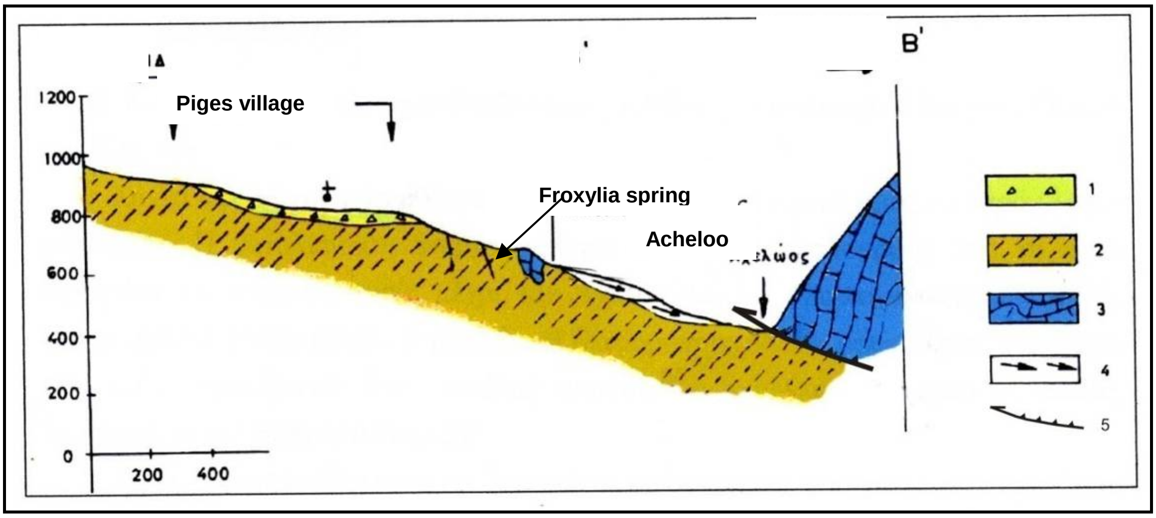


Fig. 5-5 - Geological section of the landslide

(1. scree, 2. Gavrovo flysch, 3. limestones of Pindos zone, 4. landslide, 5. overthrust)

Photo. 5-6 - View of the downstream area. The exit of Acheloos diversion tunnel is shown (blue arrow), as well as Piges village and the 2003 landslide at the background (red arrow).

A picture of the landslide taken in 2006 is shown in Photo. 5-7.

Photo. 5-7 - View of the landslide in 2006.

In general, the factors that contribute in the activation of these phenomena are the geological composition, structure, intense morphological relief, hydro-geological conditions and intense rainfall. Earthquakes can also trigger instability phenomena, especially when combined with the above mentioned preparatory instability factors.

The above landslides are considered as active and are likely to be reactivated. They are monitored by geodetic measurements.

Peak discharge rates in Acheloos River cannot activate these instability zones, since they are located far enough from the flow channel and at a higher elevation. Only in the worst case of the destruction of the existing works and the formation of a big lake, slope stability can be affected. On the contrary, peak flood rates can activate flysch slopes downslope of “Aidonia” village in the case of blocking “Korakou” Bridge. In these slopes, small scale rotational slides have already occurred but they are not connected with the river. The downstream area does not face any instability problems induced by high flow rates of the river due to gentle slopes of the flysch and a wider flow channel.

In the case of the worst scenario due to an unusual flood event which could not be managed by the existing structures of Sykia the safety could be put at risk for “Korakou” Bridge, the villages “Sykia”, “Aidonia”, “Agios Vasileios”, “Marathos” and the smaller settlements founded on the river banks at the downstream area. Also, in a longer distance Aylaki bridge, the small hydro-electric “Dafnozonara” Plant as well as “Templa” Bridge near the Kremasta reservoir.

5.4 Seismicity

According to the Greek Seismic Code (EAK 2003) the study area belongs in Zone II, in which the coefficient of horizontal ground acceleration is taken equal to a= 0.24 (A=a x g). The magnitudes of the maximum recorded earthquakes have reached 6.5 to 6.9 of Richter scale. A disastrous earthquake, having a magnitude 6.5, hit the Upper Acheloos area on May 1, 1967. Maximum intensity of IX (Mercalli –Sieberg scale) was assigned at a village which is located 10km away from Messochora site. Also, two additional earthquakes have been recorded in February and October of 1966, of magnitude 6.4 and 6 and of intensity IX and VIII respectively. These events were located 40 to 50km away from the nearest Upper Acheloos Project.

In considering the design of the Sykia dam, some potentially damaging factors, due to earthquake shocks, have as follow:

a) Movement along a fault. In the Upper Acheloos area, displacements of structures due to fault movement are considered as unlikely.

b) Elevation changes resulting from seismic activity are also considered to be unlikely in this area.

c) Landslide activation by seismic activity is considered to be a hazard in the Upper Acheloos area. It should be anticipated that future earthquakes can cause landslides on marginally safe slopes and activate existing slides.

d) Structures founded on bedrock, as in the case of Sykia, are not expected to experience settlements. Particularly, foundation on limestone bedrock is considered as much favorable. The same applies to the formations of Radiolarite Series.

5.5 Diversion Tunnels

5.5.1 Acheloos Diversion Tunnel

Diversion Tunnel No. 1 has been excavated over a length of ~650 m from the inlet portal, in the Upper Cretaceous Limestone (see Photo. 5-8 a). The rockmass exposed in the tunnel has been considered as competent, while the bedding is dipping favorably and nearly parallel to the tunnel's axis, which ensures additional stability of the underground opening. Only in narrow zones of intense folding the limestone rockmass is considered to be of lower mechanical properties. The rockmass along that length of tunnel may be considered as of high resistance to erosion by water flow.

The next 250 m of the tunnel have been excavated in the Upper parts of the Radiolarite Series, which in the Geological Map is referred as "First flysch". The Radiolarite Series consist of reddish to greenish sandstones as well as successions of thin red marls, cherts, marly limestones, shales and mudstones (see Photo. 5-8 b). This rockmass has unvaried geometry, similar to the overlying Cretaceous Limestones. In the tunnel, these formations have been exposed as fresh rockmass, without signs of weathering, as it is experienced on the surface. The rockmass along that length of tunnel may be considered as of low resistance to erosion by water flow.

The last 50 m of tunnel have been excavated in thinly bedded limestones of the Radiolarite Series (see Photo. 5-8 c), which are fresh and with the same geometry as above. The rockmass along that length of tunnel may be considered as of medium resistance to erosion by water flow.

It is recorded that during both visits by panel members to the site, the water immerging from the tunnel was completely clean.

(c) (b) (a)

Photo. 5-8 - Formations along the Acheloos Diversion Tunnel

5.5.2 Koubourjiannitikos Diversion Tunnel

The 602 meters long DT-2 is entirely hosted in the Upper Cretaceous limestone. These formations are characterized by the well defined bedding, oriented favorably to the tunnel's axis. Under these geological conditions, detachments and falls of rock blocks are quite unlikely to happen, especially in tunnels.

5.5.3 Additional Diversion Tunnel

This tunnel is mainly driven through limestones of the Radiolarite Series.

5.6 Conclusions

Taking into consideration the existing data and the site observations the following concluding remarks can be stated:

1) The geological formations exposed in the foundation of the dam are primarily Upper Cretaceous limestone in the north and, on a much smaller extent, the lower Rariolarite Series at the south. Both formations comprise a continuous sedimentary sequence which retains its structure and favorable bedding geometry. Bedding generally deeps intensely to the North. Limestone and Radiolarites are locally heavily folded and intensively fractured thus forming rockmass zones with poor mechanical properties, but without any significant change in the general physical state of the rockmass.

   The founding area of the dam's core trench is completely composed by lithofaces of the Upper Cretaceous limestone. The physical state of these formations has not experienced any change during the last 10-15 years after the excavations or any signs of instability.

   In the whole dam site's area global instability phenomena are not expected in the natural and man-made slopes and excavations, except some detachments and falls of small rock blocks and, in some cases, of small limestone rock masses. These instability phenomena can be caused in sites where intense folding and jointing can induce planar or wedge failures. Besides, the susceptibility of Radiolarites Series in weathering processes can result to the development of loose zones, especially on surface, while at the upper horizons, where sandstones mainly dominate, local falls can occur.

   This general view of the dam site's foundation zone and the surrounding slopes is unlikely to be changed in the next 5-10 years, given the fact that new excavations are not expected, especially inside the Rariolarite Series.

2) The above geological formations are characterized by moderate water permeability.

3) Landslide phenomena are not recorded in the dam's site but at the area of "Piges" village which is located at the downstream area. Movements in these slopes are old but the area was reactivated in 2003, after intense and prolonged rainfalls. These landslides were mudflows inside the flysch including scree materials, causing closure of the river and

formation of a lake. It is noted that the reactivation of these landslides is not connected with river bank erosion or peak flow rates. They were triggered by heavy rainfalls and influenced also by other landslide preparatory factors. Extensive reactivations in the future can happen influencing the river's flow regime.

Peak flow rates can cause problems in the case of blocking of the flow at "Korakou" bridge and with activation of the susceptible flysch slopes downslope of "Aidonia" village. On the contrary, the downstream area has gentle slopes of flysch without the possibility of causing instability problems.

4) According to the Greek Seismic Code (EAK 2003) the study area belongs in Zone II, in which the coefficient of horizontal ground acceleration is taken equal to a= 0.24 (A=a x g). The magnitude of the maximum recorded earthquakes has reached 6.5 to 6.9 on the Richter scale. Regarding seismicity, the stability of the slopes within the project area are expected without any problems. Only activation of landslides triggered by big earthquakes is considered to be a hazard. It is certain that future earthquakes could activate landslides in the wider landslide-prone areas of "Piges" village.

5) In the case of s peak flood event, which could not be managed by the existing structures of Sykia dam site, the safety could be put at risk for "Korakou" bridge, the villages "Sykia", "Aidonia", "Agios Vasileios", "Marathos" and the smaller settlements founded on the river banks at the downstream area. Also, in a longer distance Aylaki bridge, the small hydro-electric "Dafnozonara" Plant as well as "Templa" bridge close to Kremasta reservoir.

6) Regarding the Koubourjiannitikos river diversion tunnel it is noted that has been completely hosted in Upper Cretaceous limestones, which are characterized as a very sound bedrock without the potential of falls and erosion problems. The main diversion tunnel of Acheloos river has been also opened in sound Upper Cretaceous limestones with a smaller part in the Radiolarite Series. Problems of detachments and falls are not expected in both lithofacies, but it is noted that erosion can be favored in Radiolarites in case that lining is absent.

5.7 References

- Public Power Corporation, Greece, 1972. Upper Acheloos Hydroelectric Development, Sykia Engineering Report – Geology Report. Surveyer, Nenniger and Chénevert INC. Consultants, Montreal.

- Barbier, R., 1965. Rapport Géologique Préliminaire sur les Sites Possibles à l' amont d' Avlaki. Report to PPC, 15 pages.
- Karageorgiou, E., 1966. Geological Conditions and mapping along the Acheloos river between Glystra and Avlaki sites. Report to PPC, 15 pages with photos and geological map 1:25000, Athens.
- Geological Map of Greece, 1993. Mirofillon sheet, 1:50000. Institute of Geology and Mineral Exploration, Athens.
- Nikolaou, N. and Paschos, P., 2003. Engineering geological study of landslide phenomena in Piges village, Arta Prefecture. Institute of Geology and Mineral Exploration, Athens, 19p.

6. POTENTIAL HAZARDS ON CONSTRUCTED WORKS

6.1 Introduction

The works of Sykia project that have been completed prior to the abrupt interruption, are subject to five potential hazards: a) seismic activity, b) instability of excavations, c) overtopping of cofferdams caused by floods, d) internal erosion of cofferdams and e) failure of diversion tunnels structures.

These potential hazards are reviewed by the PoE in the following paragraphs.

6.2 Earthquakes

Regarding potential hazards due to earthquakes, the PoE believes that earthquakes do not constitute an important hazard to the project, since active faults have not been mapped within the narrow dam site area and, furthermore, major earthquakes (above Ms=6) have not been recorded within a range of 40-50 km from the dam site So, structures founded on sound bedrock are not expected to experience any settlement due to an earthquake. Earthquakes may trigger landslides in the wider landslide-prone areas of “Piges” village, but without any effect on the Sykia dam structures.

Concluding, the PoE is of the opinion that earthquakes do not constitute a hazard to the project.

6.3 Stability of Excavations

The man-made slopes in the broader dam site area have been exposed for a considerable number of years since their excavation. The slopes above dam crest elevation on the right abutment are protected locally by shotcrete and support measures. The slopes lower than the dam crest on both abutments have no support measures. Instability phenomena have not been recorded and are not expected in the natural and man-made slopes and excavations, except some detachments and falls of small rock blocks and, in some cases, of small limestone rock masses.

This general view of the dam site’s foundation zone and the surrounding slopes is unlikely to be changed in the next 5-10 years, given the fact that new excavations are not expected, especially in the Rariolarite Series.

Concluding, the PoE is of the opinion that instability of excavations does not constitute a hazard to the project.

6.4 Floods - Hydrological Safety of Cofferdams

6.4.1 Introduction

Acheloos River has a total catchment area of 1,173 km^2 at the Sykia dam site, i.e. 944 km^2 in the Acheloos branch and 229 km^2 in the Koubourjiannitikos branch. The mean annual rainfall in the catchment area is ~2,000 mm and the mean annual runoff is 44 m^3/sec which means an annual runoff of 1,386 hm^3/year.

Floods usually take place from November to February. Annual peak flows for the time period 1965-1988 are shown on Table 6-1 and in a graphic form in Fig. 6-1.

Table 6-1. Annual Peak Flow – m^3/sec. 1965-1988

YEAR	Annual Peak Flow.	Month
1965	443	nov
1966	878	jan
1967	431	jan
1968	676	jan
1969	691	dec
1970	1339	dec
1971	416	mar
1972	401	mar
1973	470	dec
1974	673	nov
1975	574	dec
1976	993	dec
1977	180	jan
1978	815	feb
1979	552	apr
1980	559	mar
1981	797	dec
1982	746	nov
1983	240	nov
1984	551	jan
1985	609	jan
1986	455	feb
1987	558	feb

1988	527	dec

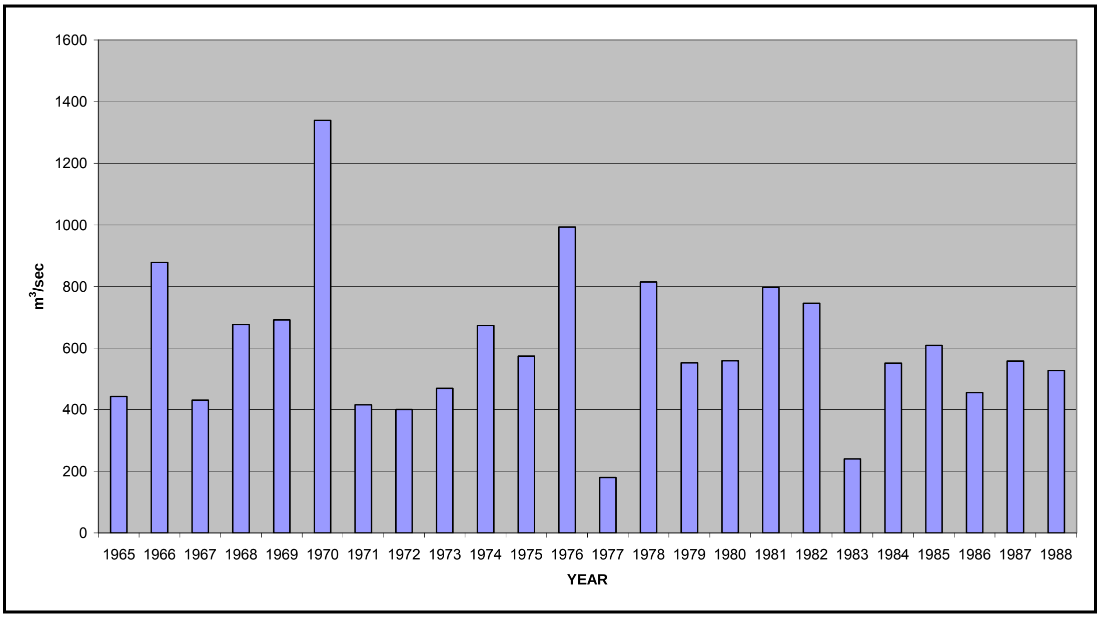


Fig. 6-1. Peak flows - 1965-1988

Peak flows versus the corresponding return period of the event are shown in Fig. 6-2.

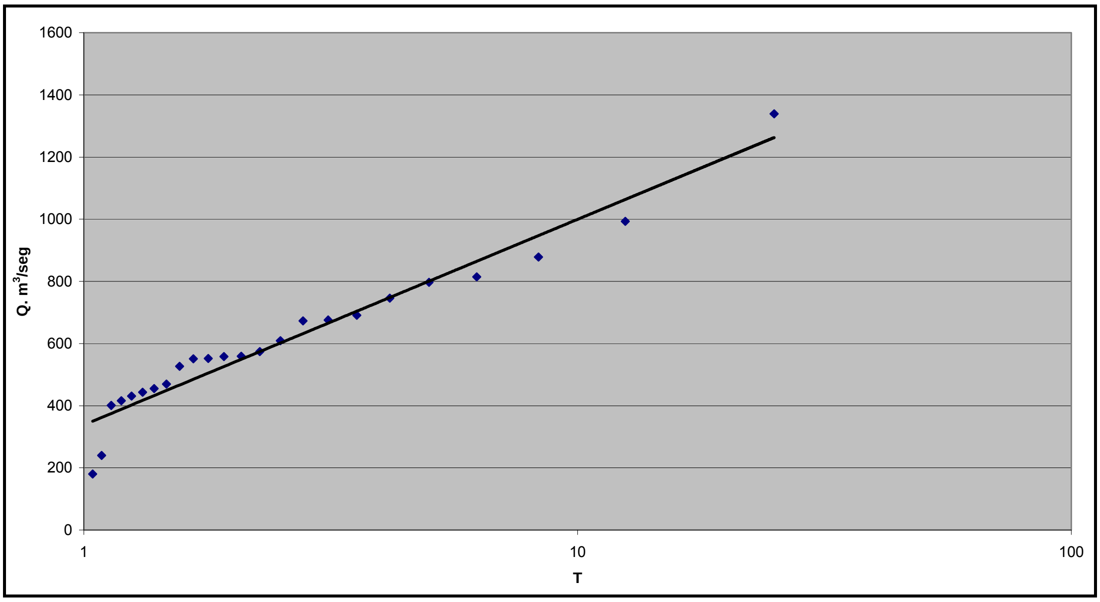


Fig. 6-2. Peak flows vs return period - Floods from 1965 to1988

In the last few years two significant floods have taken place, in December 2005 and in November - December 2009. Photo. 6-1 and Photo. 6-2 show details of the 2009 flood at the inlet and outlet portals of DT-1.

Photo. 6-1 - 2009 Flood: Intake of Acheloos Diversion Tunnel

Photo. 6-2 – 2009 Flood: Outlet of Acheloos Diversion Tunnel – Erosion of cofferdam

The presence of floating debris can be observed at the intake of the Koubourjiannitikos Diversion Tunnel (see Photo. 6-3), due to the geomorphologic characteristics of the basin, as well as the reflux erosion at the exit of the Acheloos Diversion Tunnel (see Photo. 6-2).

The data of these floods are shown in Table 6-2. 2005 and 2009 flood characteristics, with peak flows of 1.175, and 1.100 m^3 / sec for the floods of 2005 and 2009 respectively.

Table 6-2. 2005 and 2009 flood characteristics

Flood	Intake Water Level (m)	Peak Flow (m^3/sec)	Return Period (years)

Dec.2005	+434	1.175	8,3
Nov-Dec 2009	+432	1.100	6,2

Photo. 6-3 - 2009 Flood: Intake of Koubourjiannitikos Diversion Tunnel

From the available data it is deduced that the maximum flood observed during the last decades it was in December 1970, with a peak flow of 1.339 m^3 / sec (see Fig. 6-3) for a basin area of 1.173 Km^2, which corresponds to a specific flow of 1.14 m^3 / sec / Km^2.

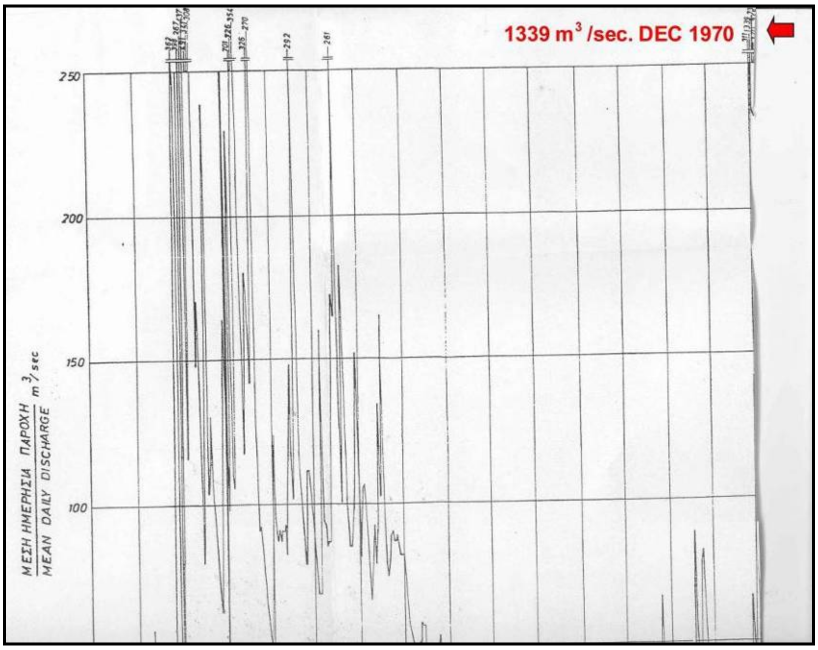


Fig. 6-3 – Recorded river flows in 1970

Comparison of the 1970 flood of the Acheloos River at Sykia dam site with the extreme floods observed in Europe are shown in Fig. 6-4. Acheloos River has a coefficient of Francou-Rodier of 4.17, while the extreme envelope curve in Europe has a coefficient of

5.61. This represents that, as a first assessment, the floods of the Acheloos River are in order of magnitude of 20% of the maximum peak floods in Europe, for a basin area of 1.173 km^2.

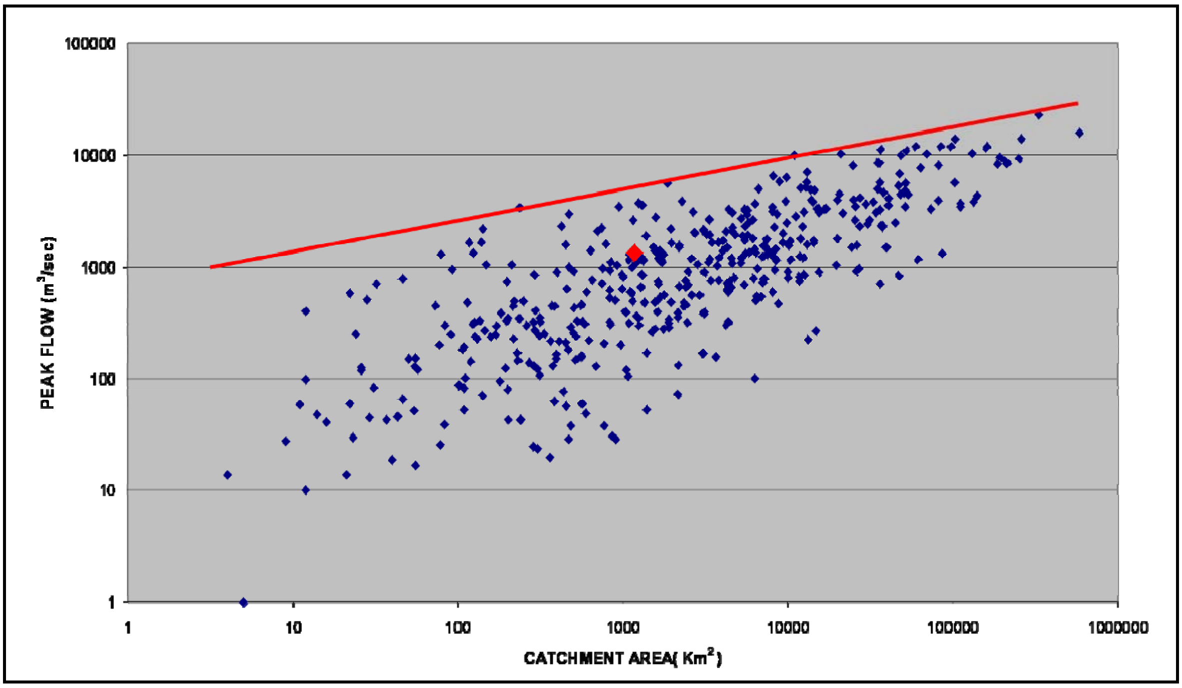


Fig. 6-4 - Extreme floods in Europe. ◆ Acheloos river.

A recent review of the floods in Sykia dam, carried out in 2007 by PPC for the study of the leakages through the foundation of the main upstream cofferdam in the Acheloos river, estimated the flood values for different return periods (low and medium return periods). Those are shown in Table 6-3 and Fig. 6-5.

Table 6-3 - Floods (m^3/sec), for low and medium return periods.

Flow	Return Period. T.
190	1,00
290	1,00
418	1,04
483	1,09
567	1,22
632	1,38
783	2,04
912	3,08
995	4,12
1048	5,00

1093	5,92
1174	8,06
1245	10,75
1372	17,86
1487	28,57
1590	43,48
1640	52,63

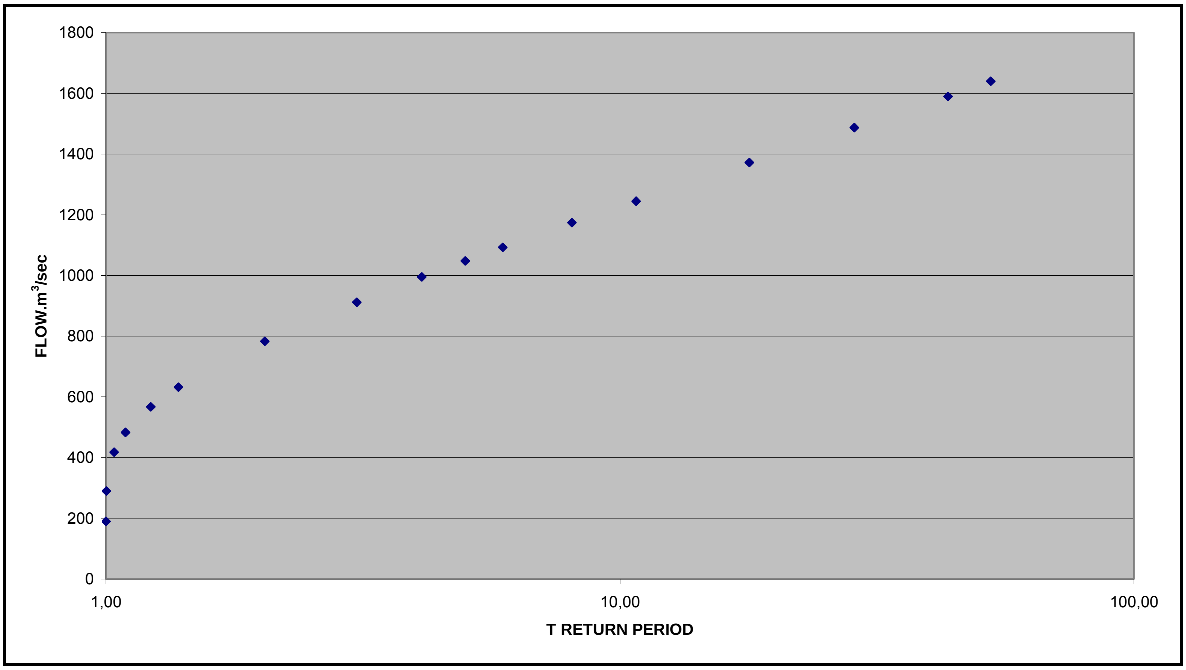


Fig. 6-5 – Flow vs return period for Achellos river at Sykia Dam site

Based on the graph of Fig. 6-5, for return periods between 3 and 50 years, the relationship between flood flow and the period of return can be analyzed with the equation (see Fig. 6-6):

$$Q = 253 \ln T + 638$$

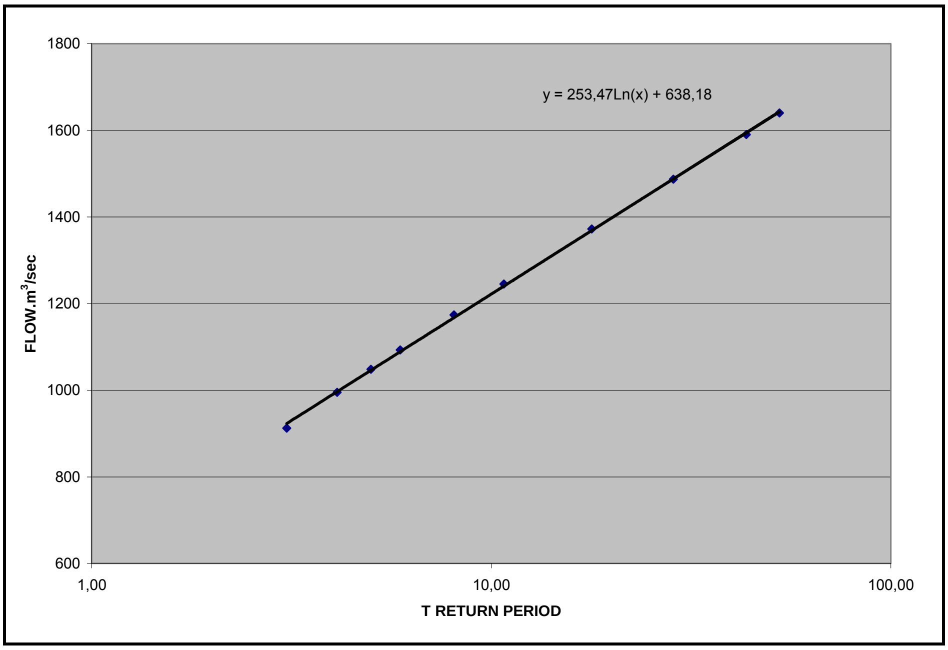


Fig. 6-6 - Relationship between flood flows and return period

Low and medium return periods

6.4.2 Overtopping

The main upstream cofferdam on the Acheloos River and the diversion tunnel (Tunnel No. 1), have been designed for the flood flow corresponding to a return period of 50 years: 1.672 m^3/sec. For this flood the water level at the intake of the tunnel rises to a maximum elevation +449.68. The crest of the cofferdam is at El. +458.50, thus providing a freeboard of 8.82 m.

The diversion tunnel began to operate in 1997. After 13 years in operation, it can be assessed that the probability of this 50 years design flood will be surpassed by means of:

$$\text{Prob } (Q \geq Q_{50\ years}) \text{ in N years} = 1 - (1 - 1/T)^N$$

Equally, it can be assessed the increase of this probability in the following years, in this case for +5 years and +10 years. Table 6-4 show these probabilities currently and for the scenarios +5 and + 10 years.

Table 6-4 - Probability of surpass the 50 years return period flood (%).

Return Period Years	N=1 Year	N=13 (currently) Years	N=13+5 Years	N=13+10 Years
50	2	23	30	37

However, it is necessary to take into account that the water level at the intake of the diversion tunnel, for the 50 year flood, is at el. +449.68 and that the crest of the cofferdam is at el. +458.50. The flow of the 50 years flood would take place with a freeboard of 8.82 m. On the other hand, in principle, the overtopping, without any freeboard would take place with a water level of +458.50, which corresponds to a flow of 1,830 m^3/sec, as it can be deduced from the rating curve of the Acheloos Diversion Tunnel shown in Fig. 4-2.

Consequently, if all the cofferdam freeboard is exploited, the maximum flow through the diversion tunnel, under limit conditions before cofferdam overtopping, it would be 1,830 m^3/sec, which corresponds to a flood return period of ~100 years.

In and of itself the potential hazard for the limit event of overtopping, it would be smaller than the probability of surpass the flood of 50 years return period. For the considered scenarios, the potential hazards for overtopping of the Acheloos main upstream cofferdam are shown in Table 6-5.

Table 6-5 - Acheloos River Cofferdam. Potential Hazard (%) Overtopping

Return Period Years	N=1 Year	N=13 (currently) Years	N=13+5 Years	N=13+10 Years
100	1	12	17	21

The catchment area of Koubourjiannitikos River is 228,84 km^2. A hydrological study by DAYE / PPC in 1986, has indicated floods and the corresponding return periods as shown in Table 6-6.

Table 6-6 - Koubourjiannitikos River Floods

Return period - years	Flood – m^3/sec
100	549
1,000	1020
PMF	1283

The Koubourjiannitikos MUC has a crest at elevation +462.00 i.e. 3.5 m higher than the Acheloos MUC. This difference in height in collaboration with the large diameter DT-2 (~10 m I.D.), facilitates the discharge of the 1:50 design flood while there is a flood in the Acheloos basin.

Concluding, the PoE is of the opinion that overtopping constitutes a hazard of medium importance to the project.

6.4.3 Internal erosion

Construction of the diaphragm wall on the D/S slope of the dam shell (see section 4.5) rendered watertight the section of the shell below elevation +425. Nevertheless, during flood events, when the water level in the flooded area U/S of the MUC was raised above the crest of the diaphragm wall (elevation +425), significant flows were observed immerging on the D/S face of the dam shell, at elevations slightly higher than the top of the diaphragm wall, as can be seen on Photo. 6-4 and Photo. 6-5.

Photo. 6-4 - Piping and water flow above the crest of the diaphragm wall

Photo. 6-5 - Piping and water flow above the crest of the diaphragm wall

The water flow overtopped the diaphragm wall and flowed on the downstream slope of the dam shell, as can be seen in Photo. 6-6.

Photo. 6-6 - 2009 Flood - Water flow over the crest of the diaphragm wall.

It is believed that water seeping through the alluvia deposits under the MUC, is forced by the watertight diaphragm wall to emerge over its crest, as shown in Fig. 6-7. It is obvious that the higher the water level in the lake upstream of the cofferdam, the higher the water flow emerging over the diaphragm wall.

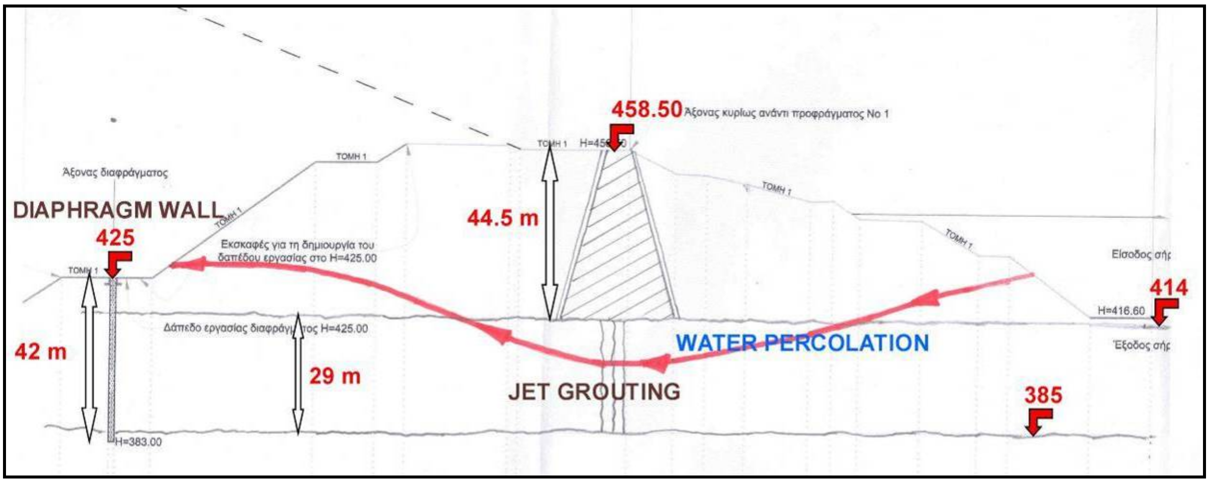


Fig. 6-7 - Cross section of the achellos coferdam and upstream dam shell.

During a flood in 2009, (after construction of the diaphragm wall), the water level upstream of the cofferdam reached elevation +434.00 m. High water flow was observed at the downstream face of the cofferdam, above the diaphragm wall. The flow was assessed (not recorded) at ~2 m^3/sec.

Water filtration through the dam shell resulted on formation of open gravels, following the washing out of the fines from the dam shell materials, (see Photo. 6-7).

Photo. 6-7 – Formation of open gravels after washing of fines from dam shell

These facts indicate that internal erosion of the alluvial of the foundation and piping through the cofferdam occurs when the water level rises above elevation 425.00. Internal erosion and piping may result in dam failure due.

The hydrological analyses show that the probability of reaching the bench mark +425.00 is very high, being evaluated currently at 91,6% while in the +5 and +10 years scenarios it is practically 100%, as shown in Table 6-7 and Fig. 6-8.

Table 6-7 - Probability of reaching the bench mark 425.00 for several scenarios

Water level	Flow m^3/sec	1 year %	+ 3 years %	+ 5 years %	+10 years %
420	190	100.00	100.00	100.00	100.00
422	290	99,8	100,00	100,00	100,00
424	418	96,4	100,00	100,00	100,00
425	**483**	**91,6**	**99,94**	**100,00**	**100,00**
426	567	81,8	99,40	99,98	100,00
427	632	72,3	97,87	99,84	100,00
428	783	49,1	86,81	96,58	99,88
429	912	32,5	69,25	85,99	98,04
430	995	24,3	56,62	75,14	93,82
431	1048	20	48,80	67,23	89,26
432	1093	16,9	42,61	60,37	84,30
434	1174	12,4	32,78	48,42	73,39

436	1245	9,3	25,39	38,62	62,32
440	1372	5,6	15,88	25,03	43,80
444	1487	3,5	10,14	16,32	29,97
448	1590	2,3	6,74	10,98	20,76
450	1640	1,9	5,59	9,15	17,46

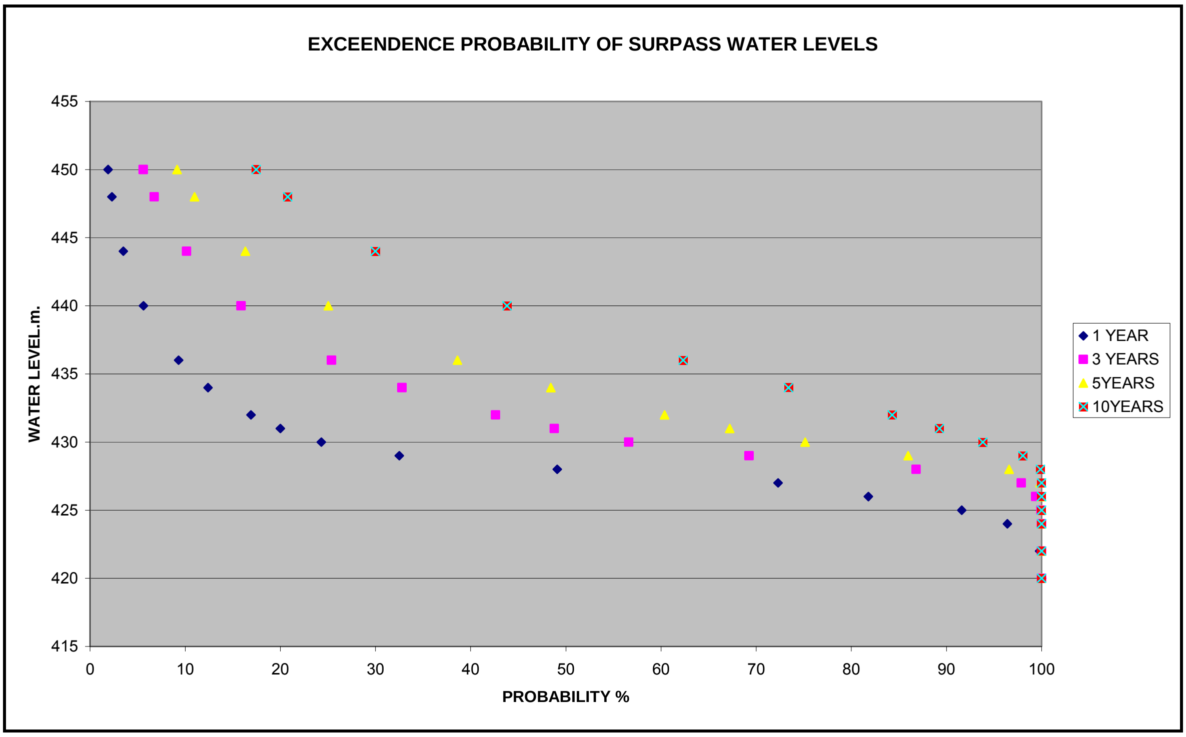


Fig. 6-8 - Probability of Reaching the Bench Mark 425.00 for Several Scenarios

Concluding, the PoE is of the opinion that internal erosion constitutes a very high hazard of serious importance to the project.

6.5 Diversion Tunnels Discharge Conditions

Reference is made exclusively to the two diversion tunnels (DT-1 and DT-2) that are in operation during the last 13 years and not to the Additional Diversion Tunnel that has not been used yet.

The inside of the both diversion tunnels is not easily accessible, since there is a constant flow of water through them. Hence the PoE **did not inspect** the inside of the tunnels during their visits to the site and consequently an accurate picture of the tunnel lining conditions cannot be formed.

Based solely on the observation that the water emerging from the tunnels, during the site visits, was completely clean, the PoE is of the opinion that there is no erosion of the rockmass surrounding the tunnel and presumably the tunnel lining still functions as a protective layer.

Nevertheless, the PoE is of the opinion that both tunnels may be inspected during low (summer) flows, aided by mechanical equipment and that such detailed inspection of the state of lining is absolutely necessary, primarily for DT-1.

Experience from similar projects in Greece, that indicates that diversion tunnels that have been in operation for less than 10 years, exhibited erosion of the concrete lining at the invert of the tunnel and exposure of the steel reinforcement. Considering that the diversion tunnels at Sykia have been in operation for 13 years and they could be in operation for the next 5 to 10 years and in addition that the tunnels have to carry the sediment loads of the Acheloos River, the need for inspection and maintenance becomes imperative.

The geomorphologic characteristics of the Koubourjiannitikos catchment area, makes the transport of mean size silts to be of importance (see Photo. 4-3). Also, in the situation of concomitant floods in the Acheloos River, the discharge capacity of DT-2 is influenced by the water level at its exit and that could produce a reduction in flow and a deposit of silts.

Over the years of operation of DT-2, the accumulated silts have reduced the useful cross section of the tunnel, in some sections to about 50%, according to the available data. It is henceforth recommended to carry out a detailed inspection of the state of the sedimentation inside the tunnel and of the reduction of the useful section, checking the state of lining for possible deteriorations.

Concluding, the PoE is of the opinion that the state of tunnel lining constitutes a hazard of medium importance to the project.

6.6 Conclusions

The hydrological potential hazards that may cause damage to the accomplished works relate to Sykia dam and the appurtenant works are: a) overtopping, b) internal erosion and piping and c) adequate operation of the diversion tunnels.

The probability to surpass the design diversion tunnel floods is currently at 23%. In the scenarios +5 and +10 years it increases respectively to 30% and 37%.

The potential hazard for the overtopping limit conditions of the cofferdam on the Acheloos River is currently 12%. In the scenarios +5 and +10 years it increases respectively to 17% and 21%.

The potential hazard for internal erosion and piping is very high, currently at 91,6% and in the +5 and +10 years scenarios it is practically 100%.

The potential hazards are summarized in Table 6-8.

Table 6-8 - Hydrological potential hazards

	Currently %	Scenario +5 years %	Scenario +10 years %
Probability of surpass the design cofferdam flood of 50 years return period	23	30	37
Potential hazard: limit overtopping	12	17	21
Potential hazard: internal erosion and piping	91,6	100	100

Furthermore, adequate operation of diversion tunnels is essential for the floods discharge of Acheloos River and Koubourjianitikos River and for hydrological safety of the cofferdams. It is recommended to carry out a detailed inspection of the state of lining, the state of silting, and of the possible deteriorations of the diversion tunnels.

7. POTENTIAL COFFERDAM FAILURE

7.1 Internal erosion

In Chapter 4 of the report the present situation of the completed site constructions and of the deviations and modifications to the initial basic design have been presented.

It is of major importance to repeat that the MUCs are far from granting the safety which was expected by the designers of the project.

The basic design of the project foresees a diversion system (diversion tunnels and cofferdams) with the capacity of dealing with the floods of 50 years return period or less. The assumption of the return period takes into account a) the duration of dam construction and appurtenant structures hence duration of diversion system operation, b) the type of dam, c) the consequences in case of system failure and d) the magnitude of design flood.

In the present case, the diversion system is used during a period far longer than anticipated.

Two points have to be raised:

1) Due to the long period of operation of the diversion system, the return period of the actual protection is less than the proposed value of the basic design (1/50 years).
2) The upstream cofferdam suffered various defaults which reduce considerably the assumed protection.

The first point is addressed in detail in Chapter 6.

The second point is treated in this Chapter.

The upstream cofferdams are embankment dams with a central clay cores. The shells are constituted of excavation materials from the diversion tunnels and of alluvium. These embankment dams are founded directly on the alluvium of the Acheloos and the Koubourjiannitikos rivers.

These alluviums are highly permeable. To make the foundation watertight, the design proposed the jet grouting method. The jet grouting method is efficient to improve the deformation modulus of the alluvium, but is frequently disappointing to create watertight barriers. The reason is simple: the presence of a big boulder or of a layer of consolidated clay creates a “shadow” where the grout cannot penetrate. These zones remaining untreated are permeable.

The potential hazards depend on the type of “shadows” and of the typology of the untreated zones. If the untreated zones are small and well distributed, the resulting leakages may be high but without risk of internal erosion, because the jet grouting diaphragm continues to play a role of filter towards the alluviums.

If the untreated zones are large, the fine and medium part of the alluviums may pass through. If the hydraulic gradient is high across the diaphragm, the internal erosion may be intense, leading to lose of material from the foundation of the cofferdam. Progressively, the sand and gravel eroded by water seepage, reduces the bearing capacity of the foundation and the cofferdam fails.

Clearly, the designers assumed that the second hypothesis is valid and considered necessary the construction of an additional watertight diaphragm.

It is difficult to know what have been the various technologies used and what was the difficulties encountered during the construction of the new diaphragm wall. The PoE has been informed that a tentative made upstream of the cofferdam axis was unsuccessful. The inability to construct the wall in that location is attributed coarse structure and highly permeability of the materials.

The “paroi moulée” is excavated by vertical panels or “barrettes”. During excavation, the panel is maintained open by bentonite slurry, which is continuously poured into the opening and creating what is called a “cake” on the vertical walls of the excavation. The slurry supports the vertical walls and maintains them till the excavation is filled from bottom to top with plastic concrete.

If the alluviums are too coarse and too permeable, the “cake” is not formed and the slurry leaks into the medium (in this case alluviums and rockfill coming from the excavation of the diversion tunnels).

It is pointed out that during construction of the Sykia diaphragm wall, the water table would have been in the alluvia, i.e. very low compare to diaphragm construction level (except during the floods), which increases considerably the slurry leakages into the surrounding materials. Usually the upper level of the diaphragm is not far from the water table. So the gradient of the slurry towards the medium (alluviums or rock fill) is low and the leaks are accordingly low.

An alternative solution with the diaphragm wall axis more or less on the axis of the cofferdam and of the existing jet grouting diaphragm was discarded, presumably due to the depth of wall construction (in excess of 60 m) and the concern of the contractor and its subcontractor

Bauer not being able to cross the zone treated with jet grouting, because it could be too hard even for an hydro fraise.

The PoE has been informed that the budget for the new diaphragm was fixed, wherever be the location and whatever the technology be. It was in the interest of the contractor or of the subcontractor or both, to reduce the area of the new cofferdam to a minimum.

The diaphragm finally built was on the downstream slope of the dam shell, from a berm at the elevation +425, by means of an hydro fraise. The diaphragm penetrates 2 m into the underlying rock.

At this location, the diaphragm is continuous from the right bank of the Acheloos River to the left bank of the Koubourjiannitikos River. The lowest point of the underlying bedrock on the Acheloos River is at elevation +380, while the deepest point on the Koubourjiannitikos River is at elevation +380.

The invert of the DT-1 inlet structure is at elevation +414. During the dry season, the water level is almost a few decimetres above this level. The diaphragm culminating at elevation +425 play its role preventing leakages through the foundation of the MUC.

During floods, the water elevation U/D of the cofferdam rises. For instance in 2005, before the construction of the diaphragm wall, two floods occurred. During one of the floods, the upstream level reached elevation +434.00, with a calculated discharge of 1175 m^3/s. In 2009, after construction of the diaphragm wall, the level of the water reached elevation +432.00.

In the first case (2005 flood), without the diaphragm wall, leakages occurred at the toe of the alluviums, at the bottom of the core excavation of the main dam.

In the second case (2009 flood), with the diaphragm wall, leakages occurred through the dam shell, above elevation 425. In this case the crest of the diaphragm wall is lower than the water level during the flood. However, it improves the situation by reducing the water gradient through the first diaphragm.

With the diaphragm wall constructed, the water head is: 432-425 ~7 m. Without the diaphragm wall, the water head would have been: 432-394 ~43 m, 394 being more or less the elevation of the bed rock at the bottom of the excavation of the main dam. Thus the hydraulic gradient is reduced by a factor of 4.

The maximum water level in the case of the maximum diverted flood is almost 457 which is almost the top of the clay core of the MUC. In this extreme case, the water head is 457–425 ~32 m, with the diaphragm wall, compared to 457-394~ 63 m without the diaphragm wall.

Even with the diaphragm wall, the water head for the maximum diverted flood is close to the experienced water head during the flood of 2005. It has been judged unacceptable in 2005. It should be kept in mind that bigger floods are also longer in duration. Internal erosion not only depends on the hydraulic gradient but also on the time that the gradient is applicable.

Internal erosion is a potential hazard that may result in failure of the cofferdam and of its foundation.

7.2 Sequence of Events

The volume of materials placed in the two MUCs, are widely increased by the temporary deposit of material for the construction of the main dam, in particular on the side of the Koubourjiannitikos river, where they have been stockpiled up to elevation +485.40. In case of cofferdam failure those materials will be flushed into the excavation of the main dam.

Reference is made to Fig. 7-1.

When the MUC and the U/S dam shell are overtopped, part of the flood will be directed over the dam shell and erosion of materials will commence. Following that the head of water at the entrance of the DT-1 will drop at a speed that will depend on the flood discharge and the slope wet cross section of the eroded U/S shell. This will result in drop in water level and drop in diversion tunnel discharge, thus increasing the flow over the dam shell and the erosion. The process is then continuous, unless there is a sudden drop on the incoming flood. Consequently materials from the MUC and the U/S dam shell will be transported and deposited into the space between the U/D and D/S dam shells. This constitutes Phase A of the erosion process.

If the flood continuous for sufficiently long time, the space in-between the dam shells shall eventually be filled with water and eroded materials (total capacity ~2 hm^3) and the D/S dam shell and shall then be overtopped, since its crest is at elevation +435, i.e. 33.5 m lower than the crest of the MUC. If the space between the dam shells is sufficiently large to accommodate the volume of materials eroded from the U/S dam shell, then mainly water will overtop the D/S dam shell, otherwise both water and debris will overtop it. Whatever the case may be, materials from the D/S dam shell will be eroded, transported and deposited in the space between the D/S dam shell and the main downstream cofferdam. This constitutes Phase B of the erosion process.

If the flood continuous for sufficiently long time, the space in-between the D/S dam shell and the D/S cofferdam will be filled with water and (possibly) eroded materials and the D/S cofferdam and shall then be overtopped, since its crest is at elevation +423.60, i.e. 8.4 m lower than the crest of the D/S dam shell.

The material of the D/S face of the D/S cofferdam will be eroded and the river will flow more or less on the present river bed.

Part of the eroded materials will be trapped in the space between the U/S and D/S cofferdams. However, the flow of the river will transport sand and gravel, depending on the velocity of the flow. These materials will eventually be deposited in the wider river valley of Acheloos, downstream of the Korakou Bridge (see Photo. 7-1).

Photo. 7-1 – Acheloos river downstream of the Korakou Bridge

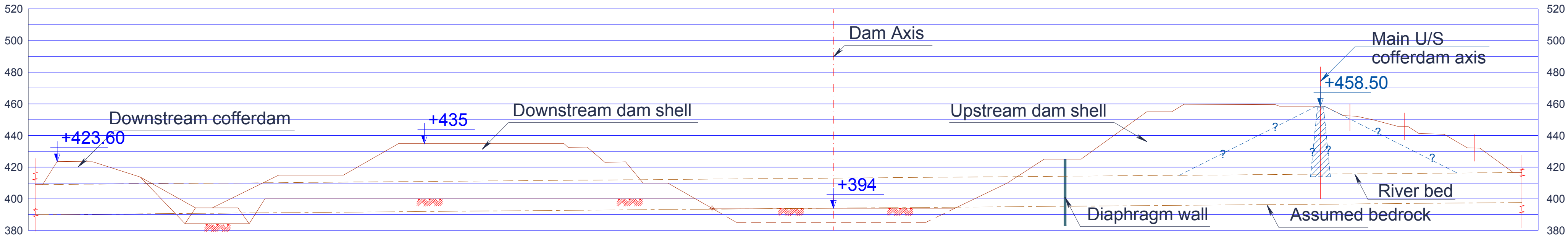


Fig. 7-1- Typical section along the Acheloos river course

(Redrawn from Contractor's Drawing TOM_1)

8. POTENTIAL HAZARDS TO THE DOWNSTREAM AREAS

8.1 Introduction

In case of cofferdam failure, a volume of water will be released and will move along the river course. In the worst scenario the total volume stored behind the cofferdam will be released, although this is rarely the case. With the water level at the crest of the cofferdam (+458.50), the stored volume is ~25 hm^3. So it can simply be deducted that in case of a cofferdam failure, a volume of water between 10 and 20 hm^3 may be released along the river, in the form of a flood wave.

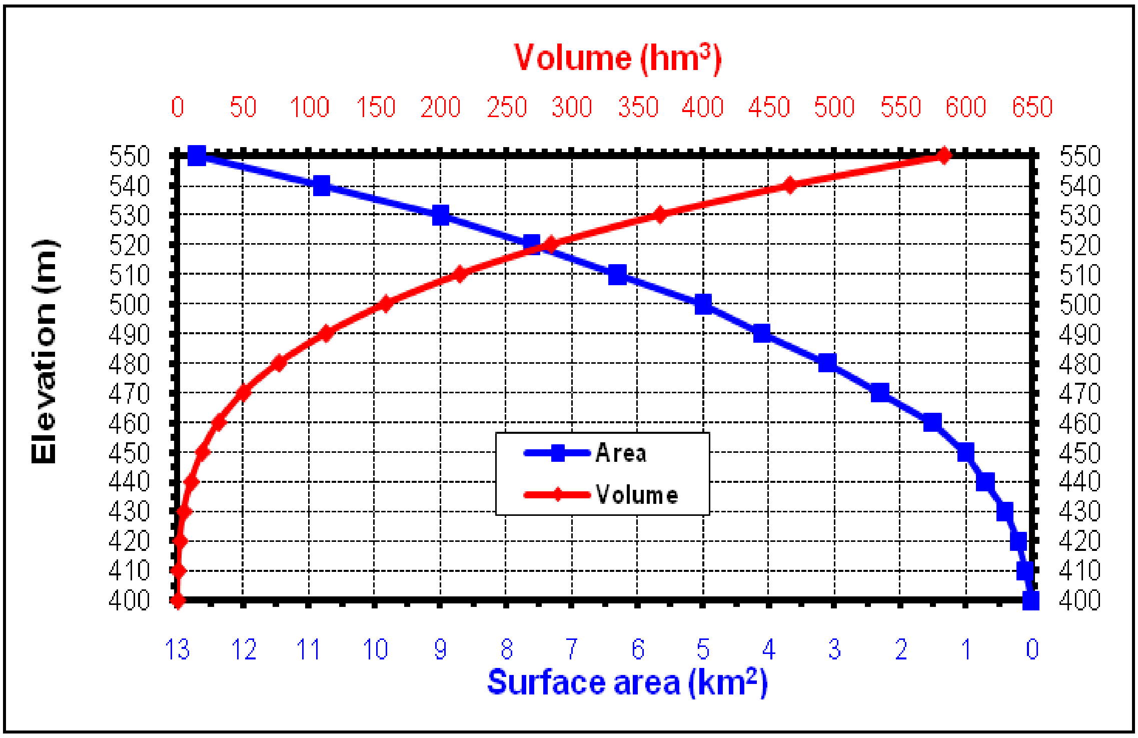


Fig. 8-1 – Reservoir curves of Sykia dam

The depth of water wave and its velocity will vary along the river, depending on various factors, e.g. the shape of valley. Detailed dam break analysis is required to investigate the advance of the flood wave along the river valley and special reference is made on that in the following paragraph.

8.2 Dam Break Mechanism

The dam break mechanism depends mainly on the depth of the reservoir at the dam site. If a section of a dam is assumed to 'disappear' instantly, a wave is created. Theoretically, the flow follows the equation of Barré de Saint-Venant. The wave, as a roll on a beach, progresses constantly. Its velocity is controlled by the energy dissipated in the water roll.

To make things simple, at the beginning, the height of the wave is almost half the height of the water at rest before the collapse of the dam. The height of the wave diminishes mainly when the valley enlarges. As the velocity diminishes also, the transported sands and gravels are deposited.

A good forecast may be made with the help of software which resolves the equation of Barré de Saint-Venant. The commercially available software *DAMBREAK*, for instance, is well known and gives a reasonable approximation, assuming a sudden failure of a dam. This assumed hypothesis is the worst case in terms of hydraulic behavior. It gives the upper limit of the hydraulic impact of a dam collapse.

In the present case, the initial conditions at the time of sudden failure may vary considerably, depending on the reason of failure:

- If the failure is due to internal erosion and piping, the failure could occur for a water depth equal to 11 m (or more) and a reservoir volume of ~12 hm^3 (or more) (HV=55).
- If the failure is due to overtopping, the failure could occur for a water depth equal to 45 m and a reservoir volume of 25 hm^3 (HV=1125).

One of the most important factors to be considered is the shape of the valley downstream of the cofferdam. In the case of Sykia project, the valley has a relatively narrow V shape for a distance of ~2.5 km from the dam site, till the Korakou Bridge, then the valley widens to almost 500 m for a distance of 10 km and then narrows again for another 10 km before it enters the Kremasta reservoir (see Photo. 8-1, a satellite photograph obtained from Google Earth and dated Nov. 2003).

Assuming an instantaneous failure of the dam, the software gives, at various locations along the river, the maximum level reached by the water and the time after failure that this maximum level is reached.

The time interval that the maximum water level is reached at various points along the river is of crucial importance, because it determines the characteristics of the warning system to be developed, in order to prevent or at least minimize the dramatic consequences of a dam failure.

It is also of interest to know, from the relevant calculations, the velocity reached by the water at various points along the river, to anticipate the solid deposit.

From the aerial photograph by Google Earth shown in Photo. 8-1, it is recorded that along the Acheloos River downstream from the Sykia dam, there are three bridges: Korakou Bridge (see Photo. 8-2), Avlaki Bridge (see Photo. 8-3) and Templa Bridge (see Photo. 8-4). It is pointed out that the last two are of historical value.

Photo. 8-1 – Aerial photo of Acheloos valley downstream of dam site

A short distance downstream of Templa Bridge, Acheloos River enters the large Kremasta reservoir, of total capacity ~4,500 hm^3. The reservoir supplies water to the Kremasta hydroelectric station, managed by PPC.

Photo. 8-2 – Korakou Bridge

Photo. 8-3 - Avlaki Bridge

Photo. 8-4 - Templa Bridge

In between the Avlaki and the Templa bridges, a private company has recently constructed the small hydro of Dafnozonara (see Photo. 8-5), of ~8.5 MW and a storage volume of ~1 hm^3.

Photo. 8-5 - Dafnozonara Small Hydro on the Acheloos River, D/S from Sykia dam site

In addition to the bridges and the small hydro, downstream of the dam site and at a distance of ~500 m from it, is the old customs building on a protruding rock on the left bank of the river (see Photo. 8-6). This building is of historical value.

Photo. 8-6 - Old customs building on the left bank of Acheloos River

The PoE strongly recommends that all the areas close to the river be searched for human activities and installations. Those should be mapped, recorded and their value estimated. During the visit of the PoE to the site, it was observed that close to Korakou Bridge on the left bank and practically at the same elevation as the bridge deck, a restaurant is in operation, with the proprietor living on the upper floor of the building.

Similar housing installations may exist at other locations along the river banks and those should be carefully investigated and accurately recorded.

8.3 Potential Effects

The effects of the collapse of the three cofferdams are:

1) The main dam excavation will be filled by material of the MUCs and of the temporary stockpiled materials deposited on the MUCs.
2) Potentially, a wave could be created by the collapse of the downstream cofferdam. The height of this wave could reach 10 to 15 m. The height will remain more or less constant till the Korakou Bridge, but then the wave will decrease in the plain downstream of the bridge. The wave may travel far beyond the flat plain, but this will be determined by the appropriate dam break analysis.
3) On the basis of historical data, Costa (USGS) proposed that the peak flow in case of dam failure is approximately related to $Q_P = 325\ (HV)^{0.42}$, which gives 1,749 m^3/s for 11 m reservoir depth and 6,213 m^3/s for 45 m reservoir depth.
4) Debris that will be transported along the river will eventually block the opening under the Korakou Bridge and the bridge will unavoidably be overtopped. The debris carried by the water flow will very likely be deposited in the plain downstream of the Korakou Bridge, where the valley increases considerably in width and flow velocities will drop.
5) When the bridge will be overtopped the nearby restaurant is in a very risky situation. The restaurant and the associated houses should be evacuated in case of floods exceeding the flood controlled by the diversion system. In the present state, the return period of such a flood is very short.
6) The flood wave will most certainly destroy the old custom building on the left bank of the river, being so close to the river level.
7) The flood wave may cause severe damages to the two historical stone bridges of Avlaki and Templa, but this will be investigated by the dam break analysis.

8) Potential damages to the Dafnozonara small hydro cannot be assessed easily. Presumably damages to the concrete structures may be detrimental, but damages to the mechanical parts (e.g. radial and slide gates, bridges and over-structures) should be expected. Deposition of river sediments into the power intakes is a potential hazard. Again the dam break analysis should offer some insight into those hazards. Unavoidably interruption of energy production is to be expected and compensations should be considered.

9) The surge or inflow of the flood wave into the Kremasta reservoir is anticipated not to be a problem, mainly due to the vast storage capacity of the Kremasta reservoir in relation to the expected inflow volume.

10) The flood wave and sediment transport should not have a strong impact on the landslide itself, but could cause important environmental impacts.

11) The Additional Diversion Tunnel has a good chance to be filled with sand and gravels.

9. MEASURES TO REDUCE THE POTENTIAL HAZARDS

9.1 Mitigation Measures

In a risk analysis, it is usual to define the potential hazard event, then to associate to this event a probability of occurrence and to finally to project it in the economical field to obtain the value of the risk.

In the case of Sykia dam, it could be useful to calculate the economical value of the potential effects of the most critical event. This value should be compared with the cost of the various mitigation measures.

However, the project administration is fully concerned because the present protection against floods is **by far below** the protection against floods foreseen by the designer of the project, accepted and approved by the administration, subjected to bidding and actually built. Whatever may be the economical cost of the occurrence of the cofferdams destruction, it shall be considered as non acceptable not to have the protection foreseen, i.e the flood of frequency of 1/50 for the whole duration of the construction.

The PoE deems absolutely necessary that the works left in a stand-by state and for a period of uncertain duration, should be passively protected against a flood which has a frequency of occurrence 1/50. The flood protection system must include both the Acheloos and Koubourjiannitikos floods.

Following this assumption, the crest elevation of each cofferdam should be determined on the basis of the hydrogram of each flood and of the rating curve of each tunnel.

9.1.1 Structural mitigation measures

Both diversion tunnels have been partially filled in the past with alluvium transported during flood events and following the landslide that occurred downstream of the dam site, that blocked the river.

The site visit to Koubourjiannitikos intake revealed that it is partly filled with sediments, but it appears that they are not very consolidated. It is the opinion of the PoE that those sediments will be flushed away easily by future incoming floods. Consequently no remedial measures seem necessary.

As stated previously, the PoE did not have the opportunity to visit and inspect the DT-1, so the state of the concrete lining is not presently known. Since its operation, the tunnel has not been inspected. The PoE strongly recommends that such an inspection should be carried out at low river flows and remedial measures should be taken if necessary.

The next step is to determine the elevation of the main upstream cofferdam crest, so as to offer the same level of flood retention as anticipated in the design of the project. To determine the level of the crest of the new cofferdam, it is necessary to analyze the level of protection offered of today as compared to the level of protection foreseen in the project design.

In 1997, when the diversion tunnels were first placed in operation, the probability to exceed the 1:50 year flood (1670 m^3/sec) was 2% for the first year, increasing to 10% in the following 5 years. It is deducted that if the project was to take ~5 years to completion, then an acceptable level of protection would be 10%. The 50 year flood can be accommodated through the DT-1 with water level at elevation +449.68.

Taking into consideration the freeboard provided by the crest at elevation 458.50, the flood that can pass through the DT-1 is ~1830 m^3/sec, which corresponds to a return period of 100 years. The corresponding probability to exceed that flood would vary between 1% and 10% for 1 to 10 years construction time.

Appling those conditions to the present year, the probability to exceed the 1:100 year flood (1830 m^3/sec) in 2010 is increased to 12%, while in the nest 5 to 10 years the value increase further to 17 and 21 % respectively.

In an attempt to re-establish the originally acceptable conditions, the probability to exceed should be reduced to at least 10% and this is achieved if the design flood corresponds to a return period of 150 (for 5 years waiting time) or for a return period of 200 years (for 10 years waiting time).

Above results are summarized in Table 9-1.

Table 9-1 – Probability to exceed floods

	T (years)	N=1	+5 years	+10years
In 1997	100	1%	5%	10%
In 2010	100	12%	17%	21%
In 2010	150	8,3%	11%	14%
In 2010	200	6,3%	8,6%	11%

The floods corresponding to return periods of 150 and 200 years are obtained by the equation Q = 253 lnT+638, derived in paragraph 6.4.1.

Return period - years	Flow – m^3/sec	Water level	Height above diaphragm - m
150	1905	+462	37
200	1978	+468	43

The PoE considers as more appropriate the selection of the 150 year return period as the new design return period for the diversion system, which corresponds to a diversion flood of 1905 m^3/sec. This design flood can be passed through the DT-1 with a cofferdam having a 'new' crest at elevation +462.00.

9.1.2 New diversion system

Various solutions have been considered to:

a) increase the crest of the cofferdam to elevation +462 and

b) make the 'new' cofferdam watertight over its whole height.

c) Increase the discharge capacity of the diversion tunnel by excavating an additional tunnel,

Cofferdam with upstream facing associated with a toe diaphragm wall (see Fig. 9-1)

This solution requires forming the upstream face of the cofferdam into a uniform surface and placing a watertight element over that surface. The watertight element could be cement concrete, asphaltic concrete or geo membrane. This solution requires that the watertight element be placed over the whole upstream surface of both cofferdams, on the Acheloos and the Koubourjiannitikos rivers, which is a fairly large area.

The river alluvia should also be made watertight by construction of a vertical diaphragm constructed in panels by hydro fraise and plastic concrete, or piles or even grouting by tube a manchette method or otherwise. The technology for diaphragm construction, taking into account the characteristics of the alluvium, is well established today, so this method is easily applicable.

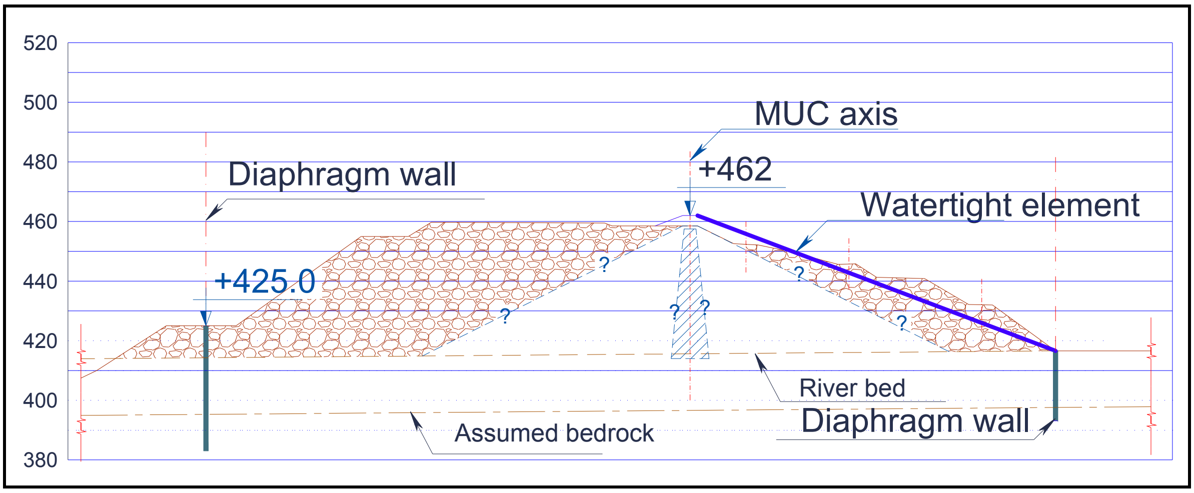


Fig. 9-1 – Solution with U/S facing associated with a toe diaphragm

<u>**Cofferdam over the diaphragm wall (see Fig. 9-2)**</u>

This solution exploits the existing diaphragm wall (which has proved to be a well functioning watertight element). by building an impervious core over it and extend the core up to the required elevation. In this case the height of the core would be 37 m above the diaphragm head.

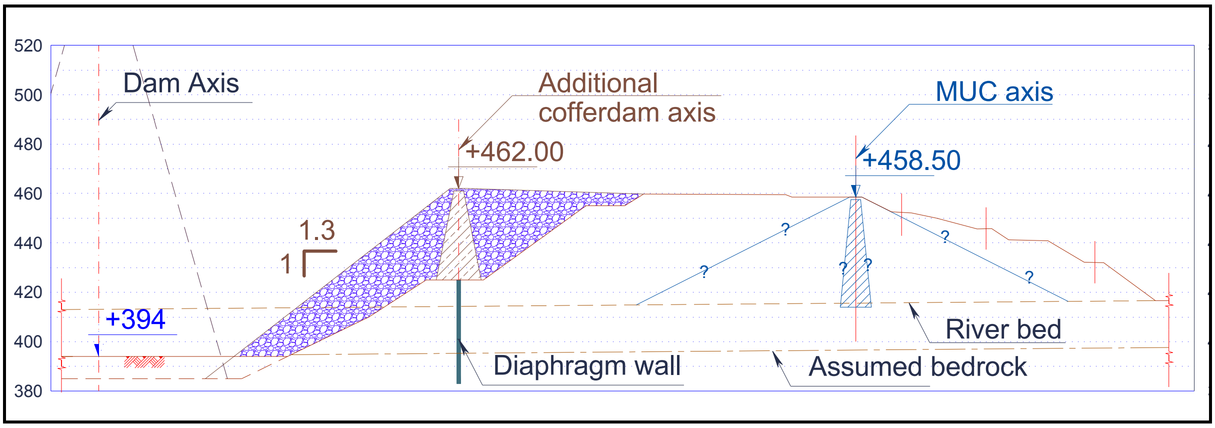


Fig. 9-2 – Cofferdam over existing diaphragm wall

The downstream shell should be made with the available stockpiled materials. It is estimated that the friction angle of the stockpiled materials could be as high as 37° to 39° with no cohesion. The actual values should be determined by relevant laboratory testing. A filter and drain system should be designed and incorporated.

The impervious core could be a clay core, asphaltic core or plastic concrete. The PoE is of the opinion that a clay core is more flexible and probably cheaper.

Additional diversion tunnel

To pass the new design flood of 1905 m^3/sec D/S of the dam site, one could retain the existing cofferdam crest, but increase the discharge capacity by building an additional diversion tunnel, parallel to DT-1. Construction of such an additional diversion tunnel would not be cost effective because the water head should be maintained below elevation +425 to prevent internal erosion and piping. Consequently, the diameter of the additional tunnel should have to be fairly large.

Having considered carefully the various alternatives, the PoE is of the opinion that the cheapest and safest solution would be the extension of the cofferdam over the existing diaphragm wall, with a 'new' crest at elevation +462.00.

9.1.3 Overtopping of 'new' cofferdam

Overtopping of the 'new' cofferdam is unavoidable In case of floods above the proposed limit. It seems too expensive to make the upstream cofferdam capable to withstand safely overtopping. If the cofferdam is overtopped, the event would be relatively similar to the event described in the Chapter 7.

The excavations of the main dam will be filled by eroded materials. The consequences of filling the present excavation of the main dam by the material resulting of the collapse of the upstream cofferdam are tolerable.

Nevertheless, It would be easier and most cost effective to make the downstream cofferdam able to withstand overtopping, with the aim of limiting the solid transport towards the downstream valley and to prevent the creation of a hydraulic wave. Various solutions can be used to make an embankment dam able to withstand overtopping. Tests and prototypes have been done.

The most common is to use large enough blocks of rock, that are able to withstand the water loads. In such cases the size of the blocks has to be large. For instance, a block with a diameter 0.10 m is stable if the velocity of the water is less than 2,7 m/s, and a block with a diameter 0,50 m is stable if the velocity is less than 4 m/s.

Availability of such blocks depends on the structure of the formations in the vicinity of the site. The limestone at the site is in general thinly bedded and it could tedious and fairly expensive to construct a rip-rap with blocks over 1 m in diameter.

The capacity of the large blocks to resist dam overtopping is never guaranteed and depends first on the good arrangement of the blocks (like cyclopean masonry) so there is effective interlocking, second on the contact between the rip-rap and the abutments, where the flow of water is concentrated and third on the duration of the flood.

Another solution tested and used in South-Africa consists in reinforcing the riprap with steel bars. A cyclopean mesh is done with re-bars (diameters 25 or 32 mm, space 70% of the size of the rip rap). This mesh is anchored in the rip-rap and in the underlying rockfill by re-bars of similar size.

There is one case that the reinforced riprap collapsed by failure at the contact with the abutment. A block was forced laterally out of the mesh and the other blocks followed. However this is a fairly cheap solution and could well fit to the case of Sykia.

9.1.4 Non structural mitigation measures

It is not clear whether or not the dam site will remain under human control with a permanent team, while waiting for the court's decision. Assuming that the answer is yes, it is highly recommended to establish a control and surveillance team that would inspect the project primarily during the flood season, from November to April.

A fence system should be installed to prevent people and animals domestic or wild entering the dam site. Sign posting should be installed in the accessible zone to explain the situation and inform of potential dangers.

A warning system should be installed to announce the floods, to warn the restaurant, the small hydro and other installations and finally close the bridges.

In case of strong floods, whenever the reservoir upstream the cofferdams reach a critical level, or, of course, if the cofferdam failing is imminent, a reliable warning system, should alert people, the bridges should be closed to traffic, the restaurant should be evacuated (It is compulsory to say where people should go, because the bridge will be unusable).

To give data to the warning system, special device should be installed to measure the level of the two upstream lakes and to detect a collapse of the cofferdam. This system should be regularly controlled.

The team in charge should alert their administration and be prepared to observe, to take photos and movies of the event. The procedures to be followed should be established.

Among the non structural measures, to minimize the potential hazards at Sykia Dam, prominent position has the **commissioning of the Messochora Dam**. Messochora's reservoir can effectively used to control the floods of the Acheloos River. As the flood of the Koubourianitikos River is by far smaller and shorter than the floods on the Acheelos River, the problem would be reduced.

9.2 Measures to Eliminate the Potential Hazards

9.2.1 Complete construction of Sykia Dam to a certain elevation

To eliminate the potential hazards, or to be more precise, to restrict the potential hazards to the values foreseen and accepted by the approved design of the project, would necessitate that all functions of the project are carried out exclusively by the permanent structures.

If the dam were to be raised to an elevation that an ungated spillway could accommodate safely the spillway design flood, and the diversion tunnels were to be permanently sealed, then the dam fulfils the basic safety requirements. In principle if the dam is raised above the spillway sill (+528.00) to accommodate the design flood of 5.250 m^3/sec **without gates**, then the potential hazards are theoretically eliminated.

It is emphasized that this approach takes advantage of the important investment of approx. 1.6 billion € in the works.

9.2.2 Dam Decommissioning

If the final decision is not to build the Sykia dam, it would be necessary to decommission the constructed works and to re-establish the river at its initial state. That would necessitate flushing away or excavation and reposition of ~2 hm^3.

9.3 Final Conclusion

This report illustrates the potential hazards to the works at Sykia dam site and the downstream areas by the abrupt interruption of the works. The report shows the potential hazards due to overtopping, internal erosion and piping, their relative importance and their consequences and comes to the conclusion that internal erosion constitutes a very high hazard of serious importance to the project. The potential failure of the cofferdams caused by internal erosion is very likely (91%) to happen and it is almost certain to happen within the next five to ten years.

Consequently the PoE is of the opinion that it is absolutely necessary to take significant measures in the immediate future, to mitigate or eliminate those potential hazards.

Having considered carefully the various alternatives, the PoE is of the opinion that the cheapest and the safest solution would be the extension of the cofferdam over the existing diaphragm wall, with a ‘new’ crest at elevation +462.00.

10. ATTACHMENTS

Attached to this report are the two drawings prepared by the Contractor and given to the PoE by the MITN site office:

1. ORIZ_1/22-07-2010: General plan of Sykia Dam
2. TOM_1/22-07-2010: Sections 1 and 2 of Sykia Dam