



**BRELA 1995
CROATIA**



INTERNATIONAL CONFERENCE PERUĆA DAM REMEDIATION



PROCEEDINGS

January 28, 1993



PUBLISHER

CROATIAN NATIONAL ELECTRICITY



PERUĆA DAM REMEDIATION

PROCEEDINGS

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CROATIAN NATIONAL ELECTRICITY



**CROATIAN COMMITTEE
ON LARGE DAMS**



CONTENTS

1. THE PERUĆA DAM ROLE WITHIN THE CETINA RIVER	1
POWER GENERATION SYSTEM, ITS OCCUPATION AND ATTEMPTED DESTRUCTION <i>Marin Vilošić, B.Sc.C.E.</i>	
2. THE PERUĆA DAM REMEDIATION SOLUTION	29
<i>Dr.Sc. Josip Rupčić</i>	
3. THE PERUĆA DAM REMEDIATION DESIGN	65
<i>Zvonimir Sever, B.Sc.C.E.</i>	
4. SITE ORGANIZATION AND REMEDIATION WORKS	113
<i>Mijo Zec, B.Sc.C.E.</i>	
5. SEISMIC AND SEISMOTECTONIC INVESTIGATIONS	177
AT THE PERUĆA DAM SITE <i>Vlado Kuk, M.Sc.</i> <i>Professor Dr.Sc. Eduard Prelogović</i>	
6. THE PERUĆA DAM FLOODS ANALYSIS	195
<i>Dr.Sc. Ranko Žugaj</i>	
7. GEOTECHNICAL INVESTIGATIONS AND DESIGN PARAMETERS	211
FOR STABILITY ANALYSES OF THE PERUĆA DAM <i>Tomislav Ivčić, M.Sc.</i> <i>Professor Dr.Sc. Anton Szavits-Nossan</i> <i>Meho Salo Kovačević, M.Sc.</i>	
8. SEISMIC STABILITY OF THE PERUĆA DAM	229
<i>Professor Dr.Sc. Anton Szavits-Nossan</i> <i>Tomislav Ivčić, M.Sc.</i> <i>Meho Salo Kovačević, M.Sc.</i>	

9. THE PERUĆA DAM STRESS-STRAIN ANALYSIS	249
<i>Professor Dr.Sc. Antun Savits-Nossan</i>	
<i>Mirko Saka Kovačević, M.Sc.</i>	
<i>Tomislav Ivšić, M.Sc.</i>	
10. INVESTIGATIONS AND CONTROL OF MATERIAL	261
PLACEMENT INTO THE DAM ROCKFILL	
<i>Krešimir Sever, Geotechnical Engineer</i>	
<i>Miroslav Ivaš, Geotechnical Engineer</i>	
<i>Ivo Barbačić, B.Sc.C.E.</i>	
11. DAM CORE GROUTING	295
<i>Dr.Sc. Josip Rupčić</i>	
12. PLASTIC CONCRETE CUT-OFF WALL CONSTRUCTION	313
<i>Dalibor Bojanić, B.Sc.C.E.</i>	
<i>Zvonimir Šepac, B.Sc.C.E.</i>	
13. LABORATORY TESTING OF THE CUT-OFF	345
WALL PLASTIC CONCRETE	
<i>Professor Dr.Sc. Velimir Ukrainczyk</i>	
<i>V. Čalopović, B.Sc., V. Žugelj, B.Sc., I. Banjad, B.Sc.</i>	
<i>Dr.Sc. Danja Mikulić</i>	
14. GROUTING GALLERY DAMAGE AND REMEDIATION	365
<i>Zvonimir Sever, B.Sc.C.E.</i>	
<i>Mijo Zec, B.Sc.C.E.</i>	
15. DAM REMEDIATION GROUTING	403
<i>Dr.Sc. Josip Rupčić</i>	
<i>Nenad Kenić, B.Sc.G.E.</i>	
16. SPILLWAY CONSTRUCTION AND HYDROMECHANICAL EQUIPMENT	423
<i>Dalibor Bojanić, B.Sc.C.E.</i>	
<i>Dr.Sc. Ante Krstulović</i>	
17. DAM MONITORING SYSTEM	437
<i>Zvonimir Sever, B.Sc.C.E.</i>	



THE PERUĆA DAM ROLE WITHIN THE CETINA RIVER POWER GENERATION SYSTEM, ITS OCCUPATION AND ATTEMPTED DESTRUCTION

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Project Manager

Croatian National Electricity Ltd.

SPLIT

ABSTRACT

At 10:48 am, on January 28, 1993 the Serbian aggressor destroyed a 32 year old structure of the Peruća HPP rockfill dam by the terrorist act of activating explosive planted in the inspection gallery. The dam withstood the attack and succeeded to retain the $470 \times 10^6 \text{ m}^3$ "water bomb" with unforeseeable destructive effect downstream the dam, all the way to the town of Omiš.

INTRODUCTION

The Cetina River, with its wider catchment area of $4,150 \text{ km}^2$ and mean discharge at the mouth into the Adriatic Sea of $102 \text{ m}^3/\text{s}$, presents an exceptional water potential.

Since the first plant, Kraljevac HPP, was built in 1912, the Cetina with its catchment become a valuable hydraulic resource for production of the electric power in five hydroelectric power plants with the total installed power of 877.6 MW.

The valuable power generation projects built on the Cetina and within its catchment started with construction of the Peruća Dam, reservoir and the hydroelectric power plant adjacent to the dam.

The Peruća Dam and reservoir, together with the power plant, were built on the upper stretch of the Cetina in 1960. It was intended to train the Cetina discharges up to the Peruća Dam and later, together with the Buško Blato reservoir, train the Cetina water in its entire catchment so that the Zakućac HPP reaches its maximum power output.

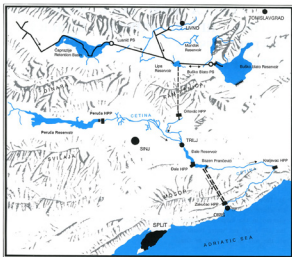


Fig. 1. The Cetina River hydropower system - map



Fig. 2. The Cetina River hydropower system - longitudinal section

ORLOVAC HPP	Qi = 70 m ³ /s,	Hk = 380 m,	Pi= 237 MW,	E = 440 Gwh
PERUĆA HPP	Qi = 120m ³ /s	Hk = 41 m,	Pi= 41,6MW,	E = 120 Gwh
DALE HPP	Qi = 220m ³ /s	Hk = 21 m,	Pi= 40 MW,	E = 157 Gwh
ZAKUČAC HPP	Qi = 220m ³ /s,	Hk = 250,4 m,	Pi=486 MW,	E =1770 Gwh
KRALJEVAC HPP	Qi = 80m ³ /s	Hk = 103 m,	Pi= 73 MW,	E = 40 Gwh

DESCRIPTION OF THE MAIN STRUCTURES

The Peruća Dam with the reservoir and the hydroelectric power plant adjacent to the dam was constructed on the Cetina River, about 14 km upstream the town of Sinj. The Peruća reservoir is one of the main storage basins within the hydropower system on the Cetina River. The head created by the dam is used for operation of the Peruća power plant adjacent to the dam.

The Peruća reservoir has total capacity of $541 \times 10^6 \text{ m}^3$ at the reservoir level at El. 360 m a.s.l. It is used for annual balance of the Cetina discharge and was created by the construction of the rockfill dam.

The spillway with gates for flood water discharge was built in the left dam end. Originally it was designed to discharge a 100 year flood at the reservoir level of 361.20 m a.s.l., namely a 1000-year flood together with the bottom outlet.

The turbine water is supplied through a tunnel running around the dam through the left bank. The tunnel opening span is 6.7 m, and it is 300 m long. Upstream the turbines, the tunnel turns into a steel distributing penstock which ends at the turbine butterfly valves. The penstock diameter reduces to 3.8 m upstream the butterfly valves. The steel part of the pipeline is laid in the concrete block.

The power house and the switchyard are located upstream the dam, on the downstream side.

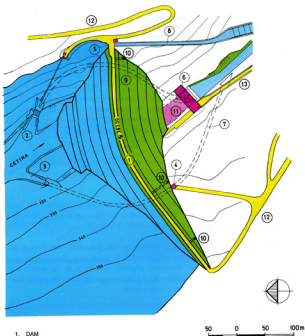
BASIC TECHNICAL AND POWER GENERATION DATA

RESERVOIR - before explosion

Reservoir levels:

maximum operating	360.00 m a.s.l.
maximum flood	361.20 m a.s.l.
minimum operating	325.00 m a.s.l.

Capacity at max. reservoir level	$541.249 \times 10^6 \text{ m}^3$
Area at max. reservoir level	20 km ²
Length at max. reservoir level	23 km



1. DAM
2. HEAD RACE TUNNEL INTAKE STRUCTURE
3. BOTTOM OUTLET INTAKE STRUCTURE
4. AUXILIARY GATE
5. SPILLWAY
6. PERUĆA HPP
7. BOTTOM OUTLET TUNNEL WITH CONTROL GATE
8. SPILLWAY CHUTE
9. HEADRACE TUNNEL
10. GROUTING GALLERY ACCESS TUNNELS
11. SWITCHYARD
12. DAM ACCESS ROADS
13. POWER HOUSE ACCESS ROAD

Fig. 3 Structures layout

DAM - before explosion

Excavation:		
	dam foundation	43,048 m ³
	gallery foundation	45,387 m ³
Dam volume:		874,097 m ³
Contains:	rockfill	709,559 m ³
	filter layers	46,976 m ³
	clay core	105,023 m ³
	concrete gallery	12,539 m ³
Dam height in the Cetina profile:		63 m
Dam width in the Cetina profile:		220 m
Dam length in the crest:		450 m
Reservoir capacity/dam volume		630/1
Discharge structures capacity		
at the reservoir level of 361.20 m a.s.l.		
	- spillway	340 m ³ /s
	- bottom outlet	220 m ³ /s

GROUT CURTAIN

Curtain length	1600 m
Max. grouting borehole depth	200 m
Boreholes length	164,000 m
Total length of grouting boreholes	140,000 m
Dry matter consumption:	51,394 t
Contains:	
clay	35,144 t
cement	14,492 t
bentonite	1,260 t
sodium	282 t
dolomite sands	216 t



Photo 1. Hydroelectric power plant and dam - downstream view

HYDROELECTRIC POWER PLANT

Tailrace water level ($Q = 60, 120, 280 \text{ m}^3/\text{s}$)	304.06/304.86/306.30 m a.s.l.
Design head	41 m
Average energy equivalent	0.089 kWh/m ³
Number of generating units	2
Turbine rating	21.3 MW
Generator rated capacity	26 MVA
Power factor	0.8
Installed plant power	41.6 MW
Installed discharge	2 x 60 m ³ /s
Turbine speed	187.5 min ⁻¹

Weight:	one turbine	460 t
	one butterfly valve	111 t
	penstock and branches	183 t
	hydromechanical equipment	545 t

Average annual output	120 GWh
Average annual flow	42 m ³ /s
Average annual inflow volume	1,330 hm ³

GEOLOGICAL DATA

A part of the Cetina valley where the reservoir is situated is generally cut into the limestones and dolomites. The bottom and part of the flanks are covered with marl, and the considerable area at the upstream reservoir end contains the ingress of the plastic shales.

The limestones are generally layered, laminated, well developed, of considerable thickness and reach, while the limestones in the vicinity of the dam site are fractured with numerous sharp scarps and cracks. The intensive process of erosion resulted in considerable quantities of red clay. The limestones contain lenses of chert.

The dolomites are generally well layered, weathered on the surface, and considerably internally crystallized, very sound and dense.

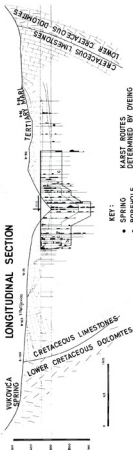
The basic underlayer of all these layers are shales which underlay the complete area.

Stratigraphically, the shales are Triassic, limestones and dolomites Jurassic and Cretaceous, and marls Neogene.

The basic structure in the reservoir area is a large syncline which contains a number of additional forms. The syncline stretches in the Dinaric direction, and its axis approximately corresponds with the Cetina basin axis.

Hydrologically, the undisturbed marls and dolomites form a barrier which is the basis for impermeability of the entire reservoir. Due to their position, the older watertight layers under the Lower Cretaceous dolomites played an important role in the geological history because they reduced the groundwater flows in the dolomitic area and thus protected them against intensive karstification processes. Due to karstification, the Cretaceous limestones are the permeable medium, and the highest permeability was detected by the dam profile.

LONGITUDINAL SECTION



LAYOUT

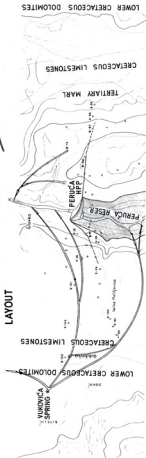


Fig. 5. Hydrogeological relations at the dam profile

The main problem in the left Cetina basin end was flooding of the strong karst springs whose inflows break along the transverse tectonic fractures through the Lower Cretaceous dolomite flanks. The investigations have proven that the groundwater tables in the back of the left Dokomite block are high above the reservoir level, and there is no possibility of water losses by this route.

HYDROLOGICAL DATA

The Peruća reservoir stores the water of the Cetina River and its tributaries at the Derven and Peruća profiles. The dam is located 76.2 m upstream the Cetina mouth into the Adriatic sea, and the normal water level reaches the bridge by the village of Vinalić, some 99.3 km far from the dam. Almost all Cetina tributaries are on the left end, and the majority is submerged even at the lower level of 360.00 m a.s.l.

Direct and indirect catchment area	1,250 km ²
Mean forty-year period discharge (1926-1965)	42.00 m ³ /s
Average precipitations	1,161 mm
Mean annual inflow volume	1,330 hm ³
Q ₉ /Q ₉₅ ratio	2.86
Minimum recorded discharge	0.166 m ³ /s
Maximum recorded discharge	268 m ³ /s
Floods:	
probability 0.0001	510 m ³ /s
0.001	427 m ³ /s
0.01	344 m ³ /s
0.1	259 m ³ /s

RESERVOIR

The Peruća reservoir is situated in the upstream stretch of the Cetina River course. The reservoir bottom elevation on the dam profile is 300.00 m a.s.l. The reservoir level is 69 m. With the capacity of 541 million m³ it maintains water balance for the use of power potential of the Cetina catchment (Peruća HPP, Dale HPP, Zakušac HPP) which exceeds 2.5 billion kWh a year.

It plays an important role by preventing flooding of the downstream Hrvatačko and Sinjsko polje fields with surface area exceeding 15,000 ha. The Peruća is the first Croatian reservoir in karst, realized after numerous geological and hydrogeological investigations.

A part of the Cetina valley in which the reservoir is situated extends along the syncline with core consisting of cretaceous limestone layers more than 2000 m thick. Underlying it, at the bottom and in the flanks, are poorly permeable thick layers of Lower Cretaceous dolomite. On the upstream and downstream side this syncline structure is dammed up by impermeable Werferian shales. These impermeable layers are an outer frame of limestone layers where a bed of the Peruća reservoir is situated. This outer frame was very important in forming hydrogeological relations in the limestone layers, directing the underground flow and therefore the development of karstification towards the Cetina valley.

One of the basic problems on which depended the success of the project was how to impound the strong karst springs below the reservoir level, especially because the total yield of these springs exceeds two thirds of the water inflow into the reservoir.

The dam site flanks are cut and made watertight by a grout curtain. Its route and depth have been chosen based on analysis of the results obtained from exploration boreholes.

The grout curtain is 1600 m long and 100 to 200 m deep. It was realized by 164,000 grouting boreholes. 51,394 tons of cement and other grout mix components was spent. Specific consumption of the dry matter in the grout mix was around 198 kg/m³ of the curtain. With the highest reservoir level, the seepage from reservoir is approximately 1.2 m³/s. The average annual seepage is around 0.7 m³/s which is 1.7% of the average annual inflow.

The grout curtain was fully efficient in cutting off of the main underground karst drains and fissure zones, thus preventing large concentrated underground outflow. It has also been confirmed that the Mt. Svilaja massif is impermeable, so there is no water seepage towards the Čikola River.

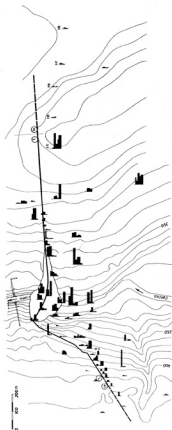
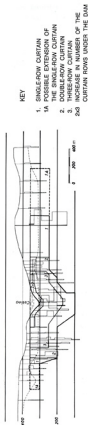
PERUĆA ROCKFILL DAM - before explosion

The Peruća Dam is 63 m high, from the lowest point in the riverbed to the crest on El. 363.00 m. The highest operating level is 360.00 m, so the dam crest is 3.0 m above the maximum water level in the reservoir.

The dam consists of the upstream and the downstream rockfills, with vertical clay core between them.

Between the core and rock are transitional filter layers consisting of natural fine sand, gravel and crushed stones.

The narrow core is chosen for the dam tightening because the nearby clay was not of sufficient quantity for the wide core. The high clay plasticity with a lot of fine particles



IMPERMEABILITY TESTING IN BOREHOLES
 $1 \text{ mm} = 2 \text{ l/min/m}^2/10 \text{ atm}$

Fig. 6. Grout curtain

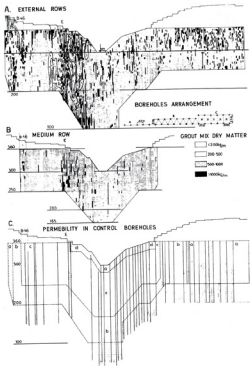


Fig. 7. Dry matter consumption in grout curtain

was favorable for the purpose. A very careful choice of filter particle size for core protection against the hydraulic erosion was mandatory.

The upstream dam slope is covered with the large rock blocks (rip-rap) protecting it from the wave erosion. The rip-rap is roughly placed on the slope profile, without any treatment or special arrangement. The downstream slope is covered with finer stone and is roughly placed in the profile by the strong jet of water directed towards the slope. Material of the slope is thus completely stabilized.

Because of the fissured bedrock in the Cetina bed, a concrete foundation is made with the upper face at El. 305.00 m, which is above the low water of the Cetina downstream the dam. The deepest part of the inspection gallery is thus elevated always to be above the water.

In the layout, the dam is concavely curved towards the downstream side in order to achieve compacting of the core due to the dam strain under the pressure of the water from the reservoir.

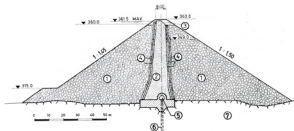
The cofferdams weirs diverting the Cetina thorough the tunnel during construction are made of large stones covered with fine stone and gravel, to be the underlayer for the upstream clay layer preventing the water inflow into the construction pit. Thus built the cofferdams were integrated with the dam body.

A diversion tunnel situated in the right end, which was later converted into the bottom outlet, is also an integral part of the dam. This tunnel is fitted with auxiliary gates driven through the vertical shaft situated by the downstream slope of the dam. At the downstream end of the tunnel an energy dissipator is installed, with a control gate installed upstream. The headrace tunnel, with 8.0 m opening span, is located in the left dam end. During the first construction stage this tunnel was also used for water discharge.

In the left dam end a spillway is located with automatically controlled 5.0 x 10.0 m flap. The water from the spillway flows through a chute along the left valley slope into the Cetina riverbed, where the kinetic energy disperses in the energy dissipator.

For 32 years of its existence the Peruća dam completely fulfilled its humanitarian role. For its designers and constructors it was an exceptional stimulation for future challenges. The dam, the most valuable human achievement, which was tailored to its surroundings so well that it became integrated with it, became a threat to the people and the nature in a brief moment. Unfortunately, it was used by a man to hurt another man.

Fortunately, it was only **the Peruća Dam** that got damaged.



KEY

1. ROCK FILL
2. CLAY CORE
3. CRUSHED CLAYEY ROCK
4. FILTER LAYERS
5. GROUTING GALLERY
6. GROUT CURTAIN
7. BEDROCK

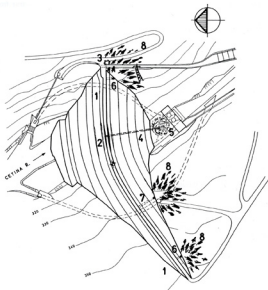
Fig. 8. Peruća Dam cross-section

EXPLOSION

At 10:45 am, on January 28, 1993, the explosive was activated in the rockfill Peruća Dam inspection gallery; this dam is the first in a series of power plants constructed on the Cetina River.

By this terrorist act, a 32-year old structure was destroyed, and yet it proved its quality by resisting the blast blow and retaining 470×10^6 m of water which threatened 20,000 inhabitants living downstream.

There was no information about the destruction scale in the first news, because the war was raging in the wider area around the dam.



KEY

1. CRATERS ON THE RIGHT AND LEFT DAM ENDS
2. SUBSIDENCE IN THE CENTRAL DAM SECTION
3. DESTROYED SPILLWAY STRUCTURE
4. GROUTING GALLERY DRAINAGE PIPE
5. CHUNKS OF CLAY CORE
6. DESTROYED GALLERY ACCESS TUNNELS
7. DAMAGED GALLERY ACCESS TUNNEL
8. BURNED FOREST

Fig. 9. Dam layout with marked damages

The explosion effect, the smoke and dust visible on the film shoot by an amateur from a long distance, indicated the intensity of explosion.

The first insight was got from the shots made by the Split HTV cameraman who succeeded in reaching the dam together with the Croatian Army, and recorded it on a film.

When the situation permitted, at 20:00 pm the same evening, a visit of the civil engineering experts from Split was organized to the dam and the power house.

At that time the fighting moved some kilometers away from the dam, so the first expert inspection was done in the light of torches and under the Croatian Army escort.

The dam was occupied by the Serbian paramilitary troops and the so called Yugoslav National Army on September 17, 1991. When the dam was taken, the enemy closed all the discharges from the reservoir, the spillway flap and the bottom outlet control gate.

This was the onset of the threats to the downstream area between the towns of Sinj and Omiš.

The dam became a detrimental weapon in the arms of the enemy who has proven capable of realizing his threats regardless of the 20,000 inhabitants who were directly imperilled by such an act.

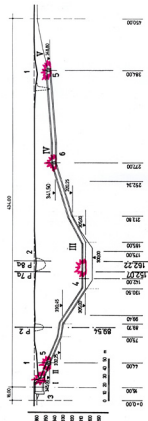
Throughout the period of its occupation, the dam was used to blackmail the whole of Croatia. For that reason, in November 1991, the study titled THE DAY AFTER was prepared by the Croatian National Electricity experts predicting such events.

During the night after the blasting, the decision was brought not to sound the natural disaster alert, thus avoiding moving out 20,000 inhabitants, which could have resulted in panic and possible casualties. Instead, a silent evacuation was ordered of about 3,000 inhabitants from the most endangered area, the villages close to the dam.

The damages on the dam requested emergency action on the dam body, to prevent water inflow and erosion, and opening of the bottom outlet gate to make water discharge from the reservoir possible, thus relieving the dam of the pressure.

During the night, the continuous monitoring of the flow and turbidity parameters was organized on the most accessible hydrometric profile in Han (approx. 12 km downstream the Peruća dam).

The purpose of these measurements was to obtain valuable information for quality estimate of the dam behavior, and therefore provide timely reaction in case of crisis.



KEY

1. CRATERS ON THE RIGHT AND LEFT DAM ENDS
2. SUBSIDENCE IN THE CENTRAL DAM SECTION
3. DESTROYED SPILLWAY STRUCTURE
4. GALLERY DRAINAGE PIPE

5. DESTROYED GALLERY ACCESS TUNNELS
6. DAMAGED GALLERY ACCESS TUNNEL

I, II, III, IV, V ASSUMED BLASTING POINTS

Fig. 10. Peruca Dam longitudinal section

The explosive in the dam was activated in five locations in the grouting gallery, as marked in cross section I-V in the Fig. 10.



Photo 2. Right dam end crater

The seismic stations at Trilj, Hvar and Puntijarka recorded the earthquake with magnitude $M=2.4$, which equals the explosion of 20 to 30 t of TNT.

The explosion caused the damages as could have been anticipated after the visual examination, and later confirmed by investigations.

The explosion opened a huge crater in explosion site V, which is the junction point between the access tunnel No 1 and the inspection gallery in the right dam end (station 3+84), which was the only possible access of the dam crest. The crater was cone-shaped, 20-25 m in diameter and 6-7 m deep. The crater was dry and the water was not noticed either in the crater or on the downstream slope, Photo 2. The access tunnel caved in, and



Photo 3. Transverse crack in the dam crest

the forest in front of the entrance and within the radius of hundred odd meters was damaged and burned by the thrown material and heat of the explosion. The same effects were noticed on the explosion sites II and IV.

A 77 m crack in asphalt was formed along the crest, 1.5 from the right edge, stretching from the crater in the right end towards the central part of the dam. There was another large damage in the left dam end, and it presented the largest threat for the structure at that moment.

The damage was caused by the explosive activated in explosion sites I and II. The explosion in site I happened by the very right spillway retaining wall, and in site II in the junction point of the access tunnel No. 3 and the inspection gallery.



Photo 4. Left dam end and spillway

The effect was complete destruction of the right retaining wall, while the rockfill, Photo 4, was moved from the upstream side all the way to the spillway flap.

The explosion made a crater 25-30 m in diameter, about 10 m deep, in site II, through which water directly entered the inspection gallery. The downstream crater end was only 30 to 50 m above the reservoir water level, while the upstream end was flooded.

These two explosions caused the largest and the most dangerous damages to the dam, threatening with excessive eruption of water into the inspection gallery and spilling over the dam slope.



Photo 5. Crest subsidence in the highest dam section

As the spillway retaining wall was blown up, the remaining parts shifted sufficiently to get the spillway flap stuck in case it was closed and prevent its opening. The water overtopping effects would have increased even at lower water levels.

The explosion was so powerful that the parts of the spillway retaining wall were scattered within the radius of 50 m, and a 12 m piece of was found 30 m far from the explosion site.

The reinforced-concrete bridge over the spillway was thrown onto the left bank, where it remained partly hanging over the retaining wall.

In the deepest part of the inspection gallery, where the dam is the highest, the explosion III caused visible 2.5 to 3 m deep subsidence in the dam crest, stations 0+89.54, 1+52.07, and 1+62.22, Photo 5.

The creeping of the upstream rip-rap was noticed on the upstream slope, and occasional whirlpools indicating the water had burst into the dam body. This part of the upstream slope requested the backfilling from the flank.

The emergency action was started when the Croatian Army permitted access to the dam, at 8:00 am, on January 29, 1993.

The craters were backfilled and the erosion development prevented by D8 bulldozer of the Caterpillar type and mobile loader, and the material was transported from the quarry some 25 km distanced from the dam.

The activities were permitted only along the marked lines, since there still were mines planted around.

The most endangered location in the left dam end was accessible only across the damage (the crater) in the right end.

The remediation of this part was carried out so that the basic dam rockfill on the both sides of the crater was moved into the crater by the bulldozer in order to fill it. The filled crater was covered with the fine material from the borrow area, which enabled the machinery access to the dam crest.

When the other, left side of the dam, was approached the water was visibly entering the inspection gallery in the explosion site II on the access tunnel No. 3 on station 0+44. The water by the spillway wall was 30-50 cm below the top of the downstream dam rockfill. This was the best proof that, due to the fact that the reservoir level at the moment of explosion was 356.20 m a.s.l., the catastrophe was avoided, although the Chetniks were preparing it throughout the period 1991/92 in which the level of the reservoir was 361.20 m a.s.l.

The water level at the time of explosion was crucial for the course of events.

The left end demanded an urgent repair because there was a real danger of water bursting in through the remaining loose rockfill.



Photo 6.
 Peruća HPP
 switchyard 110 kv



Photo 7.
 Drainage shaft at switchya

To backfill the craters, lorries were expected to transport min. 1,000 m³ of material.

Such an operation would demand about 150 lorries full of material. Every run requires 5-minute reverse drive on a 400 m distance, unloading and return to the access road. Therefore, this activity would require 12.5 hours.

Besides, at the time of bringing in and unloading the material, the D8 bulldozer should be removed from the dam and therefore the upstream slope backfilling would be stopped.

Due to the emergency of the action, it was decided on the site that the backfilling be performed only by D8 bulldozer, pushing the material from the upper parts of the dam body to the spillway retaining wall, thus providing faster protection of the rockfill along the spillway wall (explosion I), and then filling the crater through which water entered the inspection gallery (explosion site II).

Backfilling of the crater above the inspection gallery was finished after the rockfill was protected and dam overtopping prevented (11:10 am, that morning).

During the entire operation the lorries were laterally backfilling the damages in the upstream slope.

Until the moment of backfilling the crater which opened above the inspection gallery, water which entered into a body of the dam and the gallery flowed out through the drainage canal shaft of the 110 kV switchyard platform, Photos 6 and 7.

The seepage of the smaller quantities of water was noticed on the downstream slope, in the left dam end, in the point of the caved-in access tunnel, around the elevation 345.00 m a.s.l. On February 5, 1993, when the reservoir water level was 350.62 m a.s.l., the seepage almost completely stopped.

At 11:10 am, after backfilling the craters in the explosion sites I and II, the water outflow from the switchyard drainage shaft slowed and soon stopped, which indicated to clogging of the drainage canal due to decrease in water inflow. It also indicated that the major inflow into the dam body was stopped.

However, about 12:00 the water started flowing through the access tunnel No. 2, Photo 8. The water flow rate and turbidity measurements started immediately. During that day, the flow rate was about 320 l/s, and the maximum flow rate of about 570 l/s was measured on February 3, 1993. The flow rate subsequently decreased, and the seepage completely stopped on February 12, 1993.

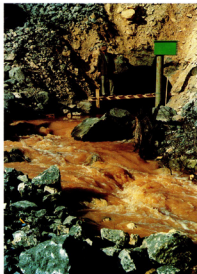


Photo 8. February 5, 1993. - Access tunnel No. 2 - Measuring the water discharge from the gallery

On the explosion sites I and II, a 2 m high freeboard was constructed above the reservoir level, and the emergency material depot (300 m³) in case the emergency action was required on the dam.



Photo 9. February 16, 1993. - Crater on station 1+12.16 to 1+62.61
- Upward view



Photo 10. February 16, 1993. - Crater on station 1+52.16 to 1+62.62
- Downward view



Photo 11. Cavern on station 0+82 to 0+89

The total amount of the material transported for the repair activities was approximately 3,000 m³.

When it was determined that the remediation of the left dam end reduced seepage, opening of the bottom outlet control gate by mining was abandoned. The credit for the action for making the normal opening mechanism operable and its success can be given to the Peruća HPP crew and the Croatian army. The gate was successfully opened without any damage at 14:15 pm on January 29, 1993.

This was a success of the Croatian National Electricity and the Croatian Army. The reservoir emptying at the discharge of 220 m³/s decreased the risk of further dam damage on a daily basis.

The dam was visually monitored on a daily basis by frequent tours of the crest. The dam slopes were checked in detail, since there was a suspicion that the mines were planted in them as well.

Additionally, the dam settlement measurements in 19 profiles set on the crest were started, along with the measurements of the water flow on the upstream dam slope. The preliminary geophysical prospecting was also conducted.

The expert team engaged on the monitoring prepared daily reports, analyzed them and initiated required actions.

On January 31, 1993, the crater approximately 5.0 m in diameter and 4.0 m deep opened in the backfilled section by the dam spillway, on station 0+40. It was immediately backfilled, and the borrow area with 500 m³ of material from the quarry for any future repair works was formed.

On February 16, 1993 a crater approximately 10.0 m in diameter and to 8.00 m deep, Photos 9 and 10, opened between the stations 1+52.16 and 1+62.61 on the downstream end of the dam crest. It was immediately filled with the quarry material. During the next days there were no noticeable signs of anything happening in this part of the dam.

On March 4, 1993, a cavern appeared between the stations 0+82.00 and 0+89.00, Photo 11, in the downstream section. It spread to the half of the dam crest. Initially, the cavern was about 5.0 m wide, 10.0 m deep, but in the days to come it started to expand while becoming shallower due to the caved in material. Since at this point the reservoir level was at El. 325.10 m a.s.l., and ground elevation 330.0 m a.s.l., there was no danger of possible water ingress. This cavern was not immediately filled because the cracks developed even on a distance of 7.0 m from its edge, indicating the dam instability in this point which imperilled the access of the machinery. The subsidences and deformations were also noticed in the upstream slope.

CONCLUSION

The Peruća Dam withstood this war challenge and played a humanitarian role of saving thousands of lives.

REFERENCES

- [1] Vilović M., Zec M., Raos J., The Days After, Split 1993
- [2] Peruća HPP 1960-1985, 25th Anniversary Monograph, Split 1985

THE PERUĆA DAM REMEDIATION SOLUTION

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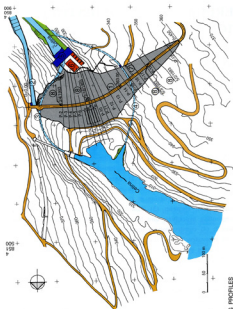
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ABSTRACT

The investigation and the dam behavior observation results after the explosion are described. Based on these data, and the visual inspection of the dam, the four basic dam remediation alternatives were set: (1) dam core section reconstruction along complete length; (2) dam core remediation in central part by construction of a plastic concrete cut-off wall, and reconstruction of the left and right dam ends with clay core as an impermeable dam element; (3) dam core remediation in its central part by grouting and jetgrouting procedure, and the left and right dam ends reconstruction as in the Alternative 2; (4) construction of upstream impervious screen as an impermeable dam element, and reconstruction of the left and right dam ends by rockfill placement. The sub-alternatives were prepared for the alternatives (1) and (4) with respect to the impermeable dam element material, core and the upstream screen. Altogether seven alternatives were considered, their costs and construction schedules determined, along with the power generation losses for the duration of the construction works. The qualitative and quantitative comparison of the considered alternatives was the basis for selection of the most favorable solution of the dam remediation - construction of the plastic concrete cut-off wall and reconstruction of the left and right dam ends as the impermeable dam elements.

INTRODUCTION

It had been planned to base the dam remediation solution on the results of comprehensive investigation works which had been expected to give as precise determination as possible of the damage location and scale in each of the dam sections, and particularly in the clay core. However, due to the permanent threat of enemy attacks, only the dam settlement monitoring was carried out. Therefore, the dam remediation solutions were based on the visual inspection, the dam behavior monitoring results obtained during several days immediately after the explosion, and settlement monitoring.



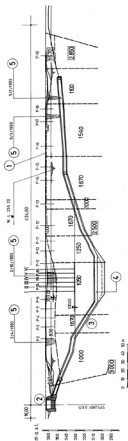
KEY

1. DAM
2. SPILLWAY CHUTE
3. HEADRACE TUNNEL
4. BOTTOM OUTLET TUNNEL
5. PERUĆA HPP
6. SWITCHYARD
7. DAMAGED GALLERY ACCESS TUNNEL
8. SUBSEQUENTLY CAVED-IN CRATERS

P-2 TO P-19 SETTLEMENT MONITORING PROFILES

1 TO VI WATER FLOW VELOCITY RECORDING PROFILES

Figure 1 Peruća Dam layout



KEY

1. DAM CREST
2. SPILLWAY
3. GROUTING GALLERY
4. DRAINAGE CANAL
5. SUBSEQUENTLY CAVED-IN CRATERS

340 - 1670	LONGITUDINAL SEISMIC WAVE VELOCITY (m/s) IN DAM BODY
1670 - 2000	LONGITUDINAL SEISMIC WAVE VELOCITY (m/s) IN BEDROCK
P-2 TO P-19	SETTLEMENT MONITORING PROFILES
I TO VI	WATER FLOW VELOCITY RECORDING PROFILES

Figure 2 Peruca Dam longitudinal section

The largest visible damage was detected in the left dam end where the spillway structure was completely damaged, while in the dam crest, to the right of the spillway, a dia. 25-30 m and about 10 m deep crater was open through which water directly flowed into the grouting gallery. Movement of rock material was noticed in the upstream slope, along with occasional whirls, which indicated that the water was penetrating the dam body.

The blasting of the central section caused 2.5 to 3.0 m settlement in a 50-60 m long stretch. In the right end a 20-25 m in diameter and 6-7 m deep crater was formed.

RESULTS OF THE DAM BEHAVIOR MONITORING AFTER EXPLOSION

The geophysical prospecting, measurements of the water flow velocity by the upstream dam slope, and flow rate and turbidity of water running out of the grouting gallery access tunnel were conducted immediately after the mining.

Geophysical prospecting was carried out by the "MOHO" company for applied geophysics from Zagreb.

The geophysical prospecting was carried out in order to determine the locations and extent of the dam damage. The methods of seismic refraction, self potential, and resistivity profiling (mapping) were applied, Figs. 1 and 2.

The velocities of the longitudinal seismic waves enabled general separation of the dam sections which suffered major damage, i.e. the top section to the elevation of 5 to 10 m from the crest, left and right ends and the highest central dam section.

The prospecting results were confirmed by the subsequent crater-shaped carving-ins in four points: on February 16, on the central dam section, between the stations 1+52.16 and 1+62.61, diameter about 10 m, depth to 8.0 m; on March 4, in station 0+85.50, diameter about 5 m, depth to 10 m, with tendency of increasing as indicated by the cracks which appeared even 7 m far from the crater; on May 2, in station 3+62.00 and June 3, in station 3+42.00, 4-5 m in diameter, from which the dam material was falling on the grouting gallery through the damaged concrete.

Water flow measurements were carried out by the **Institute of Oceanography and Fisheries** from Split.

The measurements were intended to locate the points in the upstream dam slope, through which the water ingress in the dam body was more or less concentrated.

The measurements were conducted from the top dam section, where the explosion caused the crest subsidence of approximately 3.0 m. This lead us to the conclusion that the

explosive was also planted and activated in this dam gallery section, which caused the cracks and cavities in the upstream shell through which the water was entering the grouting gallery. These would have been the potential internal erosion development sites, causing even more intensive damage and dam collapse with heavy consequences.

The measurements were carried out on February 2 to 5, 1993, when the water discharge from the gallery was maximum, Fig. 3.

The measurements were carried out on a 25 m long stretch along the dam, located some 35 m in the direction of the reservoir, between 353 and 330 m a.s.l., Fig. 1 and 2. The measurement results indicated that the water was generally flowing in the SW-NE direction, namely towards the left dam end. But deviations from this pattern were observed on elevations between 330 and 340 m a.s.l., where the flow direction was perpendicular to the dam axis. However, no concentrated water ingress into the dam was detected at the measurement site.

Flow rate and turbidity monitoring of water discharged from the access tunnel. The monitoring was carried out by the **Civil Engineering Institute** from Split.

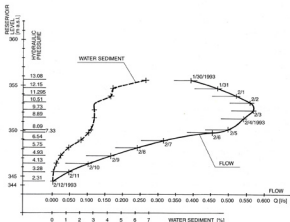


Fig. 3 Water discharge and turbidity diagram

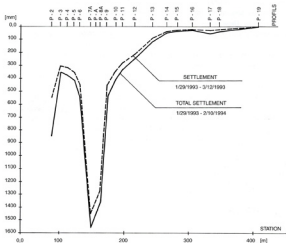


Fig. 4 *Dam settlement in profiles*

The measurements have shown that during the period between January 30, 1993 and February 3, 1993 the flow rate increased from approximately 400 l/s to 570 l/s. Simultaneously, the reservoir water level (namely the hydrostatic pressure) decreased for 3.35 m, as well as the water turbidity, Fig. 3. Since then, the flow decreased continually, and the water seepage ceased completely on February 12, 1993. At that moment, the reservoir level was on El. 344.41 m a.s.l., namely the difference in elevation of the reservoir level and the access tunnel bottom was 2.31 m. The water turbidity continually decreased, so the water outflowing the last day was completely clear.

Based on these data, it was concluded that there was no intensive internal erosion development in the dam body. Based on the discharged water volume and sediment concentration it has been calculated that 1,500 m³ of clay has been washed out from the dam by this route.

Dam settlement monitoring was carried out continually by the **Surveying Bureau** from Split from January 29, 1993 to February 10, 1994, namely for 377 days.

The monitoring was carried out in 19 profiles and in three monitoring points set on the dam crest i.e. on the upstream edge, centrally, and at the downstream dam crest edge. The profiles were set in the dam crest section which had not been completely destroyed by blasting, so that the settlement monitoring covered about 290 m or about 70% of the total dam crest length.

The dam settlement data, Fig. 4, indicate that the settlement was the most intensive in profiles P-7a and P-8a, to 1.554 m and 1.367 m, respectively, which is 26.79% and 23.57% of the dam height, and in the profile P-2, where the settlement was 0.846 m, namely 24.88 % of the dam height, Fig. 5.

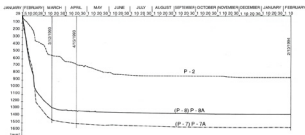


Fig. 5 Dam settlement at crest centre in profiles P2, P-7A, and P-8A

These data revealed that the dam suffered the major damage in these sections. This particularly regards the profiles P-7a and P-8a where, before blasting, the dam crest was on elevation = 363.80 m a.s.l., and after blasting on elevation 361.345 m a.s.l., namely 361.407 m a.s.l. Therefore, the initial settlement at the moment of explosion was approximately min. 3 m, since the benchmarks levelling was carried out after backfilling this part of the dam crest to evaluated 0.5-1.0 m to enable vehicle traffic to the left dam end.

The dam settlement analysis was carried on the settlement diagrams for particular profiles in order to define the general time trend. The diagrams were transformed into the coordinate system in which the relative settlement as compared to the dam height in the monitoring profile $s [‰]$ was plotted on the ordinate, and $\sqrt{t}$ where t is monitoring duration in days was plotted on the abscise, which gives the settlement data concentration on a time axis.

To illustrate the analysis and the obtained results, a characteristic settlement diagram for the profile P-4 was selected. In this profile, the dam height is 48 m, namely about 83% of its maximum height. The upstream slope bottom elevation in this profile is 315 m a.s.l., which corresponds with the lowest reservoir level recorded during its discharging after the dam blasting. The settlement diagram with absolute settlement values and calendar time axis is given in Fig. 6, and the transformed settlement diagram in coordinate system " $\sqrt{t}$ ", " s " [See Fig. 7].

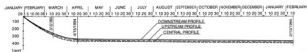


Fig. 6 Dam settlement in profile P-4

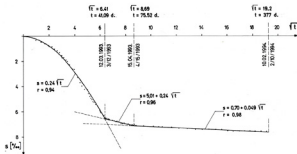


Fig. 7 Dam settlement in profile P-4

The regression analysis results have shown three dam settlement time intervals where the relative settlement trend during the first period can be approximated with a parabola, and the other two with straight lines with high coefficient of correlation ranging between $r = 0.94$ and $r = 0.98$.

The point of intersection of the parabola and the first line is defined as the end of the first period and corresponds with $t = 41$ day, which is well in concordance with the completion of the reservoir discharging which lasted for 42 days. The end of the second period is defined by the intersection of the lines corresponding with $t = 76$ days, so that the second dam settlement period lasted for about 34 days, while the third period lasted until the end of the monitoring, i.e. 301 days.

The results of the settlement monitoring diagram analysis have shown that three periods are generally characteristic for the dam settlement duration:

- From January 29 to March 12, 1993, which corresponds with the reservoir discharge to EL 314.80 m a.s.l. (42 days); the reservoir volume at this elevation was about 10 million m^3 , namely about 2% of the total volume and actually presents an empty reservoir.
- From March 13 to April 15, 1993 (34 days);
- From April 16, 1993 to February 10, 1994 (301 days), namely until the monitoring was completed.

The period of the reservoir discharge could be considered the period of the primary post-explosion dam consolidation which was accelerated by increase in effective vertical stresses and decrease in pore pressure in the clay core caused by the reservoir level drop. During this period, the settlement was the most intensive and reached almost 85% of the total in all profiles except for the profile P-2, P-7a, P-8a, P-17, P-18.

During this period, the settlements on profiles P-2, P-17 and P-18 were 65%, 67% and 56% of total, respectively, which leads to a conclusion that the primarily post-explosion dam consolidation in this section has not been achieved during the reservoir discharge.

Quite contrary, the settlement on the profiles P-7a and P-8a were 94% of total, although the visible damage indicated that the explosive was blasted right in this section of the grouting gallery.

However, the settlement rate on these profiles was the highest during this period, i.e. 35 and 31 mm/day, respectively, while the settlement rate of the adjacent profiles P-6 and P-9 was about three times lower, i.e. 11 mm/day. The settlement rate on individual profiles towards the left and right dam end was continuously descending, with the exception of the profile P-2 with 13 mm/day.

During the period between March 13, 1993 and April 15, 1993, the dam settlement was considerably decelerated in comparison with the period of discharge, i.e. about 6.5% of total settlement, while the settlement rate was about 0.6 mm/day or about 23 cm/year.

The exceptions were profiles P-2, P-17 and P-18 on which the settlement was 14.7, 19.3 and 16.3% of total, respectively, and the profiles P-7a and P-8a with 2.7% and 2.9% settlement of total, respectively.

The settlement rates on profiles P-2, P-7a, P-8a, P-17 and P-18 were 3.65, 1.24, 1.15, 0.3 and 0.2 mm/day, namely 133, 45, 42, 11 and 7 cm/year, respectively.

The comparison with the dam settlement rate after its completion in 1960 (25 cm/year) shows that on about 75% of the length on which the dam settlement monitoring was performed, the same level of consolidation was achieved as at the time of the construction completion. The settlement rate on the profile P-2 of 133 cm/year, and on profiles P-7a and P-8a of 45 and 42 cm/year, respectively, indicated a considerable damage in this dam section.

During the period between April 16, 1993 and monitoring completion on February 10, 1994, the dam settlement on a 75% of the monitored stretch was 8.5% of total, and the settlement rate was about 0.09 mm/day, namely about 3 cm a year.

The dam settlement on the profiles P-7a and P-8a was 3.3 and 3.1% of the total, at rate of 0.18 and 0.14 mm/day, namely 7 and 5 cm/year, respectively. The settlement on the profile P-2 was 20.3% of the total, the settlement rate was 0.56 mm/day, namely 20 cm/year.

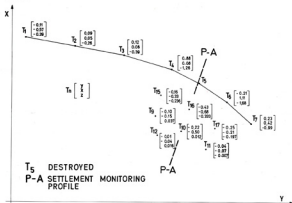


Fig. 8 Benchmark displacements in the downstream dam slope compared to 1991

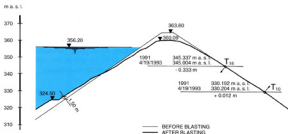


Fig. 9 Dam cross section before and after explosion in profile P-A

The above data lead to the conclusion that dam material texture which was considerably disturbed by the blasting reached the state of secondary consolidation at a quite uniform settlement rate, with the exception of the profile P-2 on which the settlement rate was in approximate concordance with the dam settlement rate recorded two years after the dam completion.

The goodetk control of the dam displacement benchmarks minimum deformations in the downstream slope were noticed as compared to the measurement conducted in 1991, Figs. 8 and 9.

However, the visual inspection after the reservoir discharge has shown considerable deformation in the upstream dam slope.

Conclusions

The results of the above investigations and measurements lead to the below conclusions:

- (1) The major damage caused by the dam blasting was suffered by its top section, in the height of 5-10 m along the entire length, in the left dam end to the profile P-3 in the approx. 80 long stretch to the spillway, in the highest central dam section between the profiles P-6 and P-9, and the right dam end between the profiles P-17 and P-19 in the length of approx. 80 m.

- (2) According to the settlement results the dam, the material texture of which had been seriously disturbed by the blasting, has generally reached the stage of primary and is presently in the stage of secondary consolidation.
The downstream supporting zone (rockfill shell) suffered no serious damage and shall take over the function of the dam support structure.
The clay core damage is of such a nature that it cannot assume the role of the impermeable dam element.
- (3) It has been assumed that the grouting gallery had been completely damaged in the left dam end, in the length of approximately 45 m from the spillway, in the highest central dam section in the length of 25-30 m, and in the right dam end between the profiles P-18 and P-19 in the length of 25 m. Otherwise, the gallery had been damaged to a greater or smaller extent.
- (4) The right spillway wall, access bridge and the operating building have suffered the damage which requests their full reconstruction. The spillway gate and its driving mechanism are not operable and should be replaced with the new ones.

BASIC DAM REMEDIATION CONCEPTS

The dam post-explosion behavior monitoring and visual inspection results were used as the basis for remediation alternatives which were an outcome of the below concepts:

- (1) In the left dam end, reconstruct the central, core section of the dam;
- (2) In the right dam end, due to small height, reconstruct the complete dam cross section;
- (3) Reconstruct the upper dam part along the entire length and to the height of 5 to 10 m;
- (4) Reconstruct the dam core in the central section in order to increase its impermeability;
- (5) The downstream rockfill shell is not seriously damaged and could assume the function of the dam supporting section;
- (6) Reconstruct the grouting gallery, from the access tunnel in the left dam end to the access tunnel in the right end ;
- (7) The civil works on the spillway shall follow the original design documentation, while the complete hydromechanical equipment shall be replaced;
- (8) Take into consideration the monitoring of the dam behavior after the remediation works have been completed.

- (9) Because the Peruća reservoir is of particular importance for the Cetina River Hydro Power System, the dam remediation should be completed within the shortest possible period of time under the guaranteed security preconditions and with required quality of work.
- (10) Additionally, the Cetina River flood wave should be discharged through the bottom outlet and Peruća HPP turbines during the remediation works, so that the hydro potential be at least partially used. However, the care should be taken that the remediation works would not be obstructed and the security precautions should be undertaken.

Four basic remediation alternatives have been established, i.e.

1. Dam core section reconstruction along complete length;
2. Dam core remediation in central part by construction of the plastic concrete cut-off wall, dam core section replacement in the left dam end and reconstruction of the complete cross-section in the right dam end, according to the original design documentation containing the solution with clay core as the impermeable dam element;
3. Dam core remediation in its central part by grouting and jetgrouting procedure, while the left and right dam ends would be reconstructed as in the Alternative 2;
4. Construction of upstream impervious screen as the impermeable dam element, replacement of the core section in the left end and reconstruction of the complete cross-section of the dam in the right end by rockfilling.

The alternatives 1 and 4 had sub-alternatives with respect to the impermeable dam element, core and upstream screen material. Some of the above alternatives had sub-alternatives, so that altogether 7 alternative solutions were considered. All the analyzed solutions were detailed to the same level in order to enable the most objective quantitative and qualitative evaluation and selection of the most favorable solution.

DESCRIPTION OF ALTERNATIVES

Alternative 1

This alternative is the most radical operation which actually proposes construction of a new dam. Two solutions have been discussed for the impervious element, i.e.

- (a) clay core, as in the original design,
- (b) asphalt-concrete core, Fig. 10.

KEY

1. ASPHALTIC-CONCRETE CORE
2. FILTER LAYERS
3. CRUSHED CLAYEY STONE
4. ROCKFILL
5. CLAY CORE
6. EXISTING ROCK FILL

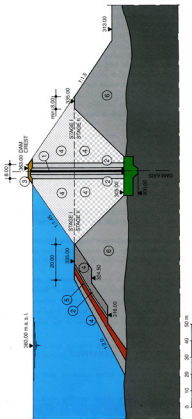


Fig. 10 Alternative 1b Dam reconstruction including asphaltic-concrete core - cross-section

The alternative with the asphalt-concrete core has been analyzed because the on-site climatic conditions are more favorable for the asphalt-concrete than for the clay core; also, it has been assumed that quite a large quantity of clay should be transported from the clay deposit some 10 km far from the site. These could be the reasons of higher cost-effectiveness of the asphalt-concrete core alternative.

This remediation alternative has been planned for execution in **four stages**, Fig. 10.

The **first stage** includes the dam material removal along the entire length to the El. 335 m a.s.l., material selecting and its removal to the dump. Portion of the rock and clayey material is transported to the upstream dam slope, to form the upstream cofferdam enabling utilization of the reservoir to the level of 330 m a.s.l. in dry, and to the water level of 320 m a.s.l. in the rainy season of the year and protecting the site from the 100-year Cetina river floods. Additionally, the water seepage into the dam core section is prevented.

During this stage of works, the damaged grouting gallery is also supported to prevent caving-in of the dam sections under the load of construction machinery.

During the **second stage**, the dam core section is excavated to the grouting gallery bottom level, namely to the foundation ground. Additionally, the damaged gallery is knocked down in this stage.

During the **third stage**, the grouting gallery is constructed in sub-alternative with the clay core, and the concrete underlayer placed the asphalt-concrete sub-alternative. The contact grouting of the grouting gallery bottom, namely concrete underlayer in the alternative with the asphalt concrete is to be carried out as well as the grouting of the grout curtain in the sections requesting it according to the results of the impermeability inspection.

The **fourth stage** is reconstruction of the dam core section to the El. 335 m a.s.l., and subsequent backfilling of the entire cross-section to the dam crest.

In the **alternative involving the dam clay core**, the reconstruction is carried out according to the original design which includes construction of the clay core, upstream and downstream filter layers and upstream and downstream rockfill shells. The material excavated from the dam is used to the maximum possible extent.

In **alternative involving the asphalt-concrete core** includes a 1.0 m wide core, 1.5 m wide upstream and downstream filter layers with grain size of 0-6 cm, 3 m wide transitive filter layers with grain size 0-20 cm, and upstream and downstream dam rockfill shells.

KEY

1. DAM RECONSTRUCTION (5-10 m)
2. CRUSHED CLAYEY ROCK
3. PLASTIC CONCRETE CUT-OFF WALL
4. GROUTING
5. EXISTING GROUT CURTAIN
6. ROCKFILL
7. CLAY CORE
8. FILTER LAYERS
9. RIP-RAP
10. RETAINING WALL

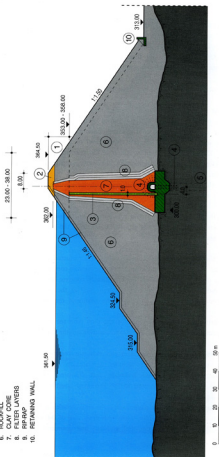


Fig. 11 Alternative 2 Dam reconstruction including plastic concrete cut-off wall
(accepted alternative) - typical cross section

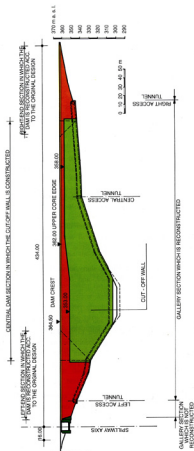


Fig. 12 Alternative 2 Dam reconstruction including plastic concrete cut-off wall (accepted alternative) - typical longitudinal section

Alternative 2

The essential concept of the dam remediation after this alternative is the construction of a plastic-concrete cut-off wall as the impermeable element in the central dam part, and replacement of some 100 m of the dam core section in the left end, and about 100 m of the complete dam cross-section in the right end, with the clay core as the impermeable dam element. In the central dam part, where the cut-off wall is to be constructed, the dam top section is to be reconstructed to the height of 5 - 10 m, with the clay core as the impermeable dam element. The grouting gallery is planned for reconstruction from the access tunnel No. 3 in the left end to the access tunnel No. 1 in the right dam end, Fig. 11 and 12.

The plastic-concrete cut-off wall shall have the below basic mechanical characteristics: compressive strength $> 1 \text{ MN/m}^2$, modulus of elasticity $< 400 \text{ MN/m}^2$.

The cutoff-wall is to be constructed in the dam clay core, upstream the grouting gallery, with the 4 m center-line distance. The cut-off wall is 1 m thick, and its depth varies from 13 to 52 m, depending on the foundation soil depth. The cut-off wall is founded in the rock, namely on the concrete slab in the central dam section, and the foundation depth is about 2 m. The cut-off wall - foundation joint impermeability is achieved by grouting from the grouting gallery.

The hydraulic trench cutter is used for its construction from the dam plateau sufficiently wide to provide for simultaneous construction of cut-off wall and running of traffic along the entire dam length. This enables simultaneous reconstruction works on the dam left and right end.

It is planned to grout the core with cement-bentonite slurry along the cut-off wall route. Thus, the explosion-induced cracks in clay are filled, and the danger of slurry losses lowered, which results in cut-off wall trench stability.

In its left end, the dam is to be excavated along the entire cross-section to the El. 345 m a.s.l., namely to the bottom of the access tunnel and grouting gallery joint, while only the core section shall be excavated below this elevation to the El. 330 m a.s.l. In the section where gallery reconstruction is not planned, the foundation slab shall be placed for the clay core, and contact grouting performed and the curtain grouted where it has been determined necessary by the impermeability inspection. The next stage is the dam backfilling to the El. 353 m a.s.l.

The remediation concept for the right end is equal to the concept for the left end remediation, but for the fact that the complete dam cross-section is reconstructed to the El. 358 m a.s.l.

When the cut-off wall is built, and the reconstruction of the left and right dam end completed to the El. 353 m a.s.l. and 358 m a.s.l., respectively, the 5 - 10 m high top dam section is reconstructed.

The blasting caused deformations in the upstream dam slope. Its remediation includes fixing of the unsteady rip-rap, levelling of irregularities and thus achieving the design slide slope ratio of 1:1.45.

The grouting gallery remediation shall be carried out for several reasons. One of the most important reasons is that its reconstruction would actually be the investigation work which would enable good insight into the clay core damage in the area of the highest hydrostatic pressures along the entire dam length, including the mine planting sites.

When reconstructed, the gallery shall enable quality contact grouting of the cut-off wall and its foundation, control of grout curtain impermeability and corrective actions where required.

In exploitation, the gallery shall serve for control of the cut-off wall impermeability and, if necessary, for additional corrective actions in order to prevent seepage.

According to the available data, the grouting gallery concrete is mainly cracked, but the gallery has generally maintained its shape, while in the blasting sites it was completely destroyed.

The gallery reconstruction shall generally be carried out in two ways. In sections where the gallery is most caved-in, supporting with steel arches is planned, the concrete lining by 15 cm thick shotcrete reinforced with double steel mesh shall be carried out as well as the grouting of the contact surfaces between and the clay core, namely the foundation rock.

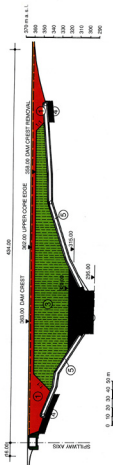
In damaged gallery sections, the excavation shall be carried out under the protection of steel arches and panels, and the 80 cm thick lining be constructed.

The gallery reconstruction is the priority dam remediation activity which should be carried out during the dry season when the reservoir water level can be maintained as low as required. In this way, the water seepage into the damaged gallery would possibly be prevented or the seepage water quantity reduced to a minimum. The water seepage would considerably hinder the works, particularly in the far bottom section where the gallery is assumed to be completely destroyed in an approximately 25 to 30 m long stretch.

This dam remediation alternative, in which the reservoir would be used to the El. 330 m a.s.l., which is also the lowest trench bottom elevation in the left dam end, enables discharge of the 100-year flood water through the bottom outlet and turbine conduits assuming the gallery has been reconstructed.

Alternative 3

The basic dam remediation concept is that the clay core in the central dam part be remediated by jetgrouting and grouting, while the left and right dam ends should be reconstructed as in Alternative 2. The grouting gallery would not be reconstructed in the



KEY

1. DAM RECONSTRUCTION ACCORDING TO ORIGINAL DESIGN
2. JETGROUTING
3. CLAY CORE GROUTING
4. GROUTING
5. GALLERY RECONSTRUCTION

Fig. 13 Alternative 3 Dam reconstruction including core grouting - longitudinal section

central highest dam part, in order to reduce the remediation costs, while in other dam sections the remediation would be as in Alternative 2, Fig. 13.

The jetgrouting method would be applied in order to consolidate the dam clay core from the gallery bottom to the El. 320 m a.s.l. It has been planned to place four rows of dia. 1.5 to 2.0 m cylinders of irregular shape in the cross-section, which would form a 6.5 m wide impermeable dam element.

Above El. 320 m a.s.l., the dam clay core remediation is to be realized by upward grouting from the crest with cement-bentonite mix with addition of sand, in 3-5 m sections, either by gravitation or under low pressures.

The three rows of 2.25 m distanced boreholes are planned, placed in the direction of the dam axis and with the distance of 3 m between the boreholes; the boreholes in the middle row are offset for 1.5 m from those in external rows. In this way the 6.5 m wide remediated strip of the clay core is formed.

The dam core section remediated by grouting, and the one remediated by jetgrouting, shall be bonded by extending the grouting boreholes for 5 m and by grouting of the contact strip.

The curtain impermeability control and contact grouting in the central dam part, where the gallery will not be reconstructed, shall be carried out through the external rows of grouting boreholes.

The upper dam section shall be reconstructed in this stretch to the height of 5 m, according to the original design.

In this dam remediation alternative, by using the reservoir to El. 330 m a.s.l. (which is the elevation of the trench bottom in the left dam end), and assuming the gallery has already been reconstructed, the discharge of the 100-year flood water through the bottom outlet and turbine conduits shall be enabled.

Alternative 4

The dam remediation concept is based on construction of a screen placed on the upstream dam slope as its impervious element instead of the clay core, while the dam should be reconstructed in its left and right ends without the clay core. The grouting gallery should be reconstructed as in Alternatives 2.

The material for the upstream screen could be selected out of the three alternatives, i.e. **reinforced concrete slabs, clay, PEHD foil.**

KEY

1. CLAY CORE
2. FILTER LAYERS
3. ROCKFILL
4. CRUSHED CLAYEY STONE
5. RECONSTRUCTED EXISTING GALLERY
6. EXISTING GROUT CURTAIN
7. DRAIN PIPES
8. NEW GROUT CURTAIN

9. NEW GROUTING GALLERY
10. UPSTREAM SLOPE REINFORCED-CONCRETE LINING (10x15 m SLABS)
11. LEVELING ROCK MATERIAL LAYER

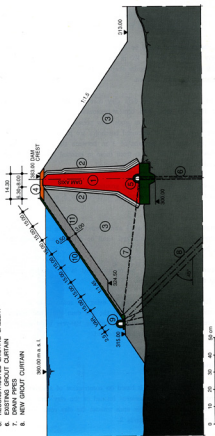


Fig. 14 Alternative 4a Dam reconstruction including upstream reinforced-concrete screen - typical cross-section

KEY

1. CLAY CORE
2. FILTER LAYERS
3. ROCKFILL
4. CRUSHED CLAYEY STONE
5. RECONSTRUCTED EXISTING GALLERY
6. EXISTING GROUT CURTAIN
7. DRAIN PIPES
8. NEW GROUT CURTAIN

9. NEW GROUTING GALLERY
10. CONCRETE SLAB
11. LEVELING ROCK MATERIAL LAYER

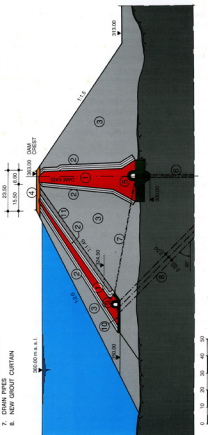


Fig. 15 Alternative 4b Dam reconstruction including upstream clay screen - typical cross-section

In the lowest dam section, the screens are founded on the former upstream cofferdam crest which is on El. 315 m a.s.l.

The impermeability of the dam section going to the Cetina riverbed bottom, and of the joint between the screen and the existing grout curtain, shall be achieved by the new inclined grout curtain descending from the upstream cofferdam crest at the angle of 45° along the entire upstream dam perimeter. In the deepest point the curtain length is about 125 m.

The curtain is of the three-row type, with 4.0 m apart boreholes, and 2.5 m distanced rows. The part of the curtain passing through the upstream cofferdam material is realized with the high-density grout mix and applying the prepack method, while the remaining part would be realized by upward grouting in 5 m sections.

The subsequent grouting during reservoir operation could be carried out from the new grouting gallery along the entire upstream dam perimeter.

Since the upstream dam slope is protected with rip-rap, and the slope is interrupted with some 5 m wide bank on El. - 324.5 m a.s.l., the slope shall be levelled with 3 m thick layer of compacted stone in order to obtain flat, undeformable and sufficiently permeable screen base.

The **reinforced concrete screen** shall be place on 50 cm thick drainage layer and made of 10×15 m, 50 cm thick double-reinforced slabs, connected with seals and putty of permanent elasticity. The slabs shall be poured on-site, in a sliding formwork, Fig. 14. Along its perimeter, the screen leans against 1 m thick concrete slab on the grouting gallery bottom.

The **clay screen**, Fig. 15, shall be placed on a 4 m thick three-layer filter consisting of finely sized gravel, sand and sandy clay. In its bottom part, vertically to its plane, the clay screen thickness is 4.5 m, and in its top part it is 1 m thick. The protection from the top end is with 2 m thick two-layer gravel and sand filter, and rip-rap.

The clay screen requests the most thorough works. The screen stability requests decrease in the upstream slope inclination by widening of the upstream cofferdam against which the screen is leaned.

The **PEHD foil**, Fig. 16, is laid on a 50 cm thick drainage layer. The lower surface is fitted with spikes preventing the foil to slide down the slope. On the upper end the foil is protected by 10×10 m 15 cm thick plastic-fiber reinforced shotcrete slabs. The slabs are separated from the foil by geotextile, and the construction joints are closed with plastic putty. In the lower end, the foil is embedded in the concrete slab of the grouting gallery, and in concrete threshold in the dam crest, Fig. 16.

KEY

1. CLAY CORE
2. FILTER LAYERS
3. ROCKFILL
4. CRUSHED CLAYEY STONE
5. RECONSTRUCTED EXISTING GALLERY
6. EXISTING GROUT CURTAIN
7. DRAIN PILES
8. NEW GROUT CURTAIN

9. NEW GROUTING GALLERY
10. PEHD FOIL
11. SHOTCRETE SLABS (10x10 m) ON PEHD FOIL
12. LEVELING ROCK MATERIAL LAYER

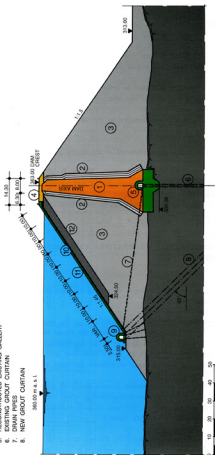


Fig. 16 Alternative 4c Dam reconstruction including upstream PEHD foil screen - typical cross-section

In all the above alternatives, the screen shall be drained through drain pipes drilled-in from the reconstructed grouting gallery to the drainage layer below the screen. The pipes are 10 cm in diameter and 10 - 80 m long. The seepage water is collected in the grouting gallery collector ditch, and conveyed through the reconstructed drainage canal below the downstream dam rockfill shell into the existing drainage system.

The left and right dam ends shall be remediated to the same extent as in Alternatives 2 and 3, as rockfill dam without clay core.

The grouting gallery is reconstructed to the same extent and in the same way as in Alternative 2.

The top dam section is reconstructed in the central part to the height of 5 m, as a rockfill.

Utilization of the reservoir to the El. 315 m a.s.l. enables discharge of 25-year Cetina River floods during July and August. This means that the works on the upstream cofferdam crest, under the specified security precautions, could be performed only during the two months of a year.

DAM REMEDIATION ALTERNATIVES COMPARISON AND ALTERNATIVE SELECTION

Comparison of the dam remediation alternatives has been completed applying quantitative and qualitative analysis.

The quantitative analysis has been carried out on the basis of economic evaluation of alternatives by determining the present value of the Peruća Reservoir contribution to power generation over the period of 50 years in the downstream hydroelectric power plants of the Cetina Hydro Power System, i.e. Peruća HPP, Dale HPP and Zakušac HPP. From this value, the costs of each particular remediation alternative are deduced.

The civil works cost and remediation duration have been determined on the basis of scope of each particular job and remediation schedule which encompasses all the work stages, columns (2) and (3), Table 1. The costs of each remediation alternative are added the 10% current interest, column (4), Table 1.

The dam remediation costs, column (6), Table 1, include the costs of civil works, and the value of lost power generation during remediation works for each discussed alternative.

Alternative	Civil costs (USD)	Remediation duration (month)	Current interest (USD)	Lost power value (USD)	Total costs (USD)	Cost ratio	Perú's Reservoir and HPP contribution (USD)	Contribution ratio
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1a	9,303,000	23	1,377,147	29,750,289	40,430,436	2.80	113,466,224	0.81
1b	11,532,000	23	1,707,070	29,750,289	42,989,359	2.98	110,507,301	0.80
2	5,186,000	9	384,252	11,641,418	17,211,670	1.19	136,684,990	0.98
3	4,438,000	7.5	291,028	9,701,181	14,428,209	1.00	139,468,451	1.00
4a	9,689,000	11	884,562	14,228,399	24,428,209	1.72	129,094,699	0.93
4b	14,884,000	18	1,754,817	23,282,835	39,921,652	2.77	113,975,008	0.82
4c	8,467,000	11	773,007	14,228,399	23,468,406	1.63	130,420,254	0.94

REMARK

- Alternative 1a & 1b - dam reconstruction with clay, namely asphaltic-concrete core
 Alternative 2 - plastic concrete cut-off wall,
 Alternative 3 - jetgrouting and grouting,
 Alternative 4a, 4b & 4c - upstream screen of reinforced concrete, clay and PEHD foil

Table 1 Present value of the Perú's Reservoir and HPP contribution to power generation

Annual value of power generation losses during the remediation works consists of power generation losses of Peruća HPP, and decreased value of power generation of Dale and Zakućac HPP's which work under the conditions of natural Cetina River discharge since the Peruća Reservoir shall be out of operation.

The mean annual discharge of the Cetina River in the Peruća Dam profile is about 42 m³/s so, with energy equivalent of 0.049 kWh/m³ for Dale HPP and 0.579 kWh/m³ for Zakućac HPP, this discharge provides for an annual power generation of 64,900,000 kWh in Dale HPP and 766,835,000 kWh in Zakućac HPP. The mean annual power generation of Peruća HPP is 121,175,000 kWh.

With the unit price of electricity generated with and without the Peruća Reservoir of 0.046 USD/kWh and 0.032 USD/kWh, respectively, the value of annual power losses for Dale and Zakućac HPP's amounts to USD 11,644,290. This amount should be added the value of power generation in Peruća HPP of USD 3,877,600, so the total value of the power losses, or the value of the Peruća Reservoir and HPP contribution amounts to USD 15,521,890. This amount varies for particular alternatives, depending on their time schedule, column (5), Table 1.

The present value of the reservoir and Peruća HPP contribution to the power generation for the period of 50 years is USD 153,896,660. This value, reduced by the total remediation costs, gives the present value of the contribution for a particular remediation alternative, column (8), Table 1.

According to the results of quantitative analysis, the most favorable alternative is Alternative 3, where the dam clay core shall be remediated in the central part using the jetgrouting methods, and the left and right dam ends shall be reconstructed according to the original design documentation.

According to the results of the quantitative analysis, the most favorable is Alternative 3, according to which the clay core in the central part is remediated by the jetgrouting procedure and grouting, and the left and right dam ends are reconstructed according to the original design documentation.

The cost-benefit ratio of the Peruća HPP and Reservoir in the power generation for each individual alternative and the most favorable Alternative 3 is given in columns (7) and (9), Table 1.

The most expensive remediation alternative is the one with asphaltic-concrete core, since its value the Peruća Reservoir and HPP contribution to power generation during the 50-year period is 20% lower than for the most favorable alternative, and it is initially three times more expensive than the Alternative 3. There are no great deviations from this

alternative either by the alternative with reconstruction of the dam core section according to the original design with clay core, or by the alternative anticipating the upstream clay screen as the impermeable dam element.

In other alternatives, i.e. Alternative 2 with the plastic concrete cut-off wall and Alternatives 4c and 4a with upstream screens of PEHD foil and reinforced concrete slabs, the value of the Peruća Reservoir and HPP contribution to power generation in 50-year period is 2-7% lower than in the most favorable Alternative 3, while the initial costs are 19-72% higher.

Based on the results of quantitative analysis the Alternatives 1a, 1b and 4b have been excluded from further consideration.

The remediation alternative selection has been based on qualitative analysis of Alternatives 2, 3, 4a and 4c, according to the following characteristics of the alternatives:

- (1) reliability,
- (2) complexity of civil works,
- (3) possibility of subsequent repair actions if necessary,
- (4) possibility of reservoir utilization in power generation during the remediation works.

The characteristics of **Alternative 2** fully satisfy the reliability criteria, since the new dam is constructed in the left and right dam end, and the plastic-concrete cut-off wall the dam core grouting as well as in the central part shall repair all the transverse and longitudinal cracks and cavities in the core and form, together with the clay core an impermeable dam element.

As regards the cut-off wall construction, the construction methods are not too complex, and the Croatian contractors have the required experience.

The subsequent repairs in the bottom section of the membrane would be possible from the reconstructed grouting gallery.

The reservoir could be used to the El. 330 m a.s.l., provided the grouting gallery has already been reconstructed.

Reliability of **Alternative 3** is lower as compared to Alternative 2, since even extensive investigation works would not result in recording of all the dam core damage sites. This solution requests close monitoring of the dam behavior after the remediation, particularly related to the clay core impermeability, and the probability of the subsequent repair works is higher.

This solution requests two clay core remediation methods which decrease reliability and might prolong the construction schedule.

The subsequent repair works are possible only by grouting from the dam crest.

Annual value of power generation losses during the remediation works consists of power generation losses of Peruća HPP, and decreased value of power generation of Dale and Zakućac HPP's which work under the conditions of natural Cetina River discharge since the Peruća Reservoir shall be out of operation.

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alternative either by the alternative with reconstruction of the dam core section according to the original design with clay core, or by the alternative anticipating the upstream clay screen as the impermeable dam element.

In other alternatives, i.e. Alternative 2 with the plastic concrete cut-off wall and Alternatives 4c and 4a with upstream screens of PEHD foil and reinforced concrete slabs, the value of the Peruća Reservoir and HPP contribution to power generation in 50-year period is 2.7% lower than in the most favorable Alternative 3, while the initial costs are 19.72% higher.

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The remediation alternative selection has been based on qualitative analysis of Alternatives 2, 3, 4a and 4c, according to the following characteristics of the alternatives:

- (1) reliability,
- (2) complexity of civil works,
- (3) possibility of subsequent repair actions if necessary,
- (4) possibility of reservoir utilization in power generation during the remediation works.

The characteristics of **Alternative 2** fully satisfy the reliability criteria, since the new dam is constructed in the left and right dam end, and the plastic-concrete cut-off wall the dam core grouting as well as in the central part shall repair all the transverse and longitudinal cracks and cavities in the core and form, together with the clay core an impermeable dam element.

As regards the cut-off wall construction, the construction methods are not too complex, and the Croatian contractors have the required experience.

The subsequent repairs in the bottom section of the membrane would be possible from the reconstructed grouting gallery.

The reservoir could be used to the El. 330 m a.s.l., provided the grouting gallery has already been reconstructed.

Reliability of **Alternative 3** is lower as compared to Alternative 2, since even extensive investigation works would not result in recording of all the dam core damage sites. This solution requests close monitoring of the dam behavior after the remediation, particularly related to the clay core impermeability, and the probability of the subsequent repair works is higher.

This solution requests two clay core remediation methods which decrease reliability and might prolong the construction schedule.

The subsequent repair works are possible only by grouting from the dam crest.

The conditions for utilization of the reservoir are identical with those in Alternative 2.

The lowest reliability is achieved by **Alternatives 4a and 4c**, with upstream screens. One of the reasons is construction of a new, inclined grouting curtain which shall be realized by grouting in the rock applying the prepack method in the area of the former upstream cofferdam. The attention should be particularly focused on the joint with the existing curtain, grouting in highly-permeable surface rock belt where the grouting pressures should be kept low, and achieving sufficient impermeability of the upstream cofferdam constructed of various rock material, including the tunnel excavation material.

Additionally, the curtain is affected both by horizontal and by vertical component of the hydrostatic pressure, which increases the possibility of its damage, namely requests top-quality workmanship which would be difficult to achieve due to the above said reasons. Therefore, there is a realistic risk of seepage through the curtain which could damage the upstream screen due to hydrostatic pressure on its lower section.

Therefore, the upstream screen drainage should be impeccable, with minimum distance between drainage pipes so that damages are easily detected and repaired. Installation of drainage through the upstream dam rockfill shell consisting of various rock material to 80 m wide in the foot, which could cause a number of problems so that it should be considered as one of drawbacks of this alternative.

Reliability in of PEHD foil impermeable screen is higher than the reliability of the reinforced-concrete screen, since quality welded plastic foil joints are easier to achieve than the quality reinforced-concrete slab joints.

The subsequent repairs of the new grout curtain during exploitation are possible from the grouting gallery, and by direct access to the upstream screen with prior lowering of the reservoir level, would it be required by the site conditions and extent of damage.

The reservoir could be used during the remediation works to the elevation to which the new grout curtain, grouting gallery and upstream screen have been completed.

CONCLUSION

Based on the results of quantitative and qualitative evaluation, the remediation alternative including the cut-off wall in the central part and reconstruction of the left and right dam ends according to the original design documentation was found the most favorable. This alternative enables quality construction of the impermeable dam element and does not require extensive investigation works for detection of the clay core damage site and extent. As compared to the results of quantitative analysis, value of the Peruća reservoir and HPP contribution to the power generation in the analyzed 50-year period is only 2% lower than in the most favorable Alternative 3.

One of the basic characteristics of the selected solution for the dam remediation was that all the planned works were also the only detailed investigation works the results



Photo 1. Final dam remediation works

of which, now when the dam remediation is completed and back in operation, can be used for evaluation of correctness and justification of the selected solution.

The reconstruction of the left and right dam end and its upper section to the height of 5 - 10 m was beyond doubt, since the detected crater-shaped damage and the crest deformation, along with the deformation of the upstream and downstream slope, certainly lead to the conclusion that the dam material structure in these sections is disintegrated. Therefore implementation of another remediation method was actually impossible.

However, the reconstruction section lengths for the left and right dam end determined on the basis of the dam settlement monitoring results were confirmed correct, since the material structure in the ends of these sections was not disturbed, Photo 2.

Further, the reconstruction of the subject dam sections confirmed the assumption that the quantity of clay excavated from the existing dam was not sufficient for construction of the new core. Only 63% of the existing clay was usable, while the rest was brought in from a 10 km remote deposit found after a painstaking and time-consuming reconnaissance and location investigation.

Since in the dam bottom part the core is wider, there is no doubt that this percentage could be different. However, it does not contradict the above assumption having in mind the total clay quantity which would in this case be larger, and the additional problems with finding of a new deposit would arise.

Therefore, discussions about the sub-alternative which includes asphalt-concrete core were fully justified.

Reconstruction of the grouting gallery was specially difficult and complex construction project. The anticipated types of reconstruction based on the assumed classification of damage were confirmed as correct, but the evaluation of the reconstruction scope with respect to the damage scale was too optimistic.

It was assumed that about 100 m or 27% of total gallery length was completely damaged, and the remaining 265 m, or 73% of the length included cracked concrete, while the gallery generally maintained its internal form.

However, it has finally turned out that about 186 m or 51% of the gallery was totally destroyed. It was possible to reconstruct the remaining 179 m or 49% of the gallery using the steel arches, a layer of double reinforced shotcrete, and grouting behind the cracked concrete lining.

The considerable increase in the scope of works, practically exclusively manual because of the limited possibilities for use of machinery, caused delay of the gallery reconstruction completion deadline for 10 months.

However, the damage has confirmed that remediation of this part of the dam through the gallery remediation was necessary otherwise this section, which is the most delicate with respect to impermeability of the contact surfaces of various material exposed to the maximum hydrostatic pressures, would always remain uncertain from the aspect of successful remediation of the complete structure.

The need for impermeable contact surfaces was obvious already during the cut-off wall construction, when the disturbed joint of the clay core and the bedrock with the ungrouted gallery joint on the outside perimeter caused leakage of slurry from the cut-off wall trench into the gallery.

Further, the grouting gallery was reconstructed in 79% of the dam length in the zone of the maximum hydrostatic pressures. When the left and right dam end reconstruction was completed, and the cut-off wall constructed at the reservoir level at 353.00 m a.s.l. when the hydrostatic pressure in the top dam section is 85% of the maximum, the water seepage into the gallery was estimated to be about 0.3 l/s. Thus, it may be concluded with sufficient certainty that the gallery remediation was of good quality.

Additionally, one of the basic design preconditions and concepts is confirmed, i.e. that this part of the dam should be successfully remediated and constantly accessible for control and any necessary emergency operation.

In its central part, in about 59% of its total length, the dam core was remediated using the plastic concrete cut-off wall.

The goodness of this solution was confirmed by the core grouting in the cut-off wall activities aimed at filling in the cavities and cracks in clay caused by the explosion, in order to prevent abrupt slurry loss and increase the cut-off wall trench shear strength.

The grouting results lead to the conclusion that the core damages are inconsistent and disperse, both lengthwise and heighthwise. The difference in average slurry consumption in two neighboring boreholes set on a 1.4 m distance sometimes reached 427 l of slurry or about 150 kg of dry matter per 1 m of borehole. The deviations in different grouting sections of the same borehole sometimes reached 2,300 l of slurry or 811 kg of dry matter per meter of a borehole station.

These data indicate that only a continuous impermeable element, such as a cut-off wall, offers a reliable and safe remediation solution for the dam core. This conclusion is confirmed by the above data on the water seepage into the gallery of 0.3 l/s.

Since the grouting works were to a certain degree and scope also the core damage investigation works, the obtained results confirm the conclusion from the qualitative



Photo 2. Left dam end excavation completed

comparison of alternatives that the core remediation in this dam section with exclusive application of grouting would be less reliable, and would request permanent monitoring of dam behavior, particularly for impermeability. The probability of the subsequent repairs would be higher.

During the remediation works, the Cetina floods with maximum discharge of about 270 m³/s were successfully stored in the reservoir and evacuated through the power plant turbines. The reservoir level did not significantly exceed the El. 330.00 m a.s.l. specified on the basis of the minimum dam excavation elevation, gallery reconstruction safety with respect to the seepage quantities, and possible operation of the hydroelectric power plant generating units. The incoming quantity of water into the gallery was the largest while the level was about 330.00 m a.s.l., and its discharge was approximately 130 l/min. It was pumped from the gallery so the activities in the gallery were not obstructed.

After the cut-off wall was finished, the maximum reservoir level was increased. The care was taken about the possibility of the Cetina flood discharge through the power plant turbines and its utilization in power generation, about the protection of the downstream Hrvatačko polje and Sinjsko polje, and undisturbed construction activities on the dam.

Therefore, the maximum impoundment level for July and August was set to 353.000 m a.s.l. and the reservoir volume about 410 million m³ or about 73% of effective volume.

In this way, two months before the remediation works were completed the dam was capable of fulfilling its basic function, regulating of the Cetina River flows in this particularly significant hydropower system.

REFERENCES

- [1] Dadić, V; Grbec B: Measurements of the Water Flow and Temperature in the Peruća Dam Reservoir; International Workshop on Dam Safety Evaluation, Gründelwald, 1993
- [2] Salković A; Dujmić D: Geophysical Investigation at the Peruća Dam; International Workshop on Dam Safety Evaluation, Gründelwald 1993
- [3] Nonveiller E: The Peruća Dam after the Explosion, International Workshop on Dam Safety Evaluation, Gründelwald 1993
- [4] Lombardi G; Deere D: Grouting Design and Control Using the GIN Principle; Water Power and Dam Construction, June 1993

- [5] Vaughan P.R.; Kluth D.J.; Leonard M.W.; Pradoura H.H.M.: Cracking and Erosion of the Rolled Clay Core of Balderhead Dam and the Remedial Works Adopted for its Repair, X Congress ICOLD, Q 36, Montreal 1970
- [6] Chen Y.J.: Grouting for Sealing the Cohesive Fills of Embankment Dams, XIV Congress ICOLD, Q 55, Rio de Janeiro 1982
- [7] Strobl T; Schmid R.: The Behavior of Dams with Asphaltic Concrete Cores During Impounding; Water Power and Dam Construction, March 1993
- [8] Valstad t; Selnes P.B; Nadim F; Aspen B; Seismic Response of a Rockfill Dam with an Asphaltic Concrete Core, Water Power and Dam construction, April 1991
- [9] Hoeg K.: An Evaluation of Asphaltic Concrete Cores for Embankment Dams, Water Power and Dam Construction, July 1992
- [10] Fenoux G.Y.: Ecrans en paroi realises apres edification des remblais. Nouveaux outillages, nouvelles methodes pour travaux neufs ou travaux de reparation, XVI Congress CIGB, Q 61, San Francisco 1988
- [11] Graybill K., Levallois J.: Construction of a Cutoff Wall with the Hydrofraise through the Core of Mud Mountain Dam (USA); XVII congress ICOLD; Q 66, R 49; Vienna 1991
- [12] Filling Materials for Watertight Cut off Walls - Bulletin 51 ICOLD 1985
- [13] Rockfill Dams with Concrete Facing - State of the art, Bulletin 70, ICOLD 1989
- [14] The Peruća Dam Remediation, Investigation Works Program, Elektroprojekt Ltd., Zagreb, March 1993
- [15] The Peruća Dam Remediation, Remediation Alternatives, Elektroprojekt Ltd., Zagreb, June 1993

THE PERUĆA DAM REMEDIATION DESIGN

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ZAGREB

ABSTRACT

Engagement on the damaged Peruća Dam remediation project was a challenge for all participants since there has been no experience with the remediation of this type of structures damaged in war operations anywhere in the world.

The remediation design documentation was prepared according to the state-of-the-art engineering practice and governing regulations. This primarily relates to the dam seismic stability and 10,000-year flood analysis. Based on the results obtained by the analyses, the crest of the remediated dam was heightened by 1.5 m as compared to the original dam. Also, the maximum reservoir level was increased by 1.5 m, which results in additional $24 \times 10^6 \text{ m}^3$ of water, which is about 4% increase.

The basic dam remediation solution consisted of removal and complete reconstruction of the dam ends and the upper part to the height of 5-10 m, and construction of an impermeable cut-off wall in the central dam section which shall take over the function of the damaged clay core.

The present paper is a summary of the project realization and application of the technical solutions.

GENERAL

The activities on development of the final design for the damaged Peruća Dam remediation started in June 1993, immediately after the conceptual design for the dam remediation was approved. The accepted solution planned removal and complete reconstruction of the dam ends and the upper part to the height of 5-10 m and construction of an impermeable cut-off wall in the central dam section to take over the function of the damaged clay core. The reconstruction of a heavily damaged and partly trapped grouting gallery and its bringing into function was also planned.

The design process itself was a great challenge for all the participants, since there has been no experience with the remediation of this type of structures damaged in war operations anywhere in the world.

The difficulties encountered in the initial stages of work were predominantly those caused by the lack of adequate data on the dam body damage scale and nature. The investigation works, expected to give the data on the damage, were planned for April 1993, but were obstructed by the fighting in the area. Therefore, the scope of investigations was narrowed, and they were conducted only during May and June 1994, when the design activities were in full swing. However, some data on the clay core were obtained earlier, during the remediation activities on the grouting gallery which started in August 1993.

In addition to the data on the damage scale and nature, the majority of the original design documentation was missing, while the as-built documentation for the Peruća Dam has never been prepared. The more comprehensive information on the original dam construction, particularly regarding the original technical requirements, was obtained from the participants in the original dam design and construction.

The lack of data caused a situation in which it was necessary to estimate the damage scale and nature, along with the complete condition, particularly on the clay core, and prepare each technical solution on the basis of such estimates. It should be noted that these estimates used as the design data were mainly confirmed after the dam ends and upper part were removed and investigations carried out.

Since the objective was to make the Peruća Dam and reservoir functional within the shortest possible period of time, primarily because of their role in the Cetina river hydropower system, the design and remediation works run in parallel. The design process was organized so that the building site was under full work load, and the design documentation was divided in volumes for independent plant structures which allowed for separate building activities.

Due to a very poor energy situation in Dalmatia, the efforts were exercised during the remediation to maintain the reservoir level at approximately 325.00 m a.s.l., which enabled the Peruća Hydroelectric Power Plant operation.

The design philosophy was that, regardless of the fact that the Peruća Dam operation for more of 30 years prior to explosion was impeccable, the remediation design should follow the state-of-the-art engineering practice. This primarily relates to the dam seismic stability, 10,000-year flood analysis and installation of sophisticated monitoring instrumentation.

INVESTIGATIONS

The investigation works conducted in order to obtain the design data for the Peruća Dam remediation were expected to give the feedback on:

- the condition of the damaged Peruća dam, including the foundation ground and grout curtain;
- the seismological and seismotectonic characteristics of the Peruća Dam site;
- the Cetina river floods at the dam.

The below investigations were carried out for the foundation ground and grout curtain:

- Continuous measurement of the dam body settlement
- Geophysical cross-hole prospecting of core and a part of the foundation ground, and electric logging of the natural gamma radioactivity
- Geophysical prospecting by a seismic polar method along the dam and in a transverse section
- Geotechnical drilling through the core and a part of the foundation ground, with continuous core drilling and taking of undisturbed samples, Photo 1 and 2
- Static penetration tests (SPT)
- Laboratory determination of the core material parameters
- Measuring the borehole water losses in the bedrock and clay core
- Trial grouting of clay core and bedrock
- Continuous grout curtain seepage control.

A study was prepared for the site seismological and seismotectonic characteristics determination. The study contains the basic design seismic parameters: maximum earthquake intensity and maximum acceleration.

A hydrological study has been prepared for determination of the Cetina river floods at the Peruća dam.

Contrary to the original design preparation, when the quality recorded hydrological data were insufficient, it was now possible to offer a quality hydrological analysis because the quality recorded data collected since the original dam construction was completed were available.

The above investigations are covered by special papers, so they shall not be detailed here.

In addition to the above, the investigations have been conducted on potential clay deposits which have also been described in a special paper. The rock deposits were not specially

investigated since the rock was taken from the same quarry as for the original dam, located in the right reservoir end, some 200 m upstream the dam.

A number of investigations were also conducted related to solving of some minor problems.

DAM STABILITY ANALYSIS

The dam stability analysis included:

- dam static stability analysis,
- dam seismic stability analysis.

Dam Static Stability Analysis

The dam static stability analysis was conducted for the dam construction completion and dam exploitation stages. The analysis for the direct dam construction stage used the strength parameters in total stresses from the combined failure mechanism. The analysis for the dam exploitation stage used the strength parameters in effective stresses for three water levels:

- water level at El. 362.20 m a.s.l. - flood water
- water level at El. 343.00 m a.s.l. - reservoir gravity centre
- water level at El. 361.50 m a.s.l. - maximum operating

Dam stability calculation under the static conditions was performed using the SSTAB1 program (Wright, 1974) based on the Spencer slope stability analysis by the limit equilibrium method (Spencer, 1967).

The stability calculation assumed the below strength parameters for various materials:

(a) clay

- unit weight $\gamma = 20 \text{ kN/m}^3$
- strength parameters in effective stresses acc. to Bowles (1982) and Goodwin (1978) for liquid limit 60% and plastic index 40%:
 - cohesion $c' = 5 \text{ kN/m}^2$, angle of friction $\varphi' = 20^\circ$
- strength parameters in total stresses (undrained strength)
 - cohesion $c_u = 40 - 60 \text{ kN/m}^2$, angle of friction $\varphi_u = 0^\circ$

(b) rockfill

- unit weight $\gamma = 21 \text{ kN/m}^3$
- strength parameters in effective and total stresses:
 - cohesion $c' = 0 \text{ kN/m}^2$
 - angle of friction $\varphi' = 45^\circ$, for normal stresses 0-200 kN/m^2
 - angle of friction $\varphi' = 41^\circ$, for normal stresses above 200 kN/m^2

(c) filters

- unit weight $\gamma = 20 \text{ kN/m}^3$
- strength parameters in effective and total stresses:
 - cohesion $c' = 0 \text{ kN/m}^2$
 - angle of friction $\varphi' = 35^\circ$

d) clayey rock in dam crest

- unit weight $\gamma = 20 \text{ kN/m}^3$
- strength parameters:
 - cohesion $c' = 0 \text{ kN/m}^2$
 - angle of friction $\varphi' = 38^\circ$

The safety factors obtained by calculations for all sliding surfaces exceed the required minimum. The safety factors for individual cases are shown in Tables 1 and 2, and the potential sliding surfaces are given in Fig. 1.

	1	2	3	4	5	6	7	8
Construction completion								
Case I	1.52	1.62	1.60	1.97	2.30	1.56	1.83	2.12
Case II	1.52	1.62	1.60	1.99	3.32	1.55	1.85	2.14
Exploitation stage								
Water level I	1.52	1.62	1.61	1.83	-	1.54	1.92	2.31
Water level II	1.52	1.73	2.33	3.56	4.62	-	3.49	4.25
Water level III	-	-	-	-	16.75	16.27	12.98	13.45

Table 1 Safety factors - upstream sliding surfaces

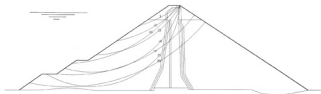


Fig. 1 Upstream and downstream sliding surfaces

	1	2	3	4	5	6
Construction completion						
Case I	1.72	1.67	1.90	1.60	1.77	1.91
Case II	1.72	1.67	1.91	1.59	1.92	1.92
Exploitation stage						
Water level I	1.72	1.67	1.90	1.59	1.81	1.96
Water level II	1.72	1.67	1.90	1.59	1.81	1.97
Water level III	1.71	1.64	1.89	1.57	1.77	1.96

Table 2 Safety factors - downstream sliding surfaces

Case I Depth 0.00 - 5.00 m - strenght parameters of drained clay
 Depth over 5.00 m - strenght parameters of undrained clay

Case II Depth 0.00 - 7.50 m - strenght parameters of drained clay
 Depth over 7.50 m - strenght parameters of undrained clay

Water level I Water level El. 362.20 m a.s.l.

Water level II Water level El. 343.00 m a.s.l.

Water level III Water level El. 361.50 m a.s.l.

Dam Seismic Stability Analysis

The Newmark sliding block method was used for the seismic stability analysis. The seismic analysis included:

- Comparative dam seismic response analysis by the Ambraseys and Sarma procedure (1967), applying the SHAKE program (Schnabel, Lysmer, Seed 1972);
- Calculation of yielding horizontal accelerations causing the unit safety factors on the potential sliding surfaces in the dam body, applying the modified SSTAB1 program.
- Calculation of permanent sliding body displacements applying the Newmark sliding block method (Newmark 1965, Seed 1979), based on the results of the seismic response analysis and calculation of the yielding horizontal accelerations as the supporting data for evaluation of the permanent dam deformations.

The seismic load on the Peruća site was determined by the "Andrija Mohorovičić" Geophysical Department of the University of Zagreb, 1994.

The maximum expected bedrock accelerations were obtained by the deterministic procedure for the three types of earthquake:

- (a) earthquake on the dam site, magnitude $M=5.7$, maximum acceleration $a_{\max}=0.160$ g, maximum intensity $I_{\max}=8.0^\circ$ MCS
- (b) close earthquake, epicentral distance - 7 km, magnitude $M=6.50$, maximum acceleration $a_{\max}=0.286$ g, maximum intensity $I_{\max}=8.6^\circ$ MCS
- (c) remote earthquake, epicentral distance - 21 km, magnitude $M=6.6$, maximum acceleration $a_{\max}=0.197$ g, maximum intensity $I_{\max}=7.80^\circ$ MCS.

The maximum seismic acceleration $a_{\max}=0.22$ g for the return period $T=1,000$ years was obtained by the probabilistic method, earthquake, S2, while the maximum acceleration for the operating basis earthquake, S1, is $a_{\max}=0.17$ g.

Many regulations request that the seismic stability of an structure be inspected for operating basis earthquake. For the rockfill dam with the satisfactory level of stability for the maximum probable earthquake, it is not common to conduct special operating basis earthquake analyses. Therefore the maximum value $a_{\max}=0.29$ g was used for further analyses, defined by the deterministic procedure for close earthquake.

The analyses results have indicated that the static safety factors are above 1.5, even to 2.53 for the upstream sliding surfaces, and 2.05 for the downstream sliding surfaces. The exception is a very shallow downstream sliding surface where the static safety factor is 1.47.

The yielding accelerations with the safety factor 1.0 range between 0.1 g and 0.28 g for the upstream sliding surfaces, and 0.22 g and 0.63 g for the downstream sliding surfaces.

The permanent displacements along the sliding surfaces range between 0 and 46 cm on the downstream dam end, and 2 and 108 cm on the upstream dam end, Fig. 2.

On the basis of the analysis results the below conclusions on the dam seismic stability were reached:

- The dam body damage is possible under the effect of the maximum probable earthquake on the site with maximum acceleration $a_{\max}=0.29$ g due to permanent displacements of potential sliding masses.

SEIZMIC ANALYSIS UPSTREAM AND DOWNSTREAM END

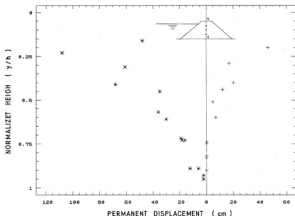


Fig. 2 Permanent displacements along the sliding surfaces

- It is estimated that the global dam stability shall not be impaired due to the dam overtopping because the maximum probable permanent dam crest settlement is about 60 cm and there is an ample freeboard between the maximum operating level of the reservoir and the crest; any possible dam crest damage is not considered significant for the global stability;
- The potential displacements of the sliding masses in the upper half of the dam may cause local damage in the slope and cracks in the dam body, but it is estimated that the planned construction of filters on the both core flanks, and the impermeable cut-off wall in the central section, shall sufficiently prevent any possible outbursts and seepage through the core.
- The seismic instrumentation and the dam behavior monitoring according to the regulations are of particular importance. Would the earthquake with acceleration over $a_{max}=0.2$ g occur on the bedrock, the reservoir level should be lowered or the water discharged; subsequently, the dam should be inspected, particularly the sensitive upstream slope.

FLOOD DISCHARGE

For sizing of the remediated Peruća Dam spillway, the relevant flood wave was the one with the 10,000-years recurrence period.

The flood analysis for this design documentation has been prepared for various recurrence periods. The analysis results are presented in Table 3. All the results from the Table are expressed in rounded numbers, so the actual value of the 10,000-year recurrence period flood used in the analysis is 509.60 m³/sec.

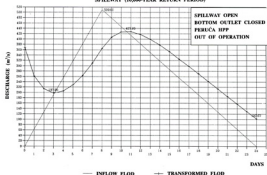
Recurrence period (yrs)	Q _{max} (m ³ /s)	
	WET PERIOD	DRY PERIOD
	Oct - May	Jun - Sept
2	192	67.0
5	232	98.6
10	259	120
20	285	140
25	294	146
50	319	166
100	344	185
1000	427	249
10000	510	314

Table 3 The maximum Cetina annual discharges with different recurrence periods for wet and dry period of the year

As required for determination of the relevant spillway discharge and its sizing, the 10,000-year flood transformation simulation was performed in situation when it enters the reservoir in which the maximum operating level is on El. 361.50 m a.s.l. The maximum operating level was determined for the new dam crest elevation which is by 1.5 m higher than the earlier (to be detailed in the next section).

The analysis has shown that the 10,000-year flood can completely discharge over the spillway. The water level in the reservoir in that case raises from the El. 361.50 m a.s.l. to the El. 362.20 m a.s.l., with maximum discharge of 427.93 m³/sec to which the chute had been sized. This high discharge lasts for approximately one day. It should be underlined that the maximum flood inflow discharge of 509.60 m³/sec transforms into 427.93 m³/sec at outflow.

FLOOD WAVE TRANSFORMATION OVER THE
SPILLWAY (10,000-YEAR RETURN PERIOD)



TRANSFORMED FLOOD WAVE LEVEL IN THE
RESERVOIR (10,000-YEAR RETURN PERIOD)

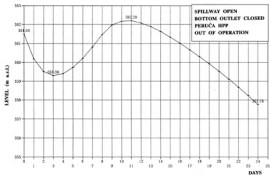


Fig. 3 Discharge of the 10 000-year flood wave through the reservoir

The assumption that only the spillway participates in flood discharge is on the safe side, and it assumes the low probability of the bottom outlet gate clogging, impaired water discharge through turbines and occurrence of an extremely high flood wave, all together.

The Fig. 3 shows the transformation analysis results of the 10,000-year flood over the spillway and the corresponding levels in the reservoir.

DAM CREST ELEVATION DETERMINATION

The dam crest was on the El. 363.00 m a.s.l., the upstream slope ratio 1:1.45, the downstream slope ratio 1:1.5, the operating reservoir level on the El. 360.00 m a.s.l., and the maximum reservoir level on the El. 361.2 m a.s.l., Fig. 4.

The activities related to the development of the damaged Peruća Dam remediation design documentation included analyses and calculations of dam stability under the seismic loads and discharge of 10,000-year floods through the existing discharge structures which were not included in the original design documentation.

The original design documentation did not include the dam stability analysis of the seismic forces effect because it was not a common practice at the time, and the calculation possibilities were limited because the modern electronic computers did not exist.

On the other hand, the present criteria for determination of the dam crest elevation are different compared to the ones applicable at the time.

The three crucial effects were 10,000-year flood wave discharge, dam stability under the effect of seismic forces, and height of a wave set-up on the dam slope due to wind.

The hydrological analysis of the time series of natural daily inflow discharges of the Peruća reservoir for the period 1961-1986 defined the hydrograph of the 10,000-year flood wave with $Q_{\max}=510 \text{ m}^3/\text{sec}$ and $T=24$ days as the basic characteristics. Simulation of its discharge through the reservoir had shown that the reservoir level rouse from the maximum operating level of 360.00 m a.s.l. to the El. 362.17 m a.s.l., while the discharge over the spillway is $424.93 \text{ m}^3/\text{sec}$. As compared with the design status, the increase in water level makes 0.97 m, and the discharge increased by $84.93 \text{ m}^3/\text{sec}$.

The seismological and neotectonic investigations defined the maximum earthquake acceleration on the Peruća site as $a_{\max}=0.29 \text{ g}$. This value was used to check the dam stability, and the dam crest settlement caused by the seismic forces is about 0.6 m.

THE ORIGINAL DAM DATA

- MAX. WATER LEVEL	361.20 m a.s.l.
- MAX. OPERATING LEVEL	360.00 m a.s.l.
- RESERVOIR VOLUME AT EL. 360.00 m a.s.l.	541 mil m ³
- DAM VOLUME	874,087 m ³
- MAXIMUM HEIGHT	63 m

KEY

1. ROCK FILL
2. CLAY CORE
3. CRUSHED CLAYEY ROCK
4. FILTER LAYERS

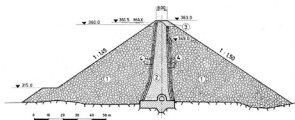


Fig. 4. Typical cross section of the original dam

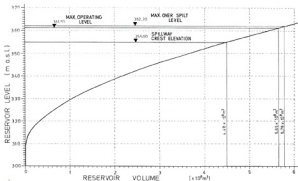


Fig. 5. The Peruca reservoir volume curve

The analysis of the wind speed, direction and frequency has shown that the highest waves are generated by the NW-SE wind at the speed of 135 km/h, and wind blow radius of 1.43 km. The wave height is 1.40 m, length 19 m, and the wave set-up 0.80 cm.

According to the applicable criteria, the dam crests in the seismically active zones should be designed on elevation 2.0 - 2.5 m above the flood wave level. Therefore, the Peruća Dam crest should be between the El. 364.18 m a.s.l. and 364.67 m a.s.l., which is the heightening of the design and constructed elevation of the original dam crest at 363.00 m a.s.l. by 1.17 to 1.67 m.

According to another criterion, the dam crest elevation is defined so that the 10,000-year flood wave level is added the height of wave set-up caused by the wind and freeboard of min. 1 m, which for the Peruća Dam is 363.97 m a.s.l.

Therefore, the seismic criterion according to which the dam crest should be between the El. 364.17 m a.s.l. and 364.67 m a.s.l. is relevant for the Peruća Dam. The dam settlement of 50 cm after completion of works is estimated, so that the crest elevation of 365.00 m a.s.l., i.e. 2 m above the dam crest from the original design, with gradual descend to 364.50 m a.s.l. towards the flanks was preliminarily accepted.

Since the seismic criterion requested heightening of the dam crest for min. 1.5 m, the possibility of raising the operating level of the Peruća reservoir has also been considered. This would give additional water quantities. Simulation of discharge of 10,000-year flood when it encounters the reservoir level of 361.50 m a.s.l. has shown that the level rose to the El. 362.20 m a.s.l., Fig. 3. With respect to the maximum water level of 362.17 m a.s.l., in situation when the 10,000-year flood encounters the maximum operating level at El. 360.00 m a.s.l., this is the level increase of only 3 cm.

Since the level of 362.20 m a.s.l. meets the criteria that the dam crest should be on El. 364.50 m a.s.l., it has been decided to raise the operating water level in the reservoir by 1.5 m, i.e. from the earlier 360.00 m a.s.l. to 361.50 m a.s.l. This would provide an additional $24 \times 10^6 \text{ m}^3$ of water, i.e. an increase of approximately 4%, Fig. 5.

SELECTING THE DAM HEIGHTENING METHOD

Since the analysis results indicated that the dam crest has to be heightened by 1.5 - 2.0 m, the heightening method was discussed.

Two possibilities were considered:

- heightening by backfilling on the upstream slope of the existing dam,
- heightening by backfilling on the downstream slope of the existing dam.

Dam Heightening by Backfilling on the Upstream Slope

The backfilling on the upstream slope requests the horizontal width of 7.5 m. This includes 6 m required for geometrical reasons provided the dam crest width remains 8 m. This is also the width providing for normal work and traffic during performance of works, while 1.5 m is required for the rip-rap.

The backfilling is carried out by rolling in approx. 50-100 cm thick layers, followed by placement of rip-rap. The reservoir water is to be emptied to the bottom, under the than allowable EL 330.00 m a.s.l., namely the minimum operating level of the Peruća HPP of 325.00 m a.s.l. The said elevations are achieved only after the backfilling is completed on additional 5 m, which with respect to duration of works for the EL 335.00 m a.s.l. means approximately 2.5 months. During this period the power plant would be out of operation.

Special problem is filling of joints between individual blocks of the existing rip-rap by pressing the rock material under the water jet pressure.

Total of approximately 130,000 m³ of material is used for backfilling.

Dam Heightening by Backfilling on the Downstream Slope

The backfilling on the downstream slope requests the width of 6 m, which is both geometrical and technological minimum.

The basic problem related to the backfilling on the downstream slope is vicinity of the Peruća HPP switchyard, i.e. the fact that the dam toe penetrates the switchyard area. For that reason, the construction of a 5.0 m high retaining wall is planned which would minimally enter the switchyard.

Such a solution requests only relocation of three electrical cabinets and reconstruction of a concrete pillar, i.e. its base would be founded in the retaining wall. The works also includes the retaining wall construction around the bottom outlet auxiliary gate house, and extension of the access tunnels leading to the grouting gallery for 6 m.

In this case, throughout performance of the activities the reservoir level can be maintained on the EL 330.00 m a.s.l. With good organization, the work on the retaining wall in the switchyard would last for a month. The power plant shutdown is necessary only while the part of the wall directly entering the switchyard area is constructed, for not more than two weeks.

KEY

1. DAM RECONSTRUCTION (5-10 m)
2. CRUSHED CLAYEY ROCK
3. PLASTIC CONCRETE CUT-OFF WALL
4. GROUTING
5. EXISTING GROUT CURTAIN
6. ROCKFILL
7. CLAY CORE
8. FILTER LAYERS
9. RIP-RAP
10. RETAINING WALL

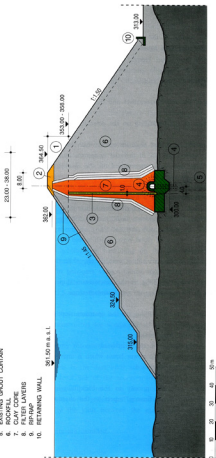


Fig. 6 Remediated Peruća Dam - typical cross-section

The backfilling on the downstream slope requests about 80,000 m³ of material, the retaining wall volume in the switchyard includes approximately 700 m³ of concrete, and around the auxiliary gate house 350 m³ of concrete, Figs. 6 and 9.

Selected Solution

The analysis of the above alternatives and the cost estimate lead to the below conclusions.

The advantages of the backfilling on the downstream slope are:

- (a) The downstream dam shell increases, which favorably affects the dam stability;
- (b) The required quantity of material is approx. by 55-60% lower;
- (c) The large rocks are not required for the rip-rap;
- (d) There is no need for filling in the cavities between the rocks in the downstream shell;
- (e) There is no need for a forced emptying of the reservoir, which is very important with respect to the Cetina river regulating;
- (f) The Peruća HPP shutdown would last for 15 days instead of 2.5 months requested for the alternative with the backfilling on the upstream slope; this is important from the aspect of flood wave which would have to be discharged through the power plant;
- (g) The works could start almost immediately, while the alternative with the backfilling on the upstream slope would require emptying of the reservoir;
- (h) The costs of backfilling on the downstream slope are about 34% lower than the costs of the backfilling on the upstream slope.

The advantages of the backfilling on the upstream slope are:

- (a) Simpler work organization
- (b) No interferences with the switchyard
- (c) No traffic through the power house which is unavoidable in case the protective wall is built on the switchyard.

The comparison gave the priority to the backfilling on the downstream slope from structural, technological and financial aspect, so this alternative was accepted for the Peruća Dam heightening.

SOLUTION OF THE REMEDIATION

The final design for the Peruća Dam remediation was based on the accepted design in June 1993. The final design did not considerably depart from that design, and the differences were an outcome of the new information obtained through hydrologic, meteorological, seismic investigations studies and calculations, and need to respect the actual field conditions, Figs. 7 and 8.

Therefore, the final design documentation for the Peruća Dam remediation included the below activities:

- removal and reconstruction of the heavily damaged left and right dam ends and 5-10 m high section of the dam crest,
- the plastic concrete cut-off wall is built-in into the central dam part to take over the role of an impermeable element instead of the damaged clay core,
- the cut-off wall construction is preceded by grouting the clay core along the cut-off wall route in order to protect the wall trench,
- the grouting gallery is reconstructed in full length,
- the dam crest is heightened to the design elevation of 364.50 m a.s.l., with construction elevation in the central part 365.00 m a.s.l. and in the ends 364.50 m a.s.l.,
- the dam heightening is achieved by backfilling on the downstream end, and a switchyard and auxiliary gate house retaining wall is constructed in the dam toe. Figs. 9 and 10,
- the spillway, including the gate house, is completely removed and built anew; the flap is increased by 1.5 m, and the spillway flap span is 10 x 7 m instead of 10 x 5 m;
the new hydromechanical equipment is installed;
- the chute is partially reconstructed for acceptance of spilled discharges in case of a 10,000-year flood wave;
- the Cetina riverbed is partly reconstructed in the point in which the chute enters the Cetina;
- the left and right platforms in the dam ends are heightened to the El. 364.50 m a.s.l., which elevates the level line of the access roads;
- the consolidation grouting is performed on the grout curtain and the contact grouting in the joint between the gallery and grout curtain;
to achieve impermeability the joint between the grout gallery and cut-off wall foundation is grouted;
- the monitoring instruments are installed.

The technical solutions shall be detailed below.

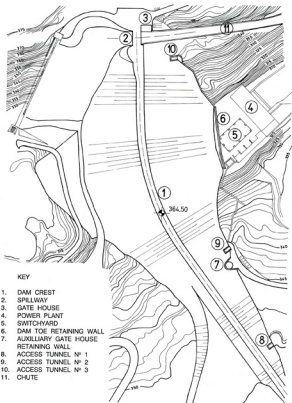


Fig. 7 Remediated Peruća Dam - layout

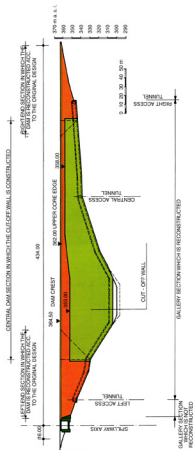




Photo 3. Auxiliary gate house retaining wall

GROUTING GALLERY REMEDIATION

The essential problem related to generation of a solution for the grouting gallery remediation was lack of precise data on the gallery condition. The gallery condition and the scale of its damage could not have been determined since the access tunnels Nos. 1 and 3 were demolished and completely blocked, while the access tunnel No. 2 was heavily damaged. However, from this point the gallery was accessible to a certain degree, which enabled inspection of some 50 m.

Therefore, a decision was brought to prepare a standard solution for the gallery remediation for three degrees of damage. This led to four types of gallery remediation solutions.

(a) TYPE-1

The assumed damage included cracking of gallery vault and walls, with no deformation, while the vault retained its static function.

The remediation for this degree of damage would include placement of 10 cm thick shotcrete lining reinforced with double Q-131 steel mesh.

The preparatory works would include cleaning of the existing gallery concrete covered with soot after the explosion, and sandblasting of surface to provide for a quality bond with the shotcrete, Fig. 10.

(b) TYPE-2

The assumed damage included major cracks and deformations in the gallery which had still retained its shape. The parts of the vault were ripped off. However, the vault retained its static function although there was a threat of its partial or complete caving-in.

The remediation for this degree of damage would include placement of 15 cm thick shotcrete lining with steel stilted arches YU-21 with flexible joint, and double Q-131 steel mesh reinforcement. The average axial distance of arches of 1.0 m is anticipated. The points in which the concrete broke off should first be filled in with the shotcrete, Fig. 10.

The preparatory works would include cleaning of the existing gallery concrete covered with soot and sandblasting of its surface.

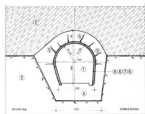
(c) TYPE-3

The assumed damage included complete collapse of gallery, vault and walls, while the foundation slab would be damaged and the new one should be placed.

The remediation is performed in two stages:

- First, the excavation is carried out under the protection of steel stilted arches YU-21 with flexible joint and steel plates. The plates are fixed above the arches. The axis-to-axis distance between the arches is approximately 65 cm, but it can be smaller if necessary.

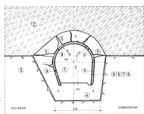
The excavation goes along the profile perimeter to the steel arch, after which the internal part of the profile is excavated.



TYPE 1

KEY

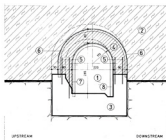
1. GROUTING GALLERY
2. EXISTING CLAY CORE
3. BEDROCK
4. EXISTING CONCRETE
5. CRACKS
6. STEEL MASH
7. STEEL ARCHES
8. SHOTCRETE



TYPE 2

KEY

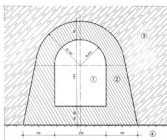
1. GROUTING GALLERY
2. EXISTING CLAY CORE
3. BEDROCK
4. EXISTING CONCRETE
5. CRACKS
6. STEEL MASH
7. STEEL ARCHES
8. SHOTCRETE
9. STEEL BARS



TYPE 3

KEY

1. GROUTING GALLERY
2. EXISTING CLAY CORE
3. EXISTING CONCRETE
4. NEW CONCRETE
5. ANCHORS
6. STEEL PLATES
7. FORMWORK
8. STEEL ARCHES



TYPE 4

KEY

1. GROUTING GALLERY OR ACCESS TUNNEL
2. NEW CONCRETE
3. NEW CLAY
4. BEDROCK

Fig. 10 Grouting gallery remediation - standard solutions

- The 80 cm thick gallery concrete lining is placed in a steel-wood formwork, and the pumped concrete is placed.

It is planned to bond the newly constructed part of the gallery with the existing foundation slab by anchors. The anchors are to be drilled into the slab. The gallery is to be constructed in 2.0 m excavation units, Fig. 10.

The gallery cross-section is reduced as compared to the original gallery in order to achieve the reduction in excavation works.

(d) TYPE-4

This type of remediation is applied on the gallery parts reconstructed and access tunnels in an open trench, after the dam ends have been removed. The 80 cm concrete lining is also placed in the steel-wood formwork using the pumped concrete. The gallery lining is not reinforced, except for the joints between the old and new concrete and in branches.

It is also planned that the gallery reconstruction would be followed by contact grouting behind the lining in order to fill in the remaining cracks caused by shifting of individual blocks which could not have been filled in during the remediation, Fig. 10. This is important because in this way the full contact between the gallery lining and the clay core is achieved, primarily in order to fix the plain concrete structure of the vault and provide its symmetric loading.

The design documentation also anticipated monitoring during the gallery remediation. The monitoring was aimed at the structure behavior observations, primarily regarding safety and structural stability. Since the gallery remediation actually presented the investigation work, it was focused on obtaining as many information about the core condition as possible.

The following monitoring, investigation and measurement activities were planed to be carried out during the remediation works:

- geotechnical monitoring,
- convergence measurements,
- geodetic surveying,
- taking of undisturbed dam core samples.

The geotechnical monitoring includes determination of the material properties in the front section of the caved-in gallery part trench, surveying of the construction procedures and daily records of quantities of the built-in support, namely timbering.

The convergence measurements and geodetic surveying were planned to be conducted in the profiles set in specified intervals, for each change in excavation conditions and in critical points such as fractured zones, etc.

The measurements to be conducted in each profile were:

- diagonal convergence,
- horizontal convergence,
- absolute arch shift (geodetic).

To determine the dam core condition, the undisturbed clay core samples were to be taken along the entire gallery.

CONSOLIDATION GROUTING ALONG THE CUT-OFF WALL ROUTE

The primary objective of the core grouting was filling in the cracks and cavities generated in core by the explosion in the dam, in order to prevent slurry losses and caving-in of the cut-off wall trench walls.

The consolidation grouting is related with the type of technology applied in construction of cut-off walls. Regardless of whether a grab or hydraulic trench cutter is used for excavation, the trench walls are secured with slurry, which is the standard technology applied in this type of works. The ever present problem with such technology is the possibility of slurry losses through a cavity in soil or presence of material of larger grain size, which might have the trench caving-in and trapping of machinery as consequence. However, such cases are rare in trenches excavated in the natural soil.

Still, we expected a situation in the damaged Peruća Dam core to be completely different. The assumption was that a number of finer and larger cracks opened in the dam core as a result of the explosion. Also, it was assumed that the considerable settlement, as the consequence of the gallery caving-in, generated the relief arches in the core, which leads to formation of caverns. This assumption was confirmed during the grouting gallery remediation works, when a considerably large cavern was encountered in the caved-in core.

All the above said indicated that there was a realistic risk of abrupt slurry loss during the cut-off wall trench excavation. This would cause the trench walls caving-in and trapping of excavation machinery, and consequently longer stoppage or works, the loss of machinery or extensive expenses for its salvage. Therefore, the measures were considered which should be undertaken to minimize the possibility of the abrupt slurry losses during the cut-off wall construction. One of these measures is consolidation grouting.

The decision on consolidation grouting of the core and partly of filter layers along the cut-off wall route was brought by the Peruća Dam Remediation Board of Experts on the meeting convened on December 7, 1993, after a comprehensive consideration of the issue.

The grouting was planned to be carried out in the central dam section which has not been removed, in a 256.30 m long stretch, between the stations 0+87.70 and 3+44.00.

The grouting was to be performed in the boundary zone along the route of the future cut-off wall trench, from the working platform between the El. 353.00 m a.s.l. and 358.00 m a.s.l. from which the cut-off wall construction was also to be carried out.

The primary row of boreholes was to be drilled first. When grouting of the primary boreholes was completed, the drilling of control boreholes for the grouting control were to be drilled. The control boreholes are made by core drilling. Would the control boreholes results lead to a conclusion that a row of secondary boreholes is necessary, they shall be drilled in the section where they are requested.

A triangular arrangement of the primary grouting boreholes was designed. The distance between the boreholes in direction of the cut-off wall axis and in the layout is 1.4 m, and 1.4 m along the height of the triangle which is vertical to the cut-off wall axis, Fig. 11. The control boreholes are drilled in the designed secondary boreholes locations, in triangular arrangement and located in the mid-distance between the primary boreholes.

Two grouting methods were planned for the clay core consolidation along the cut-off wall route, i.e.

- downward grouting in parallel with borehole drilling (SIBUP),
- grouting with gravity filling (gravity grouting).

Both selected methods meet the specific conditions of the medium in which the grouting is performed and the specified requirements.

The grouting medium is the dam clay core and partially the filters, and it is completely atypical for this type of works, which considerably affects the grouting technology. Thus, it was not simple to decide which grouting method would be the most suitable.

It was concluded that the decision on the most suitable method shall be brought after the trial grouting planned within the geotechnical and geophysical investigations was completed.

As regards the grout mix ratio to be used for filling in the cracks in clay, the use the slurry with higher percentage of bentonite was planned. For this purpose the slurry mix consisting of 80% cement and 20% bentonite was planned, mixed with water in ratio 1:3 to 1:0.7. Also, would the cavern be encountered during the grouting the slurry would be used with addition of sand. The slurry content in such case would be: 30% cement, 10% bentonite and 60% sand.

As regards the grouting pressure the actual risk of hydraulic fracture of material existed since the grouting was performed upstream the gallery in a comparatively thin clay core segment. Therefore the grouting pressure determined related to the effective vertical stresses in the clay core and varied between 100 kN/m² for the highest and 800 kN/m² for the deepest sections of the injected boreholes.

The consolidation grouting progress and results are presented in a special paper.

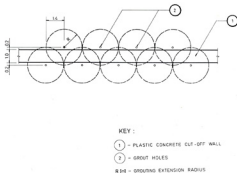


Fig. 11 Arrangement of boreholes for consolidation grouting along the cut-off wall trench route

CUT-OFF WALL

The cut-off wall is built-in into the dam body 4 m upstream the grouting gallery axis, in order to take over the impermeability function of the damaged clay core, Figs. 6 and 8.

In its joint with the left and right dam end the cut-off wall is cut into them for 2 m. It is also planned that the cut-off wall cuts for 2 m into the bedrock, namely the concrete plinth immediately below the central dam section.

The basic problem faced during the cut-off wall design was determination of the mechanical characteristics of the cut-off wall material and its depth. The calculation was conducted of the cut-off wall characteristics under various loadings. The calculation included the stress-strain analysis for different loadings. This subject is covered by a special paper, and this paper shall give a summary of the analysis related to the cut-off wall.

The calculation were based on application of the FEM method, respecting the elastoplastic behavior of the material and nonlinear relation of stress and stiffness, namely strength and effective stresses of the earth material in the dam. The FLAC computer program was used which enables solution of such complex geotechnical strain problems.

For calculation of the cut-off wall internal forces, the following dam body loading stages were simulated:

- 6 phases of backfilling during the dam construction in 10 m high layers
- removal of the top layer, dam reconstruction
- cut-off wall construction, backfilling the last layer and dam crest heightening with additional filling of the downstream slope
- seismic loading, set as the static force equivalent to 15% of the total weight.

The cut-off wall material characteristics were varied in the calculation in order to come to optimum characteristics.

- For all alternatives of dam material stiffness and cut-off wall material stiffness and strength, the plastic hinge occurs in the cut-off wall bottom, i.e. it reaches the value of its bending resistance. Actually, the dam strains cause the undesired cut-off wall displacements which cause occurrence of the plastic hinge in the supporting point on the dam foundation level. In all other points the bending moments in the cut-off wall are smaller than the cross-section bearing capacity regardless of the assumptions on stiffness and strength parameters.

- The transverse crack in the cut-off wall plastic hinge, caused by lack of cut-off wall material tensile strength, is 80 cm long from the upstream to the downstream cut-off wall edge. In the most extreme case, the crack opening is less than 5 mm.
- The parameter analyses have shown that possible softening of the cut-off wall material, and thus reduction in its strength (where the analyzed ratios of the Young's modulus and strength are from 1000:1 to 300:1) does not eliminate plastic hinge in its bottom. Since there is certain possibility of cut-off wall material erosivity in the crack location, the cut-off wall material should have satisfactory strength, and it is recommended that the cut-off wall material be designed so that its Young's modulus does not exceed 400 MN/m², and the compressive strength after 28 days is not below 1 MN/m².

The calculation has shown that the cut-off wall characteristics should be:

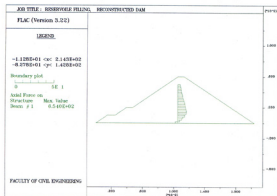
- cut-off wall compressive strength must be over 1.0 MN/m²
- Young's modulus of elasticity shall not exceed 400 MN/m²
- Darcy permeability coefficient shall not be above 10⁻⁸ m/sec.

It has also been determined that the cut-off wall thickness shall be 1.0 m.

It was decided that the cut-off wall shall be constructed of plastic concrete with the above characteristics.

The design documentation includes a comprehensive quality control program for the period before and during the cut-off wall construction, and the testing program for the completed cut-off wall.

Additional longitudinal cut-off wall forces [kN/m¹]



Cut-off wall bending moments [kNm/m¹]

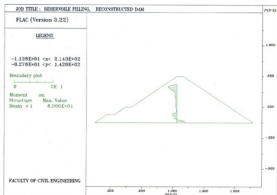
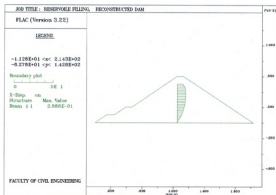


Fig. 12 Cut-off wall calculation results

Horizontal (x) cut-off wall displacement [m]



Vertical (y) cut-off wall wall displacement [m]

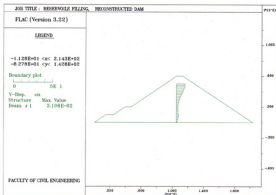


Fig. 13 Graphic presentation of the cut-off wall calculation results

The dam body consists of:

- clay core
- upstream and downstream filter layers
- upstream and downstream rock shells
- dam crest of clayey rock material.

Because of the downstream widening, the dam axis is placed asymmetrically to the roadway axis and actually respects the axis of the original dam.

Maximum dam width in the bottom, in the deepest part of the left end, is approx. 12.5 m and in the right end approx. 9.0 m. In its top section the dam tilts towards the top, which is the result of the dam heightening, and need for adequate connection with the cut-off wall top, Fig. 6.

Clay core is protected from the upstream and downstream end by 2.0 m wide filter layers. In the dam top section, in the low pressure zone, the upstream filter is narrowed to 1.5 m.

The upstream and downstream shell are planned to be made of rock material. On the upstream slope, the 1.5 m wide belt of larger size rock material is planned, including the rip-rap.

The dam crest shall have a built-in clayey rock layer, Fig. 14.

Widening of the downstream shell to 6.0 m shall be achieved with rock material and lateral step-shaped cutting in into the existing dam body.

Material Characteristics

The design characteristics of the dam body material are as follows:

(a) Clay

The core construction material is clay of intermediate to high plasticity, CUCL, taken from the spoil area on which it was disposed when earlier excavated from core or brought in from the deposit.

KEY

1. CLAY CORE
2. FILTER LEYERS
3. CRUSHED CLAYEY ROCK
4. PLASTIC CONCRETE CUT-OFF
5. RIP-RAP
6. ROCK FILL

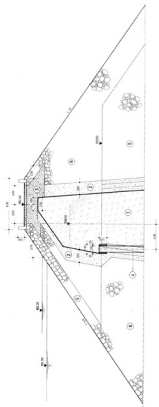


Fig. 14 Dam crest

The clay characteristics shall be:

- | | |
|--------------------------------|---|
| - grain-size composition | $35\% < 0.002 \text{ mm}$ $75\% < 0.06 \text{ mm}$ |
| - liquid limit | $43\% < W_L < 60\%$ |
| - index of plasticity | $20\% < IP < 38\%$ |
| - compaction, modified Proctor | $W_{opt} = 17-24 \text{ (\%)} \quad \gamma_d = 17.5-15.5 \text{ (kN/m}^3\text{)}$ |
| - compaction of built-in clay | $\gamma_{d \text{ min}} = 0.95 \gamma_d \text{ po modified Proctor}$ |

(b) Filter Material

The filter material is rock material of natural origin, well graded gravel and sand, or broken stone of adequate natural origin, or blasted rock. Use of the existing filter material excavated earlier from the dam body has been planned, if possible; the material from natural deposits, quarry or crushing plants was to be used.

The material for the filters had to be hard and sound broken stone with specific gravity of minimum 25 kN/m^3 .

The selected filter material should meet the Sherard criterion - the diameter of 15% filter grain $DF_{15} = 0.7-1.5 \text{ mm}$. The grain-size curves of the dam filter layer material are given in Fig. 15.

The unit weight of the built-in dam filter layer material should be 0.7 of the maximum unit weight obtained in a laboratory.

(c) Upstream and Downstream Shell and Upstream Slope Rip-Rap Material

The upstream and downstream shell and the upstream slope rip-rap material is rock of adequate composition and quality, either of natural origin or the limestone rock, or all types of eruptive rock. It has been planned to use the rock from the quarry which had been exploited during construction of the original dam.

The grain-size composition is specified for two types of rock material, the material used for the upstream and downstream shell designated with, " K_1 ", and the one for the upstream slope rip-rap, " K_2 ", Fig. 16.

The largest fraction for the K_1 material is specified not to exceed 500 mm. The rock material with more fine grains may be built-in into the shell inside, while the grain of $d < 2 \text{ mm}$ shall not participate with more than 40%, and the grain of $d < 0.006$ with not more than 5%.

The unit weight of the built-in material for the upstream and downstream shell must be 20 kN/m^3 .

It has been planned to build 1.5 m thick and 2.7 m wide upstream slope rip-rap of the material including the large rocks namely stone blocks and coarse gravel, K_2 . As a rule, the largest grain was not to exceed 1.0 m.

The grain-size curves for the rock materials K_1 and K_2 are shown in Fig. 16.

(d) Dam Crest Clayey Gravel and Rock Material

It has been planned to build in the clayey gravel and rock material in the very dam crest. This was the solution used for the original dam, and selected for protection of the clay core against drying out.

The unit weight of the material should be min. 18 kN/m^3 .

The grain-size curve is shown in Fig. 15.

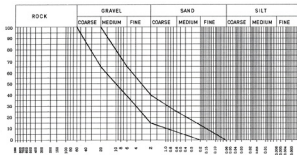
Placement of Material

Contrary to the rock material placement by hydromonitors into the original dam, the reconstructed dam material was placed by rolling and compacting in layers. All other materials have also been planned for building in by compacting in layers. The design minimum thicknesses of the compacted layers are:

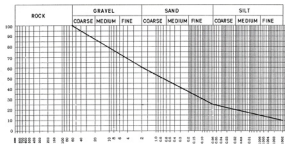
Mixed rock material	60 cm, compacted
Filter	to 50 cm, compacted
Clay	25 cm, compacted
Rip-rap	to 1.0 m

It has also been planned to determine the optimum thicknesses by trial compacting.

The design documentation also includes a comprehensive material quality control program applied prior to and after the material placement. The sampling procedure is specified, both the in-situ sampling and laboratory testing of samples. It has also been specified that the on-site soilmechanics laboratory should be set up for testing of the material samples.



Filter layers



Clayey material

Fig. 15 Grain-size curves

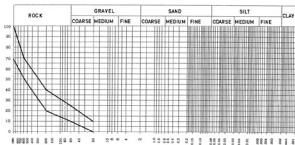
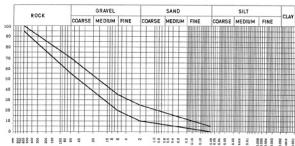


Fig. 16 Grain-size curves for the shell rock material

SPILLWAY STRUCTURE AND CHUTE

General

The original spillway, including the gate house, was heavily damaged and, after a thorough inspection, it was decided that it should be completely demolished and rebuilt.

The new spillway was sized for the 427.93 m³/sec discharge, which is the maximum discharge of the transformed 10,000-year flood wave at the maximum reservoir level at El. 362.20 m a.s.l. The discharge requires a span of 10 x 7.2 m.

The spillway crest remained as in the original dam on El. 355.00 m a.s.l. Since the maximum operation level is planned to be on the El. 361.50 m a.s.l., it was specified that the spillway flap top edge shall be 0.5 m above the operating level, on El. 362.0 m a.s.l., namely that the flap should close the area of 10 x 7 m.

Hydromechanical Equipment

When the spillway gate was selected, the intention was to retain the same gate type, namely to use the flap. In addition to being the most suitable for this type of structures by its characteristics, the possible installation of another type of gate would require larger spillway structure, which we wanted to avoid.

On the other hand, there are not many structures in the world with a 10 x 7 m flap. The basic problem with a flap of this size is to ensure its stiffness and reduce the deformations at full load to the acceptable minimum. The flap closing mode is crucial.

Therefore, three modes of flap closing have been analyzed, i.e. by hydraulic cylinder installed in one end of the flap, by two hydraulic cylinders installed in both flap ends, and one cylinder installed below the flap.

It was decided that the best solution is the one with the cylinder placed in the shaft below the flap, Figs. 17 and 18.

The flap lies on three bearings in the spillway crest concrete. Under the maximum loading, when the flap is on its top level, the central and side bearings suffer the tensile force of 2,980 kN, 1,600 kN, respectively.

These forces are taken over by 6 Φ 70 mm anchors embedded in concrete under the central bearing, and 6 Φ 50 mm anchors under the side bearings. To prevent concentrated loads within a narrow concrete strip around the anchors and their transmitting to the entire spillway structure, the spillway crest is fitted with the reinforced concrete beam fixed in the lateral walls.

The hydraulic cylinder is mounted in 8 m deep shaft, Fig 18. The cylinder takes over the 4,030 kN force when the flap is opened at maximum loading.

To prevent freezing of the lateral rubber flap seal, the design anticipated heat tracing of the lateral shields. Each shield has two heat tracing systems. One system is for that part of the shield included in the first step of flap closing to El. 359.50 m a.s.l. The other system heats the rest of the shield and is actuated only when the flap is already lowered to the El. 359.50 m a.s.l. and its further opening is anticipated.

Flap Control

The flap control is generally automatic, but there is also a possibility of manual control. The flap opening modes and requirements are determined by the design documentation, and they are based on the 10,000-year flood wave discharge over the spillway. The flap opening requirements are:

- the reservoir level should be maintained at the El. 361.50 m a.s.l.
- would the level exceed the El. 361.52 m a.s.l., the flap opening starts
- the flap opening is carried out in specified steps
- the first opening step is lowering of the top flap edge by 2.5 m, from the El. 362.00 m a.s.l. to the El. 359.50 m a.s.l.
- in case the water level continues to increase above 361.20 m a.s.l., the flap should be open in the below steps, Table 4.

OPENING STEPS	FLAP OPENING (cm)	OVERTOPPING HEIGHT (m)
1	250	2.0
2	60	2.6
3	60	3.2
4	80	4.0
5	80	4.8
6	60	5.4
7	60	6.0
8	flap fully open	6.5

Table 4 Spillway flap opening steps

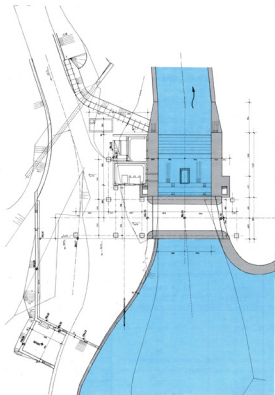


Fig. 17 Spillway structure - layout

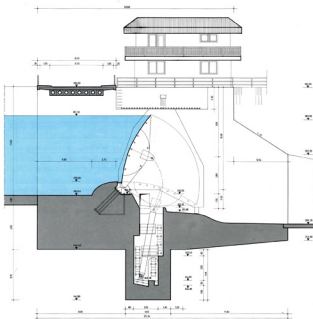


Fig. 18 Spillway structure - longitudinal section

The flap opening calculation was done for the 10,000-year flood where $Q_{max} = 509.60 \text{ m}^3/\text{s}$, wave set-up time 4 hours, which results in an hourly discharge increase of $5.31 \text{ m}^3/\text{s}$. The assumed basic Cetina river discharge equals the mean discharge of $42 \text{ m}^3/\text{s}$.

Provided the reservoir level is maintained on the El. 361.50 m a.s.l., and the allowable increase is +2 cm, the below results have been obtained, Table 5.

Level	Flap opening	Overtopping height	Overtopping discharge	Overtopping duration
(m a.s.l.)	(cm)	(m)	(m ³ /s)	(hrs)
361.52	200+50	2.02	63.59	7.0
361.60-361.52	60	2.61	93.40	5.0
361.50-361.52	60	3.21	127.39	7.0
361.50-361.52	80	4.01	177.86	12.0
361.50-361.52	80	4.81	233.66	9.0
361.50-361.52	60	5.41	278.72	8.0
361.50-361.52	60	6.01	326.35	7.0
361.50-361.52	100	6.51	367.91	55.0

Table 5 Discharge of the 10000-year flood wave over the spillway

The flap is completely open 55 hours after the flood wave inflow.

In case of the flap control system failure, the safety valve is actuated by a float in the control shaft when the reservoir level increases above 362.00 m a.s.l. In that case the flap is fully open.

Spillway Structure

The complete spillway structure, including the cylinder shaft, is founded on a hard rock. The spillway structure foundation level is 348.00 m a.s.l., and the cylinder shaft is on El. 342.85 m a.s.l.

The concrete works on the spillway structure are divided into two stages because of the hydromechanical equipment mounting. During the first stage, the reinforced concrete structure is built with the hydromechanical equipment mounting holes. During this stage, the anchor plates and anchors are embedded for fixing of the hydromechanical equipment. The second stage follows after the hydromechanical equipment has been mounted. The MB-30 concrete and RA 400/500 reinforcement is used.

The specified allowances for the concrete works are ± 1.0 cm and for the anchors and anchor plates ± 5 mm.

The spillway structure has side vent shafts for the spillway jet venting. One of these shafts is also used to access the hydraulic cylinder shaft.

The joint between the spillway structure and the clay core is usually the most sensitive point of any rockfill dam. The design has specified a concrete dent for realization of this joint in the right wall of the spillway structure.

Gate House

The gate house is located by the left wall of the spillway structure, in the left access platform. The ground floor has two rooms. One room houses the flap control equipment and electrical cabinet for power supply of the flap, headrace tunnel gate house, lighting of the dam and access roads. The other room houses a central instrumentation and dam monitoring equipment control system, and five accelerographs for seismic measurements at the dam. The upper floor are planned for accommodation of the gate house guard.

Chute

As already said, the maximum discharge over the spillway for the 10,000-year flood is 427.93 m³/sec. Since the existing chute was sized for the discharge of 340.00 m³/sec, which was the discharge capacity of the original spillway, the analysis was made using the discharge of 427.93 m³/sec. The analysis has shown that the determined water level in some chute sections exceeds the walls by max. 95 cm, so their heightening was required. In order to get the overflowed water jump in the spillway basin, the lateral walls were to be heightened on the left and right side by 1 m and 2.5 m, respectively, and the spillway sill by 1.5 m.

MONITORING SYSTEM

A complex system of measuring instruments is set up for the Peruća Dam monitoring.

The below types of monitoring are planned:

- visual observation
- geodetic survey of horizontal and vertical shift on the dam surface
- concrete wall tilting and twisting measurements
- clay core pore water pressure gaging
- total clay core pressures gaging
- measuring shifts of horizontal points in the dam body
- measuring shifts of vertical points in the dam body, settlement
- measuring piezometric levels in the downstream dam toe
- seepage measurements
- seismological monitoring
- reservoir water level measurements
- grouting gallery lining deformation, convergence, measurement
- dam crest expanding and shrinking measurement
- hydrometeorological measurements.

The measuring points and instruments grid consists of permanently installed devices which perform continuous measurements, and the measuring points and instruments for periodical measurements.

The measuring points in the dam body are grouped in 12 monitoring profiles characteristic for various reasons, on which various measurements are performed. This facilitates determination of the causes and consequences on various dam-related events on the profile.

In addition to the profiles, the measuring points are set in the retaining walls in the dam toe, and around the bottom outlet gate house, dome by the spillway, and the reservoir.

The permanently installed instruments are connected with the Monitoring Center from which the monitoring is controlled and the data collected.

The Monitoring Center is placed in a separate room in a gate house in the left dam end. It houses the computer controlled data acquisition system. The specialized programs available on the computer enable setting up of measurement time intervals and running of necessary analyses.

The Center also houses all five accelerographs which are interconnected with the sensors located in the gallery, dam crest and downstream bedrock. The seismological measurements are independent from the system controlling other measurements.

The measurements in other measuring points which are not interconnected with the central system are performed by the surveying instrumentation and other portable equipment, and present useful supplementary source of information and second check of the earlier specified measurements.

The monitoring system in the Peruća Dam is described in more detail in a special paper.

CONCLUSION

The Peruća Dam remediation project has been successfully brought to an end, regardless of numerous difficulties it was faced with.

The engagement on the project is valuable for its participants primarily because of the original technical solution applied on the project, but also because of the circumstances in which it was realized.

The Peruća Dam remediation project also attracted attention of the international professional community, and our experts have received acknowledgements on many occasions.

Regardless of the valuable professional experience and the acknowledgements awarded to us, we would like the Peruća Dam case to be the first and the last terrorist attack of this sort so no future projects similar to the Peruća Dam Remediation Project would be necessary.

SITE ORGANIZATION AND REMEDATION WORKS

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SPLIT

ABSTRACT

The Peruća Dam remediation is interesting from the civil engineering aspect, because of the specific technical and organizational issues which had to be solved. The specific characteristics of this project are:

- strict remediation schedule imposed by the electric power demands underlying the everyday life, work and development,
- lack of preconditions for a logical development of the project stages, i.e.
 - investigations
 - design
 - construction,
- establishment of organizational grid for various activities within the framework of a tight schedule and limited space,
- assurance of performance quality with respect to progress of works, quantity of built-in material and weather conditions,
- simultaneity of remediation works and reservoir exploitation,
- permanent threat of war operations.

All this requested full commitment and engagement of all the participants on the remediation project. The major decisions were made on a daily basis, and the cooperation between the design engineers, supervisors and all the contractors was very good, which lead to successful completion of project.

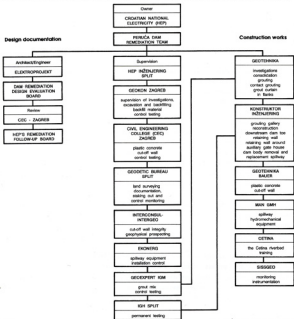
The scope of works carried out on the project is not large, but having in mind the complexity of the project, specific features of individual interconnected activities, continuous monitoring of their performance, solving of emergencies and abnormal work conditions because of the enemy operations, it may be concluded that the remediation was successfully completed within a short period of time.

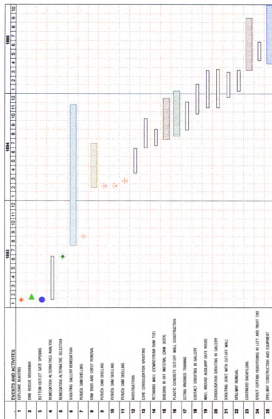
INTRODUCTION

In order to give a complete insight into the site organization during the Peruća Dam remediation we shall not focus only on the organization of individual activities but also on the topics which have been particularly important and had a specific effect on the site organization, i.e.

- chronology of events, from the explosive blasting to the completion of the remediation works,
- site organization with respect to the vicinity of the enemy artillery,
- exploitation of the reservoir during the remediation works.

PERUĆA DAM REMEDIATION PROJECT ORGANIZATION FLOW-DIAGRAM





THE PERUCA DAM REMEDIATION EVENTS AND ACTIVITIES

CHRONOLOGY OF EVENTS

The explosive, that the Serbian paramilitary forces and the former Yugoslav Army had set several month earlier in the Peruća Dam inspection gallery, was blasted at 10:48:55 am, on January 28, 1993. The intention was to destroy the dam, prevent its reconstruction and cause the catastrophe downstream the dam. Since at the time of explosion the reservoir level was on El. 356.20 m a.s.l., which is about 5 m below the level the enemy accounted for when preparing the destruction plans, the catastrophe was avoided but the dam suffered significant damage.

The dam was accessible at 8:00 am, on January 29, 1993. Then, the works started on closing the crater in the right dam end, backfilling the subsidence in the central part, and closing the crater in the left dam end. The backfilling material was brought from a 25 km remote quarry, and some of the dam material was used.

The reservoir level was 1.20 m above the spillway crest over which the water was flowing downstream, into the Cetina riverbed.

Since the downstream edge of the crater in the left dam end was only 30-50 cm above the reservoir level, the danger of water overtopping the crest and flowing down the downstream dam slope was present. It would cause erosion and weathering of the major part of the dam.

The blasting of the explosive damaged the grouting gallery and the clay core. Consequently, the water burst into the gallery. The water was flowing from the gallery through the drain pipe placed under the dam body which leads to the Peruća HPP switchyard and ends there in the form of a vertical shaft, Fig. 1.

The water running through this pipe carried chunks of clay, rock and concrete to 50 cm in diameter, Photo 1.

When the crater was closed at about 11:10 am, the water flow through the drainage pipe was stopped, only to restart from the access tunnel No. 2 in the right dam end.

The next step was opening of the bottom outlet gate at 14:15 pm, on January 29, 1993. The reservoir emptying started at discharge of approximately 220 m³/s. The gate was fully open, and the emptying lasted until March 12, 1993, when the reservoir level reached the El. 314.80 m a.s.l., which means that the reservoir was practically empty.

The remediation works run simultaneously with monitoring of the dam behavior after the explosion, i.e.

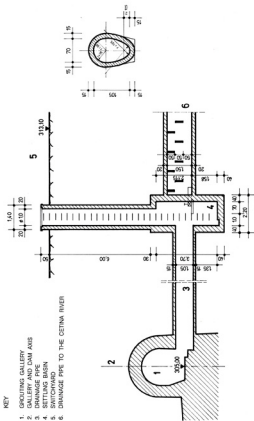


Fig. 1. Grouting gallery drainage



Photo 1. January 29, 1993 - Switchyard drainage shaft

- geophysical prospecting,
- water flow velocity measurements in the reservoir, on the upstream dam slope,
- measuring flow rate and turbidity of water from the access tunnel No. 2,
- dam settlement monitoring,
- cracks monitoring,
- visual inspection.

After the emergency measures to save the dam were undertaken, and the downstream region was protected, all further activities were oriented towards the soonest possible remediation of the dam since the Peruća reservoir significantly affects both the power generation of the downstream Peruća, Đale and Zakućac Hydroelectric Power Plants, and Hrvatačko polje and Sinjsko polje fields flood control.

The Peruća Dam remediation did not pose a major problem, because there is a number of experienced and qualified Croatian companies capable of contracting the hydroelectric power plants and other projects. They include Elektroprojekt Consulting Engineers Ltd., the architect/engineer of all the hydroelectric power plants in Croatia; Konstruktor Building Contractors Split, the contractors on a number of hydrotechnical projects, including the Peruća Dam; and Geoteknika Zagreb, a company specialized in foundations and grouting, which took part on a number of projects both in Croatia and abroad. The Peruća Dam remediation was contracted with the above companies.

The Owner, the Croatian National Electricity, has a large experience on construction supervision of the hydroelectric power plants, so that the dam remediation management and supervision organization posed no problem.

The only real problem was the vicinity of the enemy, who permanently threatened the lives of the people and machinery.

In April 1993, the Investigation Works Program was prepared for determination of the dam damage location and extent. The intention was to use investigation work results in preparation of the technical solution for the dam remediation. However, the permanent enemy activity prevented completion of the investigation works but for the geodetic surveying of the dam settlement and displacement.

Based on the visible damage and acquired data on the dam behavior after blasting, the alternative dam remediation solutions were analyzed and the most favorable selected on the basis of its reliability and cost-effectiveness.

The Board of Experts of the Civil and Hydraulic Engineering Department of the Elektroprojekt Consulting Engineers Ltd. accepted the dam remediation proposal consisting of the below activities on their meeting convened on June 14, 1993, Figs. 1 and 2:

- the grouting gallery reconstruction, which includes the investigation works for collecting of data on the clay core status in the maximum load area,
- construction of the impermeable plastic concrete cut-off wall in the central part,
- reconstruction of the left and right dam ends according to the original design documentation which specified the clay core,
- reconstruct the dam body in the central segment to the El. 353.00 m a.s.l., namely 358.00 m a.s.l., in order to remove the dam material damaged in explosion and create a sufficiently wide space for the plastic concrete cut-off wall construction,
- level the upstream slope until the design slide slope ratio is achieved,

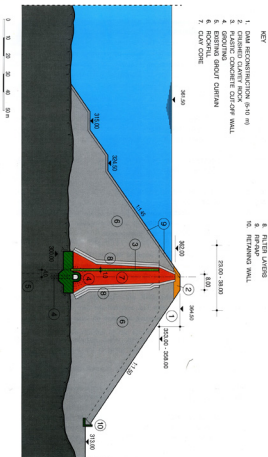


Fig. 2. Final solution - typical cross section

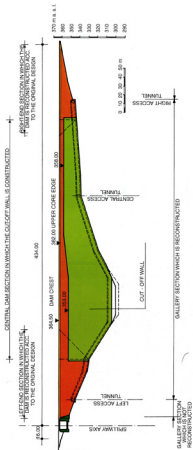


Fig. 3. Final solution - longitudinal section

- reconstruct the spillway and the gate house.

The present solution was accepted by the Croatian National Electricity Evaluation Commission in the City of Split on June 25, 1993.

The preparatory works started for the grouting gallery remediation. The remediation started on August 1, 1993, when the access tunnel No. 2 in station 2+73.25 was entered. By July 1, 1994 the remediation of the gallery was completed until the station 1+33.47, including the lowest central part of the gallery on the bottom elevation of 305.00 m a.s.l. The remediation of the gallery towards the left dam end, to the station 0+90.58, was not continued since it was estimated that the material displacements and sliding should be expected, which would endanger the lives of the workers. It has been planned that the right dam end remediation should be continued after the left dam end gallery remediation is completed in the open trench.

The works on removal of the upper dam part material to the El. 353.00 m a.s.l. and subsequent excavation in the left and right dam end should have started in February 1993, but since the enemy attacks were permanent the works did not start before April 6, 1994. The works in the left end were completed on July 1, 1994 and in the right dam end on September 15, 1994.

When the left dam end excavation was completed, Photo 1, the grouting gallery reconstruction works started between the stations 0+34.55 and 0+90.58.

The left dam end excavation enabled continuation of works on the grouting gallery excavation and reconstruction between the stations 0+90.58 and 1+30.00 which was planned to be performed by the tunnel method.

The grouting gallery reconstruction had the priority. The backfilling of the left dam end had to be started as soon as possible since the preparatory works for the cut-off wall construction were scheduled for the beginning of September 1994.

Therefore, the coordination and unobstructed progress of the works had to be ensured on the grouting gallery in the open part of the left dam end, station 0+34.55 to 0+90.58. Then, the reconstruction was continued by the tunnel method, from the open part towards the central dam section, station 0+90.58 to 1+30.00, and building in of the material in the left end, from the station 0+90.58 to the spillway, from the El. 328.58 m a.s.l. to 353.00 m a.s.l.

In order to enable parallel work on reconstruction of the external and tunnel section of the grouting gallery, and avoid the impact of the vibrating roller during building in of the material in the part of the gallery being reconstructed by the tunnel method, the lowest part of the external gallery between the stations 0+80.50 and 0+85.80 was completed last.



Photo 2. July 2, 1994 - Left dam end - completed excavation



Photo 3. September 15, 1994 - Right dam end - completed excavation



Photo 4. Grouting gallery excavation between the stations 0+90.58 and 0+93.00 from July 12 to July 28, 1994, through the uncreted part of the gallery

Through this part the gallery excavation material was removed from gallery in the stations 0+90.58 to 0+93.00, where the tunnel method of reconstruction was applied.

The works on this critical part of the gallery reconstruction were completed on July 21, 1994, and the building in of material in the left end started on July 28, 1994.

After completion of excavation on the dam crest to the EL. 353.00 m a.s.l., the stage I investigation works were conducted from May 12 to June 28, 1994.

When the stage I investigation works were completed, the core consolidation grouting was started along the cut-off wall route. The works started on July 1, 1994 and were completed on October 7, 1994. These works were performed according to the schedule and did not obstruct other remediation works.

The investigations and the core consolidation grouting along the cut-off wall route were conducted by Geotehnika Zagreb.

The Croatian National Electricity contracted the impermeable plastic concrete cut-off wall construction with Geotekhnika Zagreb and Bauer Schrobenuhausen on August 8, 1994. The contracted schedule of works was 6 months.

The preparatory works started on September 19, 1994. In the meantime, 10/04 kW substation, 630 kVA, was installed.

The excavation works for the impermeable cut-off wall started on October 13, 1994 from the panel P1, stations 3+03.77 to 3+10.77.

The cut-off wall construction started from this panel because the building in of the material in the right end to the EL. 358.00 m a.s.l. and in the left end to EL. 353.00 m a.s.l. was not completed, and the grouting gallery was not reconstructed in the left end.

On October 17, 1994, during excavation of the panel P7 between the stations 2+47.97 and 2+54.97, the slurry burst into the grouting gallery.

Although the consolidation grouting was completed along the cut-off wall route all the cavities were not filled, so the slurry was being lost and it started seeping into the gallery.

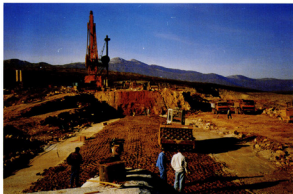


Photo 5. November 29, 1994 - Plastic concrete cut-off wall construction, station 1+55.00 and dam body construction in the left end; EL. 347.00 m a.s.l.

The concrete placement for the excavated panel was immediately started. The thick plastic concrete mix filled in all the cavities, and further seepage of the cement-bentonite suspension into the gallery was stopped.

To prevent further ingress of the slurry into the gallery, the contact grouting was performed of the joint between the gallery concrete lining and the clay core, between the stations 1+82.00 and 2+77.00. The grouting profiles were set in 10 m intervals, to be subsequently increased to 2 m intervals.

These measures have been undertaken also because simultaneous works were conducted on reconstruction of the grouting gallery in the central dam section, from the station 1+33.47 towards the left dam end, so any major ingress of slurry could have imperilled the lives of the workers who worked in the lowest part of the gallery.

The additional measures of precaution were taken, so any excavation for the cut-off wall had to be monitored by a Geotechnika-Bauer expert, in order to prevent bursting of the slurry into the gallery.

After the contact grouting had been completed, the bentonite slurry did not ingress the grouting gallery.

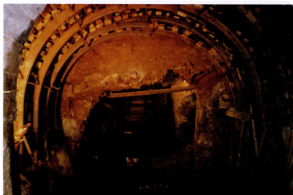


Photo 6. November 16, 1994 - Grouting gallery joined from the right and left dam end in station 1+28.00

Since the progress of the cut-off wall construction was faster than planned, the discrepancy between the works occurred, i.e.

- the dam was not completed to the elevation from which the cut-off wall was to be constructed, i.e. 353.00 m a.s.l. in the left and 358.00 m a.s.l. in the right end;
- the grouting gallery was not reconstructed between the stations 1+22.00 and 1+33.47;
- excavation of the cut-off wall panel P16 between the stations 1+64.27 and 1+71.27 was simultaneous with the gallery reconstruction works between the stations 1+26.00 and 1+32.47.

The distance between the cut-off wall and the gallery reconstruction trenches was only 32 m, so the risk was realistic of slurry seeping through the unfilled dam core cracks into the gallery.

The problem was that the drainage from the gallery, station 1+42.00, was not cleaned, so that the water and the slurry had to be pumped through the gallery and the access tunnel No. 2.



Photo 7. Dam body construction works above the
El. 353.00 and 358.00 m a.s.l

Due to the good site organization and favorable weather conditions for placement of clay, all the works related to the undisturbed and timely completion of cut-off wall were finished, i.e.

- building in of the right dam end material on the El. 358.00 m a.s.l., between the stations 3+13.00 and 3+47.97, completed on October 27, 1994;
- building in of the left dam end material on the El. 353.00 m a.s.l., completed on December 19, 1994
- the grouting gallery excavation from the left dam end in station 1+28.00 on November 16, 1994 enabled connecting with the reconstructed gallery section; Photo 6; placement of the gallery lining concrete was finished on November 24, 1994.

On January 10, 1995 the cut-off wall construction was finished, which could be taken as completion of the Peruća Dam remediation to the El. 535.00 m a.s.l. in the left, namely to the El. 358.00 m a.s.l. in the right dam end.

The quality control of the plastic concrete during the cut-off wall construction was carried out by the contractor, Geotekhnika-Bauer, and the Materials Department of the Civil Engineering Faculty, University of Zagreb.



Photo 8. June 21, 1995 - Spillway activities



Photo 9. July 1, 1995 Spillway works and dam material building in reservoir level El. 352.76 m a.s.l.

The cut-off wall integrity test was conducted by the Intergeo-Interconsul from Salzburg, between April 22 and May 1, 1995.

Until completion of the cut-off wall the works on the spillway structures were carried out with difficulties because they hindered the works on the dam body and cut-off wall construction which had the priority. Therefore, the works on removal of the parts of the spillway which was damaged in explosion started only when the cut-off wall construction was finished. Placement of the concrete for the new spillway started on April 13, 1995. Early in May 1995, when the weather conditions were favorable for placement of clay, the works on the dam body started from the El. 353.00 m a.s.l., namely 358.00 m a.s.l. to the crest on El. 364.50 m a.s.l. The building in of material into the dam body was completed in September 1995.

ORGANIZATION OF WORKS WITH RESPECT TO THE VICINITY OF THE ENEMY ARTILLERY

After the explosion on the Peruća Dam, the investigation works were required as the basis of the remediation design. The investigations planned on the dam crest were not possible because of the vicinity of the enemy artillery. The mortar explosions were frequently heard from Hrvace and Sinj, so until the beginning of August only the land surveying and visual inspections were carried out.

The caverns were noticed, i.e.

- between the stations 1+52.16 and 1+62.21, on February 16, 1993
- between the stations 0+82.00 and 0+89.00, on March 4, 1993, Photo 23
- in station 3+62.00 on May 2, 1993, Photo 24
- and in station 3+42.00 on June 3, 1993, Photo 24



Photo 10. 7:10 am September 2, 1993 - Direct hit on the Peruća HPP power house by the enemy artillery

The caverns were funnel shaped, reaching in depth from the dam crest to the grouting gallery. The assumption that they reached the gallery was confirmed during the gallery reconstruction.

The construction site for the grouting gallery reconstruction was organized on August 1, 1993. The organization was set so that the construction site exposure to the enemy was the minimum possible although the dam visibility from the enemy positions was very good.



Photo 11. January 1994
- Workers' shelter

The mortars were launched on the wider Peruća area daily, and on October 2, 1993 a mortar exploded in the vicinity of the bus transporting the Konstruktor-Inženjering workers engaged on the grouting gallery remediation, while the other hit the Peruća HPP power house, Photo 10.

It was not possible to organize the work on the dam crest because of the possible enemy actions. If there was a possibility of working on the left dam end excavation, it would enable determination of the condition of the grouting gallery between the stations 0+45.00 and 1+30.00. This would have accelerated the excavation works on the damaged dam sections and shorten the total dam remediation schedule.

During October and November 1993, the UNPROFOR gave the guarantees that the works from the dam crest could be started.

In January 1994, the shelters for the workers were built, Photo 11.



1	9/2/93 7 ^h ① 7 ^h ② 7 ^h
2	2/19/94 11 ^h ① ② ③ ④
3	2/21/94 15 ^h ① ② ③ ④ ⑤
4	3/10/94 12 ^h ① ② ③ ④ ⑤

4 5

Photo 12. Places where the enemy mortars exploded

On February 15, 1994, the excavation from the dam crest started, to be interrupted at 11:30 am on February 19, 1994 when the enemy artillery hit the narrower Peruća Dam site with five mortars, Photo 12. Fortunately, without no serious consequences. The enemy attacks followed on February 21, 1994 and March 10, 1994. The dam body excavation works were stopped because the mortars exploded in the immediate vicinity of the workers and the machinery.

When the ceasefire agreement was signed between the Croatian authorities and rebellious Serbs on April 5, 1994, Serbs withdrew their artillery for 10, namely 20 km. On April 6, 1994, the removal of the damaged dam parts started and since then the works have been performed without obstructions but with maximum precaution.

It should be noted that from January 28, 1993, when the explosive in the Peruća Dam was blasted, until completion of the dam remediation works no worker from any company engaged on the remediation refused to perform his duties, which helped successful completion of works.

INVESTIGATIONS

In April 1993, the Stage I Investigation Works Program was prepared. It included the geotechnical and geophysical investigations of the Peruća Dam condition after the damage it suffered by the explosion. The investigation works were to be the basis for the dam remediation design documentation.

Since the investigations were prevented by the war operations, the Peruća Dam Remediation Board of Experts convened on December 7, 1993 decided that the investigations shall be conducted from the El. 353.00 m a.s.l. after the top dam section has been removed.

The objective of the investigation works was to determine the dam body, bedrock, grouting gallery and grout curtain damage extent and degree, and to correct the assumptions on which the dam remediation technical solution was based.

The works were performed by the Geotekhnika Zagreb and MOHO Zagreb companies from April 5, 1994, when the preparatory works started, namely May 12, 1994 when the drilling of the investigation boreholes started, to June 28, 1994.



Photo 13. Onset of the grouting gallery reconstruction works



Photo 14.
July 2, 1994
Left dam end and
grouting gallery

Photo 15.
July 10, 1994
Grouting gallery after
completion of excavation on
the station 3+42.00
- right dam end

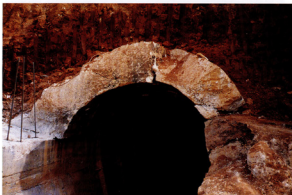




Photo 16. Grouting gallery reconstruction site facilities



Photo 17. Access tunnel No. 2 entrance

GROUTING GALLERY RECONSTRUCTION

After the Peruća Dam remediation solution had been accepted, the preparations for the grouting gallery reconstruction started. The reconstruction was planned for several reasons, the main being that the reconstruction works presented the investigation which would give the insight into the clay core damage in the area of the highest hydrostatic pressure along the dam length, including the sites in which the dam was mined.

The reconstructed gallery enabled contact grouting of the plastic concrete cut-off wall and bearing soil, and consolidation grouting. During the structure exploitation, the gallery would be used for emergency actions in case of the water seepage into the gallery.

Since the damage in the inspection gallery and the remediation procedure are given in detail in another paper, only a brief overview shall be given with respect to the other remediation activities.

The grouting gallery reconstruction was started on August 1, 1993, when it was entered through the access tunnel No. 2. Although it was planned for the gallery to be completed within 5 to 6 months, the damage was more extensive than expected. The expanded scope of works and hindered entrance into the gallery through the access tunnel No. 3 in the left dam end prolonged completion of the works, and therefore the entire remediation completion, to 16 months of work in three shifts.

The reconstruction of the grouting gallery was the most difficult tunnel operation, and one of the most difficult activities during the dam remediation.

The daily uncertainties all the participants in the grouting gallery remediation were facing were:

- presence on unexploded mines-explosives,
- presence of caverns in the clay core,
- possible seepage in the gallery,
- impossible use of excavation machinery,
- frequent general alerts and mortar shelling,
- interrelation between the various activities.

Site Organization

The site organization for the grouting gallery reconstruction was adapted to the on-site conditions. Special attention had to be paid to minimization of traffic and movement of people off the gallery, since they were well visible from the enemy positions.

In front of the access tunnel No. 2, in the right end, a platform has been prepared for the operating and working facilities, cement mixing plant, jet concrete mixer, fan and compressor stations.

The site organization of the grouting gallery remediation is given in another paper.

The access tunnel No. 2 and reconstruction works to the joint with the gallery presented no problem. The joint itself posed certain problems because of the caving in.

From the joint, the reconstruction went towards the deeper part of the gallery, i.e. towards the straight section on El. 305.00 m a.s.l.



Photo 18.

May 1, 1994. Station 1+58.00

**Testing the presence of captured
water and material composition
in 11.0 m borehole**

The optimistic attitude that the gallery can be remediated in 5 to 6 months was based on a view from the joint in the direction of the gallery section in the right end, as shown in Photo 1.

The gallery remediation to station 2+00.00 caused no major problems. Entrance into a completely caved in part of the gallery made the work more difficult, so the work progress was 40 cm per day.

The additional difficulty was posed by the inflowing water. The maximum inflow was 130 l/min, at the reservoir level of 330.00 m a.s.l. The further progress was also slowed down by the cavern in the gallery apex in station 1+90.35.



Photo 19. September 21, 1994 Gallery drainage pipe excavation
in station 1+42.00

The major gallery damage, including the gallery bottom, was encountered in its central part, between the stations 1+52.00 and 1+62.00.

Until July 1, 1994 the grouting gallery remediation was completed in the flat section, El. 305.00 m a.s.l., to the station 1+33.47. Further remediation from this section towards the left end to the station 0+90.58 was stopped because of the risk of rockslides and material displacement.



Photo 20. October 19, 1994 Preparation of formwork for placement of concrete between the stations 1+17.53 and 1+22.78

In station 1+42.00, a 2.0 x 1.0 m opening was left in concrete in the point from which the drainage pipe goes to the Peruća HPP switchyard.

The entrance was completely caved in. Further reconstruction of the drainage outlet was started in September 1994, Photo 19.

Gallery excavation and reconstruction from the left end to the station 0+90.58 started on July 12, 1994. From the station 1+16.30 towards to the right damend the gallery was completely caved in.

Excavation and reconstruction to the station 1+24.00 was completed on October 26, 1994, so the reconstruction from the station 1+33.47 in the right dam end was started. The work progressed on both sides to the station 1+28.00, where the breakthrough was achieved on November 6, 1994, Photo 6. Placement of concrete for this final section between the stations 1+26.00 and 1+35.00 was finished on November 24, 1994. This meant completion of the grouting gallery reconstruction.



Photo 21. October 11, 1994 Access tunnel to the grouting gallery in the left dam end



Photo 22. July 21, 1994 Grouting gallery works in the right dam end

Altogether, 462.57 m of gallery was reconstructed, including:

- | | | |
|-----|--|----------|
| (1) | entirely new concrete after excavation of the left and right dam ends | 130.60 m |
| (2) | entirely new concrete by the tunnel trench | 87.47 m |
| (3) | gallery arch reconstruction by the new concrete after the excavation
in the right dam end | 42.00 m |
| (4) | shotcrete reconstruction | 202.50 m |

RECONSTRUCTION OF THE DAM ENDS AND UPPER SECTION

Reconstruction Description

The left and right end, as well as the 5 - 10 m high upper part of the central dam section were heavily damaged in explosion, so the reconstruction was required. The reconstruction consisted in removal and rebuilding in of material.



Photo 23. Crater between the stations 0+82.00 and 0+89.00



Photo 24. A view from the right dam flank towards the craters in station 3+62.00 formed on May 2, 1993 and 3+42.00 formed on June 3, 1993

The left dam end was reconstructed between the stations 0+25.00 and 0+90.58. The bottom elevation in station 0+90.58 upstream the gallery was 328.56 m a.s.l., and downstream 330.25 m a.s.l. In this section the completely destroyed gallery concrete was removed. The grouting gallery was rebuilt after the excavation works were completed.

The right dam end was reconstructed between the stations 3+42.00 and 4+50.00; the lowest excavation elevation on the station 3+42.00 was on 347.50 m a.s.l. The grouting gallery was also reconstructed in this section, between the stations 3+42.00 and 3+87.00, as well as the access to the gallery.

The upper part in the central dam section, between the stations 1+13.00 and 1+83.00 was removed to the El. 353.00 m a.s.l., and between the stations 2+38.00 and 3+64.00 to the El. 358.00 m a.s.l. These two parts were connected by a transitional ramp.

On the stretch between the stations 1+13.00 and 1+83.00, after the dam material was removed, the 23 m wide platform was built; also, between the stations 2+38.00 and 3+29.00, a 38 m wide platform was built from which the cut-off wall was constructed. Between the stations 1+83.00 and 2+38.00, the platform is built of width varying between 23 and 38 m.

In parallel with removal of ends and the central dam section the parts of the spillway were removed, i.e. bridge, right flank, a part of the right chute wall, damaged hydromechanical equipment, and the spillway gate house.

The excavated material of the first 5 m of the dam body was built in into the operating platform in the right end to the El. 358.00 m a.s.l. in which the cut-off wall construction facilities were placed. The material excavated between the El. 358.00 m a.s.l. and 353.00 m a.s.l. is separated and removed to the spoil areas. The location of the spoil areas, including the site facilities, is shown in Fig. 4.

Site Organization

Although removal of the central part and dam ends was started on the February 15, 1994, they were stopped because the enemy shelled the dam. The works were not continued until April 6, 1994, namely after signing of the ceasefire agreement between the Croatian authorities and the rebellious Serbs on April 5, 1994. The remediation works were not interrupted since.

The site organization was conceived so that the complete activity was carried out both on the dam and on the right reservoir flank upstream the dam. The access roads used during the dam construction were reconstructed. Upstream the dam, the right reservoir flank houses the quarry which was used in dam construction.

The excavated material was dumped in the quarry, on El. 343.00 m a.s.l. The clay was dumped in the right flank on El. 353.00 m a.s.l.

The excavation works on the left and right end, and in the central dam section included:

- (a) Excavation of the clayey rock and similar mixed material from the dam crest and removal to the permanent or provisional spoil area.
- (b) Excavation of the core clay and removal to the provisional spoil area for rebuilding.
- (c) Excavation of the filter material and removal to the permanent spoil area. The highly mixed material prevented its selecting, so it was not used for rebuilding.

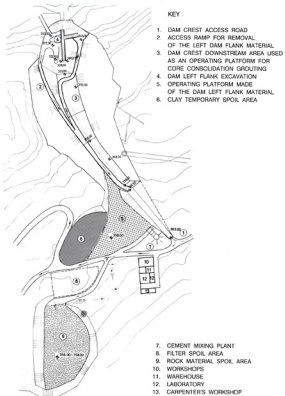


Fig. 4. Dam body site organization

- (d) Excavation of the rock material from the upstream and downstream shells, and removal to the provisional spoil area fore rebuilding.

It should be noted that all the design assumptions on the damaged gallery part in the left and right dam end have proven to be true.

Removal of the dam ends and top section

The works on removal of the dam material can be presented in three stages:

Stage I - removal of material from the central dam section to the El. 358.00 m a.s.l., namely 353.00 m a.s.l.

The material excavated to these elevations was used for construction of the operating platform used as an access to the central dam section for unhindered performance of works on the left and right end, and in the central dam section.

This platform and the access road in the right dam end are on the El. 358.00 m a.s.l. Since early August, it was used for the cut-off wall construction.

The removal included:

- the existing asphalt course from the dam crest dumped downstream
the dam 1,800 m³
- clayey and similar material 25,730 m³
- from the clay core, between the El. 362.00 m a.s.l. and 358.00 m a.s.l. 3,410 m³
- rock and filter material from the dam body between the
El. 358.00 m a.s.l. and 353.00 m a.s.l. 24,885 m³
- clay core material between the El. 358.00 m a.s.l. and 353.00 m a.s.l. 4,900 m³

Stage II - removal of material from the left dam end to the existing terrain in station 0+90.58 to El. 328.56 m a.s.l. upstream the grouting gallery, namely 330.25 m a.s.l. downstream the gallery.

The construction pit access from the platform on El. 353.00 m a.s.l. to the ultimate excavation elevation 328.56 m a.s.l. upstream the gallery, namely 330.25 m a.s.l. downstream the gallery has been realized by the access ramps built on the upstream and downstream dam shells, Photos 25 and 26.

The slide slope ratio towards the central dam section and the shell zones is 1:1, and the trench width in the bottom is about 14 m.



Photo 25. Excavation in the left dam end - Traffic on the upstream ramp



Photo 26. Access ramp towards the left dam end

The downstream slope, due to a somewhat steeper slope, has been protected with the net to avoid the material rockslide towards the grouting gallery.

Total quantities of material excavated and removed from this dam section, between the El 353.00 m a.s.l. and 328.56 m a.s.l. upstream the gallery, namely 330.25 m a.s.l. downstream the gallery:

-	rock and filter material	42,602 m ³
-	core clay	6,526 m ³

Stage III - removal of material from the right dam end from El. 358.00 m a.s.l. to the natural ground level.



Photo 27. July 2, 1994 Right dam end - damaged grouting gallery top from the station 3+42.00

Since the dam is low in this part, and its maximum height is 15.5 m, the dam material was excavated along the entire cross section.

The excavated material was removed over the ramp to the access road in the towards dam.



Photo 28. July 2, 1994 Left dam end excavation completed



Photo 29. July 2, 1994 Right dam end - removal of damaged gallery sections

Total excavated and removed material between the El. 358.00 m a.s.l. and 347.50 m a.s.l.:

- rock and filter material	10,577 m ³
- clay core material	4,468 m ³
- foundation ground excavation at the clay core and filter contact with the natural ground	1,272 m ³

After the left and right dam ends had been removed, all the assumptions on the grouting gallery damage to these stations were confirmed.

It is also clear that the excavation in these two cross-sections of the left and right dam ends did not indicate that the dam body, which is not removed, had suffered considerable damage.

The cross-sections do not show the dam body between the stations 1+52.00 and 1+62.00 where the explosive was set and blasted in the grouting gallery.

The material excavated and removed from the dam body totals 124,672 m³ of clay, filter and rock material, which is approximately 14% of the total dam volume.

The excavation and removal works on the dam body to the El. 358.00 m a.s.l., namely 353.60 m a.s.l., and the excavation and removal in the left dam end to the El. 328.56 m



Photo 30. July 15, 1994 Right dam end - completely finished excavation



Photo 31. July 1, 1994 Left dam end - completely finished excavation

a.s.l. were performed from July 6, 1993 to July 1, 1994, while the excavation and removal of material from the right dam end to the El. 347.50 m a.s.l. was finished by July 15, 1994, Photos 30 and 31.

The works were conducted by the Konstruktor-Inženjering company from Split.

Building in of Material into the Dam Body

As already described, the Peruća Dam was reconstructed in the major destruction sites, i.e.

- left dam end, between the stations 0+25.00 and 0+90.58, in full height
- right dam end, between the stations 3+42.00 and 4+50.00, in full height
- upper dam section, to the El. 353.00 m a.s.l., between the stations 1+13.00 and 1+83.00, namely to El. 358.00 m a.s.l., between the stations 2+38.00 and 3+29.00
- between the stations 1+38.00 and 2+38.00, the excavation as well as the material building in into the dam body was step-shaped.

The remaining part of the dam, in its central section, between the stations 0+87.32 and 3+47.97 was not reconstructed; the plastic concrete cut-off wall was built instead.

The dam body consists of:

- clay core
- upstream and downstream filter layers
- upstream and downstream rock shells
- dam crest of clayey rock material.

The upstream and downstream dam slope was to 1.5 m thick, namely 2.7 m wide in horizontal line, built of large rock blocks, under the slide slope ratio 1:1.45. The slide slope ratio of the downstream slope is 1:1.5. The dam crest width on the El. 364.50 m a.s.l. is 8.0 m.

The clay core is built asymmetrically to the dam crest. The core bottom section houses the reconstructed grouting gallery.

The upper clay core El. is 363.00 m a.s.l.

The clayey gravel was placed between the El. 363.00 m a.s.l. and 364.50 m a.s.l., as protection against the clay core drying out.



Photo 32.

August 2, 1994

Left dam end

Beginning of backfilling
downstream the grouting
gallery

Photo 33.

August 12, 1994

Beginning of the clay
backfilling in the right dam
end, El. 347.80 m a.s.l.





Photo 34. September 7, 1994 Building in of the material into the left dam end, El. 338.50 m a.s.l.



Photo 35. August 12, 1994 Beginning of the material building in into the right dam end

In the deepest left end section the clay core foundation is on the El. 328.56 m a.s.l. on the upstream, and 330.25 m a.s.l. on the downstream grouting gallery end. In this point the clay core is 12.5 m wide. The trench end, which is the starting point of the building in into the dam body is in station 0+90.58, Photo 31.

In the deepest right end section in station 3+42.00 the clay core bottom is on the El. 347.50 m a.s.l., and the clay core is about 9.0 m wide, Photo 30.

Clay core is protected from the upstream and downstream end by 2.0 m wide filter layers. In the dam crest, in the low hydrostatic pressures zone, the upstream filter is narrowed to 1.5 m.

The upstream and downstream shells are made of rock material taken partly from the dam body and disposed during excavation, while the remaining material was brought in from approximately 700 m distanced quarry to the central dam section.

Widening of the downstream shell to 5.0 m was executed with rock material and lateral step-shaped cutting in into the existing dam body.

Material Built in into the Dam Body

The materials used in dam body construction:

- clay for the dam core
- rock material for filters
- rock material for shells
- clayey material for dam crest.

When the excavation of 5.0 to 10.0 m in the central dam section, left dam end to the El. 328.56 m a.s.l., and right end to the El. 347.50 m a.s.l. was completed, the material was sorted into separate spoil areas and built in again.

The technical monitoring instruments were installed into the dam body, which is described in a special paper.

The dam downstream slope and crest contain benchmarks for monitoring of dam deformations.

The quality assurance and quality control program is detailed in a special paper.

Clay

The core construction material, the core being the impermeable part of the dam, is clay of intermediate to high plasticity CI/CH, taken from the spoil area on which it was



Photo 36. August 19, 1994 Backfilling the left dam end,
El. 334.10 m a.s.l.



Photo 37. March 10, 1995 Building in of the material into the
downstream shell; dam widening for 5.0 m; El. 345.00 m a.s.l.



Photo 38. September 1, 1994 Clay building in into the right dam end; El. 348.50 m a.s.l.

disposed when excavated from the earlier excavated core (10,000 m³), while the rest (21,277 m³) was brought in from the Zorice deposit. It was compacted in 30 cm layers using sheep foot rollers, with permanent control of water content and compactness.

The built in quantities are:

-	left dam end, between the El. 328.50 m a.s.l. and 353.00 m a.s.l.	..	6,344 m ³
-	right dam end, between the El. 347.50 m a.s.l. and 358.00 m a.s.l.	.	4,375 m ³
-	central dam section, from El. 357.00 m a.s.l.,		
	namely 358.00 m a.s.l. to El. 363.00 m a.s.l.		20,558 m ³
Total built in clay quantity			31,277 m ³

Filter material

Filter material is used to prevent the internal clay core erosion. The filter material was transported from the Srinjine quarry. The compacting was performed by a small vibrating compactor.



Photo 39. September 23, 1994 Building in the right dam end filter; El. 353.20 m a.s.l.

The built in quantities are:

-	left dam end, between the El. 328.50 m a.s.l. and 353.00 m a.s.l.	4,820 m ³
-	right dam end, between the El. 347.50 m a.s.l. and 358.00 m a.s.l.	2,150 m ³
-	central dam section, from El. 353.00 m a.s.l., namely 358.00 m a.s.l. to El. 363.00 m a.s.l.	9,016 m ³
Total built in filter material quantity		15,986 m ³

The upstream and downstream shells material

The rock material of various granulometric composition was built into the shells and as upstream slope protection.

The material was brought in from the spoil area for the material excavated from the reconstructed dam body, and the rest was taken from the quarry on the right reservoir bank upstream the dam. The quarry material was obtained by the deep blasting method.

The quarry capacity met the construction requirements.



Photo 40. February 27, 1995 Building in of rock material into the downstream shell; El. 339.50 m a.s.l.

The built in quantities are:

- left dam end, between the El. 330.00 m a.s.l. and 353.00 m a.s.l. . . 35,420 m³
- widening of the downstream shell in the left dam end from the El. 353.00 m a.s.l., namely 358.00 m a.s.l. in the right dam end . . 36,775 m³
- left dam end, between the El. 347.50 m a.s.l. and 358.00 m a.s.l. . . 7,740 m³
- central dam section, from El. 353.00 m a.s.l., namely 358.00 m a.s.l. to El. 364.50, namely El. 365.00 m a.s.l. 47,893 m³

Total quantity of rock material built in into the reconstructed dam body 127,828 m³

Clayey Rock Material

The clayey rock material is used for protection of the dam clay core top section against drying out. It is built in into the dam body between the El. 363.00 m a.s.l. and 364.50m

a.s.l., namely 365.00 m a.s.l. On the upstream and downstream end this material is protected with 7,500 m³ of rip-rap.

Upstream Dam Slope

The 1.5 m thick rip-rap was placed on the upstream dam slope, in the segment in which the dam was reconstructed. It contains large rock blocks dumped using the excavator. The rock blocks were placed so as to be stable under the wind-induced waves.

The upstream shell in unreconstructed dam section should be monitored. The waves could erode the finer rock material with time, the blocks might move. Such points of damage shall be subsequently remediated. The slide slope ratio is 1:1.45.

Downstream Dam Slope

After the downstream shell had been well compacted, the subsequent forming of slide slope in ratio 1:1.5 was performed in 2.0 m intervals.

The slope was planted with Spanish broom (*Spartium Junceum*) which was predominant on the downstream slope before the dam blasting. These seedlings shall prevent rockslides down the slope in the direction of the switchyard.

Total Quantities of Removed and Built in Material

Total quantity of material built in into the dam body is 175,091 m³.

Total excavated and removed quantity is 124,672 m³. Therefore, 50,419 m³ more material has been built in because of the downstream shell widening.

The works on the dam body started on July 28, 1994, and shall be completed in September 1995.

DAM BODY WIDENING ON THE DOWNSTREAM SIDE

Retaining wall on Downstream dam toe

Based on the dam stability analyses under the seismic loads and evacuation of 10,000-year floods over the spillway, the conclusion was brought that the dam should be heightened from El. 363.00 m a.s.l. to the El. 364.50 m a.s.l. . The reasons and the methods of this operation are covered in a special paper.

The explanation has also been given for dam heightening by backfilling from the downstream end.



Photo 41. August 2, 1994 Downstream dam toe retaining wall during construction

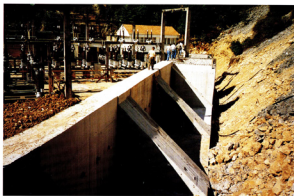


Photo 42. September 7, 1994 Constructed downstream dam toe retaining wall

Since the widening toe penetrates into the switchyard, the 3.5 m high retaining wall has been built, Photos 41 and 42. Also, the bottom outlet auxiliary gate house was protected, and the access tunnels Nos. 2 and 3 extended for 6.0 m. The stairs were built along the dam slope in the left and right dam end, along with the cable ducts leading to the spillway structure and auxiliary gate.

During the excavation and the retaining wall concreting activities the switchyard was disconnected between 8:00 am and 19:00 pm, so the works progressed smoothly.

The excavation for the retaining wall on the downstream dam toe started on July 2, 1994. The total of 1,165 m³ of material was excavated, 775 m³ of concrete built in, and about 46,178 kg of reinforcement bars (steel grade 400/500) built in.

The wall was concreted in eight segments between July 8 and September 1, 1994.

In order to enable building in of the material behind the retaining wall, the temporary ramp was built from the switchyard by September 7, 1994, and the rock material building in to the top wall section, i.e. El. 317.00 m a.s.l. was finished by September 30, 1994.

The material building in was continued according to the free capacities of the on-site machinery, under the condition that the construction be finished to the El. 353.00 m a.s.l., namely 358.00 m a.s.l. until the planned deadline for the beginning of construction from these elevations along the entire dam width, which was early May 1995.

The material from the quarry and the dam body material spoil area was used.

The machinery included:

- bulldozer D-6
- excavator for material loading
- roller
- 4-5 trucks.

On February 27, 1995, the rockfill reached the El. 339.50 m a.s.l., and on April 15, 1995, the El. 353.00, namely 358.00 m a.s.l.

Protective wall around the Bottom outlet auxiliary gate house

The bottom outlet auxiliary gate house is located immediately by the dam toe, in the right end. The house partly enters into the downstream support zone. The house encompasses the auxiliary gate opening and closing mechanism.



Photo 43. Bottom outlet auxiliary gate house and protective wall

Since the dam is heightened to the final El. 364.50 m a.s.l., and because of the settlement on the El. 365.00 m a.s.l. the backfilling of the downstream shell expands to 5.0 m, the slope of the new shell would cover the house roof. Therefore, a protective wall (maximum length 28.0 m, maximum height 9.6 m, width in toe 4.65 m) was built around the bottom outlet auxiliary gate house, Fig. 5; Photo 43.

The protective wall is shaped so that on the upstream side it follows the round form of the house, while on the downstream side it departs from the house walls and goes to the dam toe.

The maximum wall height is 9.6 m, and the minimum is 1.55 m in the toe. The wall thickness, but in the segment which leans against the house, is 65 cm; the foundation toe thickness is 1.0 m. The wall segment leaning against the auxiliary gate house is 1.0 m thick. The 50 cm thick drainage layer is placed behind the wall.

The works were completed between November 14, 1994 and March 21, 1995.

Total quantity of work:

- | | | |
|---|----------------|--------|
| - excavation in the downstream dam slope with removal to the spoil area | m ³ | 538.00 |
| - preparation, transportation and placement of M-30 concrete | m ³ | 176.00 |

- preparation, transportation and placement of M-20 base concrete m³ 61.00
- purchasing, bending and embedment of ribbed reinforcement 400/500 t 10.50
- transportation and placement of filter material for wall drainage behind the wall m³ 74.00

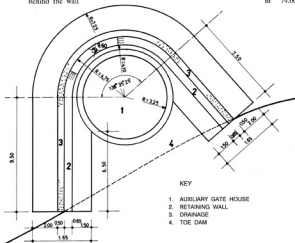


Fig. 5. Protective wall around the bottom outlet auxiliary gate house

Performance of these works did not affect the schedule of other dam remediation works.

CLAY CORE CONSOLIDATION GROUTING ALONG THE CUT-OFF WALL ROUTE

Construction of the impermeable plastic concrete cut-off wall was planned in the central dam section, between the stations 0+87.32 and 3+44.00. It was intended to take over the function that was played by the clay core before the dam damage.

It was presumed that the slurry losses could have been suffered during excavations for the cut-off wall. The function of the slurry is to protect the trench walls. The slurry could be

lost in case there are cavities in the clay core as consequence of the explosion. The slurry loss would cause caving in of the trench, and therefore partial trapping of the excavator.

During the grouting gallery remediation activities in station 1+90.35 the cavern was encountered in the gallery apex which lead to a conclusion that there are additional cracks and cavities in the clay core caused by the explosion.

The slurry loss or trapping of the excavator would cause loss of a part of the excavation machinery and huge expenses incurred by the release operation and works stoppage.

To reduce the slurry loss to minimum, on December 7, 1993 the Peruća Dam Remediation Board of Experts decided on consolidation grouting along the cut-off wall route.

The essential objective of the consolidation grouting along the edge of the trench for the future cut-off wall was detection and filling of any possible cracks and caverns in the dam core and filter layers partly penetrated into by the cut-off wall, in order to prevent an abrupt slurry loss during performance of works.

The consolidation grouting was planned between the stations 0+87.70 and 3+44.00, from the boreholes set in 1.4 m intervals, from the cut-off wall upstream and downstream end.

The consolidation grouting is covered by a special paper.

The consolidation grouting was conducted between the stations 1+14.60 and 3+19.00. The grouting in the left end, between the stations 0+87.70 and 1+14.60, and in the right end, between the stations 3+19.00 and 3+44.00 was not completed primarily due to a very tight schedule, i.e. the dam was brought to the EL. 353.00 m a.s.l. in the left end and 358.00 m a.s.l. in the right end immediately before the cut-off wall excavation works. The consolidation grouting would cause delay in the cut-off wall construction.

Another reason is that the excavation on the left and right flank slopes did not indicate major damage in the clay core and filter layers.

This assumption has been confirmed, because there was no slurry loss during the cut-off wall construction in the points in which the consolidation grouting was not performed.

The consolidation grouting was performed by the Geotecnika, Zagreb. It started on July 1, 1994, and was completed on October 7, 1994.

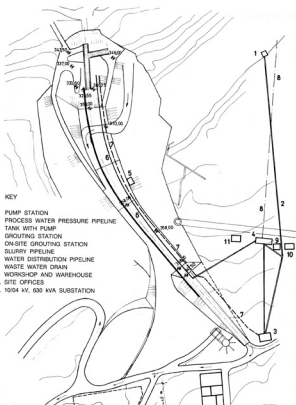


Fig. 6. Site organization for the clay core consolidation grouting along the cut-off wall route

Site Organization

The site organization for the core consolidation grouting along the cut-off wall route was such as not to disturb other dam remediation works.

Since the excavation in the central dam section and its ends started early in July 1994 was completed before the consolidation grouting began, the remaining works were:

- grouting gallery concrete placement between the stations 0+34.45 and 0+90.58
- excavation and grouting gallery reconstruction between the stations 0+90.58 and 1+30.00 from the left dam end
- placement of concrete for the access tunnel No. 3 in the left dam end
- backfilling the left dam end between the El. 328.56 m a.s.l. and 353.00 m a.s.l.

The above indicates that permanent communication had to be ensured in the left dam end.

The entire transportation was directed upstream the cut-off wall route, along the upstream shell, while between the stations 1+18.00 and 1+13.00 the transportation towards the access tunnel No. 3 was ensured over the downstream shell.

Arrangement of structures and operating areas for the consolidation grouting is given in the arrangement drawing, Fig. 6.

Quantity of Works

171 boreholes, total length 5,788 m, were drilled on the grouting route. Total slurry consumption was 591,156 l, and the average consumption was 102.13 l/m of borehole. Since 1 l of slurry contains 0.35 kg of dry matter, total consumption of cement and bentonite was 207 t.

PLASTIC CONCRETE CUT-OFF WALL

The Peruća Dam remediation includes construction of a cut-off wall which shall ensure the dam impermeability instead of the damaged clay core. The Croatian National Electricity contracted the plastic concrete cut-off wall construction with the Geotehnika Zagreb and Bauer Schrebenhausen on August 8, 1994. The contracted construction schedule was 6 months.

The preparatory works started on September 19, 1994.

The plastic concrete cut-off wall excavation works started on October 13, 1994.

All the cut-off wall construction activities were completed on January 10, 1995.

The cut-off wall was constructed between the stations 0+87.70 and 3+44.00, measured along the dam axis. Between the stations 0+87.70 and 1+82.87, the cut-off wall was constructed from the El. 353.40 m a.s.l., and between the stations 2+38.67 and 3+44.00 from the El. 358.40 m a.s.l.

Between the stations 1+82.87 and 2+38.67 the cut-off wall was constructed in four 1.0 m high steps. The cut-off wall was cut in the joints with the left and right ends for min 2.0 m into the reconstructed dam ends, measured from the lowest trench slope level. Transverse to the dam axis, the cut-off wall axis is placed 4.0 m upstream the grouting gallery axis. The maximum and minimum cut-off wall depths are 51.50 m and 12.90 m, respectively.

The cut-off wall was cut into the bedrock, namely the concrete block in the central dam section, for min 2.0 m.

The cut-off wall is 1.0 m thick, and its total length is 261.15 m.

The detailed description of the impermeable plastic concrete cut-off wall is given in a special paper.

To provide for the contact between the cut-off wall and the bedrock, namely the concrete block in the central dam section, the joints were grouted and connected with the existing grout curtain. The grouting was performed from the reconstructed grouting gallery.

Site Organization

The slurry mixing plant was located on the platform on the El. 358.00 m a.s.l., in the right end, upstream the dam.

- slurry mixing plant
- bentonite storage
- slurry desanding plant with tanks for fresh, working and waste slurry
- slurry conveyance pump to the excavation site.

The concrete mixing plant with capacity of 60 m³/h is placed upstream the dam.

The slurry mixing and desanding plants are connected to the conveying pipelines leading to the excavation, namely cut-off wall concreting site.

The equipment at the excavation site consisted of the hydraulic trench cutter, grab and crane.

The grab excavated material is transported by the trucks to the provisional spoil area on the right reservoir bank.

KEY

1. PUMP STATION
2. PROCESS WATER PRESSURE PIPELINE
3. TANK WITH PUMP
4. GROUTING STATION
5. WATER DISTRIBUTION PIPELINE
6. WASTE WATER DITCH

7. WORKSHOP AND WAREHOUSE
8. SITE OFFICES
9. RESTAURANT AND TOILETS
10. SLURRY MIXING AND DESANDING SYSTEM
11. SLURRY DESANDING SYSTEM PIPELINE
12. PRESSURE PUMP
13. SC-90 FILL MIXING PLANT
14. PROCESS WATER PIPELINE

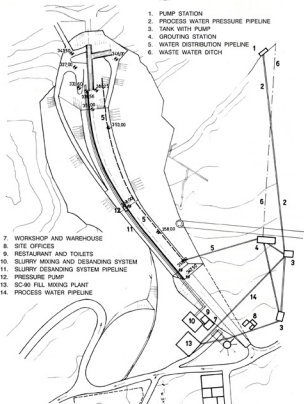


Fig. 7. Site organization for the cut-off wall construction



Photo 44. December 16, 1994 Completion of guide walls and panel excavation by hydraulic trench cutter - left dam end

The cut-off wall construction went from the right to the left dam end. The water for mixing the slurry and plastic concrete was pumped from the Cetina River downstream the power house to the tanks, and from here to the mixing plants.

The power supply was provided from the 10/04 kW BTS Substation, 630 kVA, and the existing 400 kVA substation.

The site organization is presented in Fig. 7.



Photo 45. December 2, 1994 Plastic concrete cut-off wall construction and building in of material in the left dam end

GROUTING WORKS

The accepted dam remediation concept includes the grouting works.

The below works were performed from the grouting gallery after it had been reconstructed:

1. Contact grouting, in the profiles with fire namely six boreholes in 2.0 m intervals, on the dam section in which the plastic concrete cut-off wall was built, namely between the stations 0+87.70 and 3+44.00

The grouting works were completed between October 21, 1994 and February 2, 1995

2. Consolidation grouting, with vertical boreholes along the entire gallery bottom, in two rows, upstream and downstream, with the boreholes in 4.0 m intervals in both rows.

The grouting works were completed between November 14, 1994 and March 21, 1995.

3. Contact grouting of the joint with the plastic concrete cut-off wall in the profiles with two boreholes in 2.0 m intervals.

The grouting was completed between December 20, 1994 and March 15, 1995.



Photo 46. Contact grouting in the grouting gallery



Photo 47. June 19, 1995 Left dam end grouting works



Photo 48. Main grouting post



Photo 49. Grouting from the El. 353.00 m a.s.l

After several months of monitoring in the gallery, in the points in which weeping and seepage was noticed, the additional contact and consolidation grouting.

In addition to the above grouting works, the grout curtain heightening has been performed

- (a) 53 m in the right dam end, 2.0 m and 3.0 m borehole interval, the works were completed from April 25, 1995 to May 15, 1995
- (b) 267.0 m in the left dam end, 3.0 m borehole interval, the works were completed from May 18, 1995 to June 26, 1995

The consolidation grouting under the spillway structure has also been performed. Grouting from the gallery, and in the right end, has been carried out from the main grouting post used for the consolidation grouting of the clay core along the cut-off wall route, Photo48. The site organization for the grouting works was the same as for the consolidation grouting. The grouting in the left end was performed from the auxiliary grouting post set in the left dam end.

OPERATION OF THE PERUĆA RESERVOIR DURING THE PERUĆA DAM REMEDIATION

When the explosive in the Peruća Dam grouting gallery was blasted the reservoir level was on El. 356.20 m a.s.l. A decision on complete reservoir evacuation was brought and effected on March 12, 1993 when the reservoir level was on El. 314.80 m a.s.l.

The inspection of the upstream slope and the access tunnel entrance did not show any damage or planted explosives, so the level to which the reservoir was to be impounded was specified on the basis of the extent of damage on the dam and clay core.

The reservoir utilization is important for several reasons:

- to prevent any threat to dam stability,
- to use the stored water according to the plans,
- to prevent flooding of the Hrvatačko polje and Sinjsko polje.

During the reservoir emptying, 10 m deep and about 7 m wide crater was formed in the left dam end, between the stations 0+82.00 and 0+89.00, which incorporated a part of the core. Since the grouting gallery bottom in this station is on El. 333.00 m a.s.l., the safe elevation of 330.00 m a.s.l. was determined to which the water can be stored.

Both generating sets of the Peruća HPP were not repaired after flooding of the power house, so the water had to be discharged through the bottom outlet only. One generating set was back in operation in July 1993, and the other in November 1993, so the additional water evacuation was possible.

Storage and discharge of the Peruća reservoir water is governed by the Code on the Peruća Operation Exploitation During the Peruća Dam Remediation prepared with the purpose of maintaining the safety water level in the reservoir.

The above Code particularly specifies:

- care for the dam safety,
- providing for safe discharge of floods,
- avoiding imperilment of the flood control system in Sinjsko polje,
- providing of maximum possible quantity of stored water for power generation in the downstream hydroelectric power plants.

It is important to mention that the grouting gallery works were carried out on the EL 305.00 m a.s.l. , 25.00 m below the reservoir level.

The water inflow into the grouting gallery varied with the reservoir level, and its maximum value was 130 l/min; the water was evacuated through the grouting gallery and access tunnel No. 2 using the pumps.

It may be said that the solutions were found which enabled maximum utilization of the water inflow into the reservoir for power generation in the downstream hydroelectric power plants, i.e. Peruća HPP, Dale HPP and Zakućac HPP. The contribution of the dispatchers from Split Dispatching Center was enormous.

The fall of 1994 was exceptionally humid, with 4-5 flood water waves, which continued to the spring of 1995. This confirms that the precipitations are the highest in the period when dams and hydroelectric power plants are built.

The Peruća reservoir was impounded to the EL 353.00 m a.s.l. for dam investigations and power generation purpose.

CONCLUSION

The dam related activities since the dam was blasted on January 28, 1993 can be divided into three stages:

- dam salvage after explosion
- dam remediation
- dam maintenance and monitoring of behavior after remediation.

The first two stages were successfully completed, and the dam behavior monitoring should be paid special attention in order to detect any deficiencies in due time and to correct them before they threatened the dam safety.

SEISMIC AND SEISMOTECTONIC INVESTIGATIONS AT THE PERUĆA DAM SITE

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ABSTRACT

The Peruća Dam is located in the marginal section of the regional structural unit of Dinaricum, in the fault zone dividing two major overthrust and seismogenetic structures - Svrlja and Dinara-Kamešnica. This is the area of marked seismic activity. The major earthquakes were those of 1898, $I_a=IX^+$ MCS, $M=6.5$, $D=23$ km and 1990, $I_a=8.1^+$ MCS, $M=5.6$, $D=5$ km, which manifested on the dam site in horizontal accelerations 0.16 g and 0.15 g, respectively. The possible maximum horizontal acceleration $a_{max}=0.29$ g and the macroseismic intensity $I_{max}=8.6^+$ MCS were calculated by the deterministic approach based on the seismological and seismotectonic investigations. The maximum values corresponding to 200-year return period, earthquake S_1 and 1000-year return period, earthquake S_2 are $a_{max}=0.17$ g, $I_{max}=7.7^+$ MCS, and $a_{max}=0.22$ g, $I_{max}=8.2^+$ MCS, respectively.

INTRODUCTION

The complex seismological and seismotectonic investigations were a part of numerous investigation works conducted with respect to the Peruća Dam remediation. The objective of these investigations was to define the design seismic parameters. The analysis results, description of applied methods, and used data are presented in this paper.

The investigations may be divided into three groups - seismological, seismotectonic, and engineering seismology.

The seismological investigations dealt in detail with spatial, time and energy characteristics of the seismic activity of the local and regional area neighboring to the investigated site.

The microseismic and macroseismic data were used from the Andrija Mohorovičić Geophysical Institute files. Particularly important were the results of instrumentation monitoring of seismic activity which has been systematically conducted for the Croatian National Electricity.

The seismotectonic investigations primarily used the data on neotectonic and seismic activity, while special accent was placed on the recent neotectonic activity. The seismogenetic structures were separated, and the seismotectonic model defined for the narrower regional area which affects the Peruća HPP site. The site location is set within the seismotectonic framework, and the characteristics of earthquake sources defined.

The engineering seismology investigations included calculation of maximum horizontal acceleration and macroseismic intensity. The calculation was conducted applying two approaches: deterministic, where the seismotectonic investigation results were used as the basis for determination of acceleration and macroseismic intensity values as the absolute maximums, and probabilistic, where the accelerations and intensities were calculated as the function of the return period and risk level. The calculation after the probabilistic method used the McGuire's method and theory of extremes, and the lateral anisotropic propagation of seismic energy was also assumed.

SEISMICITY CHARACTERISTICS

The Peruća Dam is located in the area of intensified seismic activity. Several recently active epicentral areas are in the immediate vicinity of the dam - Sinjsko and Livanjsko polje, Knin surroundings, areas along the mountains Dinara, Kamešnica and Svilaja. The highest seismic activity has been recorded in the Sinjsko polje, the area between Trilj and Sinj, and the in the Mt. Dinara area. Since 1769, the first records on local earthquakes, 20 events were recorded with epicentral intensity $I_0 > VII^+$ MCS and magnitude $M > 4.8$, which indicates intensive seismic activity. This is confirmed by the epicentral map, Fig. 3. The earthquake epicenters are presented by circles with radii proportional to magnitudes.

Based on the specific spatial, time and energy characteristics of the earthquakes, and the results of neotectonic investigations, the four significant seismic sources were determined in the analyzed regional area, Fig. 1:

1. Sinjsko polje
2. Mt. Svilaja
3. Mt. Mosor-Biočovo
4. Mt. Dinara-Southern Velebit

Sinjsko Polje Seismic Source

The Sinjsko polje seismic source has a comparatively narrow area important for its seismic activity, in which the strongest earthquake in the area occurred at 5^h20' (GMT) on July 2, 1898. The epicenter was in the vicinity of Trilj (43.6° N i 16.7° E), and its epicentral intensity was $I_0=IX^+$ MSK. The magnitude is estimated to $M=6.6$. The same epicenter is attributed another five earthquakes with magnitude $M > 4.8$.

On the other edge of the Sinjsko polje, in an epicenter in the vicinity of the town of Sinj (43.7° N, 16.7° E) the earthquake next in macroseismic intensity occurred at 00^h 59' (GMT), with magnitude $M=5.2$ and epicentral intensity $I_0=VII-VIII^+$ MSK. Another three earthquakes with magnitude $M > 4.1$ have been recorded in the same region.

Svilaja Seismic Source

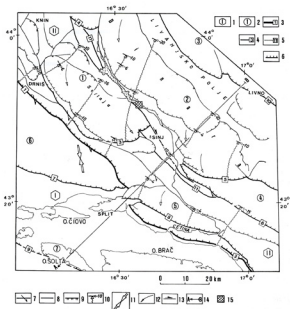
The Mt. Svilaja seismic source has two epicentral areas, i.e. the one nearby Uzdojce, between the Mt. Promina and Mt. Kozjak, and another, more active, at the Mt. Dinara, to the east of the Peruća Reservoir. The strongest earthquake with magnitude $M=5.6$ happened at the depth of 8 km, at 04^h 37' (GMT) on November 27, 1990. The epicenter was in the Mt. Dinara (43.84° N, 16.63° E), some 5 km far from the Peruća Dam site. The earthquake epicentral intensity was VII-VIII⁺ MSK. During the same day, some 14 minutes later, another even stronger earthquake happened with magnitude $M=5.5$.

Mosor-Biokovo Seismic Source

There are several sites within this area which suffered very strong earthquakes. The strongest earthquake in magnitude, $M=6.2$ was the one in the region of Herzegovina, to the west of Mostar (43.25° N, 17.30° E) at 05^h 40' (GMT) on March 15, 1923. Its depth was 16 km, and epicentral intensity $I_0=VIII-IX^+$ MCS.

The strongest earthquake in magnitude, $M=6.1$ which happened in this source on the territory of Croatia happened in the Mt. Biokovo (43.38° N, 17.02° E), at the depth of 10 km, with epicentral intensity $I_0=VIII-IX^+$ MCS, at 05^h 05^m (GMT) on January 11, 1962. With respect to the observed epicentral intensity, the strongest earthquake was IX⁺ MCS, focal depth 7 km, which happened in Imotsko polje (43.40° N, 17.20° E) at 03^h 42^m (GMT) on December 29, 1942. The earthquake magnitude was $M=6.0$.

In addition to the above, the strong earthquakes, $M > 5.0$ also happened in the vicinity of Grude and Buško Blato, and in the Mt. Kamešnica.



A. REGIONAL STRUCTURAL UNITS: 1- Adriaticum (1); Dinaricum (2). B. MAJOR OVERTHRUST STRUCTURAL UNITS: 2- Promina-Svilaja (1); Dinara-Kamešnica (2); Šator-Starešina (3); Imotski-Bekija (4); Mosor-Blokovo (5); Ravni Kotari-Kozjak (6); Čiovo-Šolta-Brač (7); C. FAULTS: 3- Faults bordering regional structural units: Obrovac-Drnish-Klis fault (1); Split-Ploče fault (2); 4- Faults bordering major overthrust structures: Muć fault (3); Dinara Mt. fault (4); Kamešnica Mt. fault (5); Triš-Tihaljina fault (6); Kozjak fault (7); Mosor Mt. and Blokovo Mt. fault (8); Šolta-Brač fault (9); Šator-Starešina fault (10); 5- The most important faults of the Vrlika-Sinj fault zone; 6- Reverse, thrust faults; 7- Sections of the reverse faults with marked horizontal displacements; 8- Sections of faults without surely defined character; 9- reverse fault sections of N, NE vergence; 10- Assumed projections of the same faults on the surface, and in the marked depth; D. STRESS, STRUCTURAL DISPLACEMENTS: 11- Regional stress; 12- Displacement of structures close to the surface; 13- Zone of horizontal displacement of structural units in the area of Sinj polje; 14- Seismotectonic profile; 15- Location of the Peruća Dam.

Figure 1. Structural map

Dinara-Southern Velebit Seismic Source

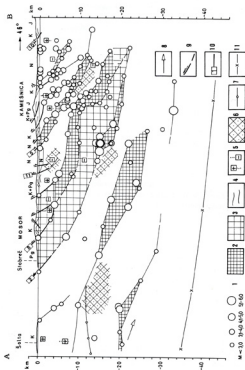
Three epicentral areas may be singled out in this source, i.e. near the town of Knin, on a stretch from Mokro polje to Razvode, and at the southern edge of the Mt. Velebit. The strongest earthquake from this source with magnitude $M=5.5$, epicentral intensity $I_p=VII-VIII^+$ MCS, focal depth 13 km nearby Knin (44.07° N, 16.32° E), happened at $13^h 59^m$ (GMT) on November 25, 1986. A large number of aftershocks followed, two with magnitude $M > 4.0$. The strongest earthquake in the epicentral area Mokro polje-Razvode happened in the vicinity of Razvode in 1972, and its magnitude was $M=4.9$. On the southern slopes of the Mt. Velebit, the strongest earthquake with magnitude $M=4.3$ was recorded on February 22, 1976.

SEISMOTECTONIC RELATIONS

The Peruća Dam is located on the marginal part of the regional structural unit of Dinaricum, in the zone of a fault separating two main overthrusts and seismogenetic structural units: Svilaja (1 in Fig. 1) and Dinara-Kamešnica (2). Deep subduction of the rock mass and overthrusting closer to the surface are important in the tectonic movements. The recent activity of the regional stress causes marked tangential displacements in the front part of the overthrust structures and diagonal displacements in space. This means that Svilaja and Dinara-Kamešnica are rotating and generally moving southward, so the horizontal displacement present along the fault zone is the most marked in the Sinjsko polje. Also, the closing and narrowing of the Vrljika-Sinj zone and Sinjsko polje is in course.

The faults from the Vrljika-Sinj (11) zone are found in the surface around the Peruća Dam. The routes of the two main faults from the zone are found in the area to the west of the dam (one route is partly covered with the Svilaja overthrust, Fig. 2). The major accompanying fault is found in the area immediately eastward from the dam. The minor faults noticed around the dam join with major dislocations in depths. The morphologically marked steep slope to the east of the dam marks the marginal Dinara-kamešnica fault (5).

The spatial concentrations of the earthquake foci may be related to the major fault zones, singled out in Fig. 2. These are the seismotectonically active zones. Within the analyzed area, the key zones are interrelated with the faults Obrovac-Drniš-Klis (1), Split-Plоче (2), Muć (3), Dinara-Kamešnica (5), Mosor-Biokovo (8) i Vrljika-Sinj (11). While determining the expected maximum earthquake magnitude M_{max} in particularly seismotectonically active zones it was possible to include, in addition to the earthquake data, the data on the most active fault section and horizontal displacement. The below was obtained:



1- Earthquake foci (marked magnitudes); 2- The seismotectonically most active zones; 3- Dinaricum overthrust zone; 4- Folds of surficial deposits; 5- Estimated positions of gravimetric maxima and minima (dotted lines indicate possible spatial displacements); 6- Zones of bigger gravimetric gradients; 7- Supposed lower boundary of sedimentary complex; 8- Direction of deep subduction of rock masses; 9- Faults with displacement marked; 10- The most important faults of the structural framework; Spil-Poše fault (2); Kamešnica Mt. fault (5); Mosor Mt. and Blokovo Mt. fault (8); Šator-Sarešina fault (10); Vtška-Saj fault zone (11); 11- Moho discontinuity.

Figure 2. Seismotectonic profile

- Obrovac-Drníš-Klis fault zone (1): the most active section length - 42 km; $M_{max}=5.7$;
- Mučki rasjed fault zone (3): the most active section length -44 km; $M_{max}=5.7$;
- Dinar-Kamešnica fault zone (5): the most active section length - 40 km; $M_{max}=5.6$;

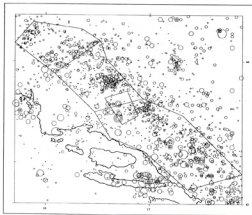


Figure 3. Epicentral and seismic sources map

- Split-Ploče fault (2) and Mosor-Biočovo fault (8) zone: the most active section length 82 km; $M_{max}=6.6$;
- Vrlika-Sinj fault zone (11): the most active section length 40 km; $M_{max}=5.6$; the Sinjsko polje horizontal displacement - 15 km; $M_{max}=6.5$;

The earthquake sources are associated with the seismotectonically active zones, Fig. 2. Based on the profiles, the spatial compliance of the widths, lengths and projections on the surface was noticed in the analyzed fault zones of Obrovac-Drnš-Klis (1) and Mučki fault (3), and zones of Split-Ploče (2) and Mosor-Biočko (8). The zones of Mučki raised fault (3) and the Obrovac-Drnš-Klis (1) fault are prominent for their higher earthquake magnitudes and cover in space the effects related to the Vrljika-Sinj (11) and Dinara-Kamešnica (5) zones. Such relations resulted in singling out of three earthquake sources in the nearest vicinity of the Peruća Dam:

1. Sinjsko polje, seismotectonically active zone related to the Vrljika-Sinj fault with possibility of including the strongest earthquake at Trilj in this zone:
- $M_{max}=6.5$;
2. Svilaja, seismotectonically active zone related to the Obrovac-Drnš-Klis and Muč
- the Peruća Dam site is within the earthquake source:
- $M_{max}=5.7$;
3. Mosor-Biočko seismotectonically active zones related to the Split-Ploče and Mosor-Biočko faults
- $M_{max}=6.6$.

Since the calculation of the seismic force parameters should respect even the most distanced epicentral areas the forth source, Dinara-Southern Velebit was defined in addition to the above three, estimated to $M_{max}=6.1$.

SEISMIC FORCE PARAMETERS

The engineering seismology investigations are used to determine the input parameters for the calculation of the seismic forces impact on the structures located on the investigated site. These are usually the maximum probable horizontal accelerations of soil during the earthquake and the highest probable macroseismic intensity of the earthquake on the site. These two parameters have also been calculated for the Peruća Dam remediation.

ATTENUATION FUNCTIONS SELECTION

The highest horizontal acceleration and possible macroseismic intensity an earthquake might cause at the investigated site are much dependent on selection of the empirical function describing their ascending with the distance from the earthquake source and dependence on the earthquake magnitude. Since there are no sufficient accelerograms for the wider Peruća Dam site which would be used for derivation of the local attenuation relation, the one which gives the best description of the available data has been selected. Based on the

analysis carried out within the 25 earthquake accelerograms, mainly for the region of the Dinarics, it has been concluded that the below attenuation relation (Prelogović et al, 1985)

$$\log a_{\max} \text{ (cm/s}^2\text{)} = 2.184 + 0.375 M - 1.468 \log (R + 18.04) \quad (1)$$

gives a good description of the empirical data and is therefore applied in calculation of the horizontal acceleration $a_{\max}$ on the Peruća Dam site, M is earthquake magnitude, R focal distance. The macroseismic intensities $I_{\max}$ on the site are calculated from the below relation (Sponheuer, 1961):

$$I_{\max} = I_0 - 3 \log (R/h) - 3 * 0.43429 W (R - h) \quad (2)$$

where I_0 - epicentral intensity, h - focal depth, and W - coefficient of absorption.

The intensity I_0 in the earthquake epicenter, if it was not known from the macroseismic data was calculated from the relation derived for the area of the Outer Dinarics (Herak, 1989):

$$I_0 = 3.63 (1 + M/3.18 - (\log h) / 1.72) \quad (3)$$

PARAMETER CALCULATION BY DETERMINISTIC METHOD

Based on the above results of the seismological and seismotectonic investigations, three earthquakes: Pot_1 , Pot_2 i Pot_3 were defined which might have the highest impact on the Peruća Dam site. The characteristics of these earthquakes are:

Pot_1 :	$M = 6.5$	$D = 7 \text{ km}$	$h = 10 \text{ km}$
Pot_2 :	$M = 6.6$	$D = 21 \text{ km}$	$h = 10 \text{ km}$
Pot_3 :	$M = 5.7$	$D = 0 \text{ km}$	$h = 10 \text{ km}$

M - magnitude, D - epicentral distance, h - earthquake focal depth.

Using the relations (1), (2) and (3) for the subject three earthquakes, the maximum values have been obtained for horizontal acceleration $a_{\max}$ i intensity $I_{\max}$:

for earthquake Pot_1 :	$a_{\max} = 0.286 \text{ g}$	$I_{\max} = 8.6^{\circ} \text{ MCS}$
for earthquake Pot_2 :	$a_{\max} = 0.197 \text{ g}$	$I_{\max} = 7.8^{\circ} \text{ MCS}$
for earthquake Pot_3 :	$a_{\max} = 0.160 \text{ g}$	$I_{\max} = 8.0^{\circ} \text{ MCS}$

The maximum values obtained for the first of the three above earthquakes, namely for the earthquake with magnitude 6.5 Richter scale, with the focal depth of 10 km, and epicentral depth of 7 km from the investigated site. Since the dam is the structure founded on a hard rock, no amplification effect of the surface layers is expected, and the dynamic amplification factor (DAF) equals unity, and there is no calculated maximum macroseismic intensity increment. This is clear from the small amplitudes of the microseismic noises recorded during the measurements carried out in October 1993. It should be mentioned that, due to the war activities, the measurements could not have been carried out on the dam site, but somewhat more eastward. However, the measurements have been carried out on the sites which, according to the geological data, are fully equivalent to the dam site. Based on these data, the prior conclusion related to the microseismic noise level on the Peruća Dam site has been brought.

Using the above relations and the results of the local seismological investigations, the maximum values have been determined for a_{max} and I_{max} at the Peruća Dam site for the historical earthquakes. The obtained values are:

- For earthquake of November 27, 1990, $M=5.6$, $h=8$ km, $D=5.2$ km, $I_0=8.12^\circ$ MCS:

$$a_{max} = 0.15 \text{ g} \quad I_{max} = 7.86^\circ \text{ MCS}$$

- For earthquake of July 2, 1898, $M=6.5$, $h=10$ km, $D=23.3$ km, $I_0=IX^\circ$ MCS:

$$a_{max} = 0.16 \text{ g} \quad I_{max} = 7.07^\circ \text{ MCS}$$

PROBABILISTIC APPROACH

The intensity of 8.6° MCS and acceleration of 0.29 g defined in the previous section are absolute maximums, regardless of the time period in the future which is considered. However, in addition to values defined in this way, the engineering practice also uses the values which are the functions of time period and probable exceedings of these values in concordance with the seismic risk concept.

The probabilistic seismic parameter determination for the Peruća Dam was based on the generally accepted methodology of determining the seismic hazard. Two methods have been used, i.e. Cornell's (Cornell, 1968), applying the McGuire's algorithm (McGuire, 1976), and the theory of extremes using the Jenkin's method (Kuk, 1982).

Maximum Acceleration Calculation by McGuire's Method

The maximum acceleration were calculated by the McGuire's method, using the EQRISK computer program. The four above specified seismic sources were taken into consideration,

along with the so called background seismicity for the 100 km radius circular area. For each source, the a and b coefficients of the Gutenberg-Richter relation were calculated, while the seismotectonically defined values were used for the upper limit magnitudes. The earthquake focal depths for particular sources were taken as constant.

The calculated values of the maximum horizontal accelerations a_{max} , related to the individual return periods T , are presented in the below table:

T (yr)	50	100	200	300	500	1000
a_{max} (g)	0.111	0.136	0.165	0.182	0.185	0.186

Table 1 Maximum horizontal accelerations a_{max} as a function of return period T

The acceleration 0.186 for the 1,000-year return period is the asymptotic value of the maximum acceleration obtained by the above procedure.

Maximum Acceleration Calculation Applying the Theory of Extremes

The calculation applying the theory of extremes has been conducted on the basis of the earthquake data for the period 1898-1993, and in two ways - using the so called equivalent focal distances, Case A and actual focal distances, Case B. The procedure described in (Prelogović, 1985) has been applied. The obtained values are presented in Table 2 where s designates standard deviation.

T (yr)	a_{max} (g)		$a_{max}-s$		$a_{max}+s$	
	A	B	A	B	A	B
50	0.117	0.108	0.101	0.090	0.135	0.128
100	0.141	0.134	0.120	0.109	0.166	0.163
200	0.164	0.159	0.137	0.127	0.196	0.200
300	0.176	0.174	0.148	0.145	0.219	0.216
500	0.190	0.192	0.155	0.147	0.233	0.251
1000	0.207	0.215	0.166	0.159	0.259	0.289
10000	0.247	0.274	0.188	0.188	0.323	0.401

Table 2 Maximum horizontal acceleration a_{max} as function of return period T

If these values are compared with the ones from Table 1, it is noticed that the applied theory of extremes gives much higher values than the McGuire's method, particularly when the return periods are higher. Further, the comparison of the acceleration value of

0.29 g (defined on the basis of the seismotectonic data) with the acceleration value of 0.274 g (which corresponds to the return period $T = 10,000$ yrs from the above Table) indicates good agreement although the completely different calculation methods were applied.

The maximum accelerations, as the function of the period of exploitation and the risk level, are presented in Table 3. Only the values corresponding to the Case B, actual focal distances are given, since they are somewhat higher than the analog values in the Case A.

E (yr)	R (%)	a_{max} (g)	$a_{max} \pm s$	a_{max} -s
100	10	0.213	0.159	0.286
100	20	0.188	0.145	0.245
100	30	0.172	0.135	0.219
200	10	0.234	0.169	0.322
200	20	0.211	0.158	0.283
200	30	0.196	0.149	0.257

Table 3 Maximum acceleration a_{max} as function of the exploitation period E and risk R

Maximum macroseismic intensity calculation

The maximum earthquake intensities are calculated by application of the theory of extremes in two ways, A and B, same as in the maximum acceleration calculation. The annual extremes sample has also been used for the period 1899-1993. The calculated values of intensity I corresponding to the individual return periods T are given in Table 4.

Comparison of the intensity value 8.6° MCS, determined on the basis of the seismotectonic data, with values 8.5 namely 8.6° MCS, which according to the above table correspond to the 10,000-year return period, indicate good agreement of the results.

T (yr)	I_{max} (° MCS)		$I_{max}-s$		$I_{max}+s$	
	A	B	A	B	A	B
50	7.13	6.99	6.88	6.70	7.39	7.29
100	7.46	7.36	7.17	7.02	7.74	7.70
200	7.71	7.66	7.39	7.27	8.03	8.05
300	7.84	7.81	7.44	7.40	8.18	8.22
500	7.98	7.98	7.61	7.52	8.34	8.44
1000	8.13	8.17	7.73	7.66	8.53	8.69
10000	8.45	8.60	7.95	7.94	8.95	9.26

Table 4 Maximum intensity I_{max} as function of return period T

The maximum intensities as the function of the period of exploitation and the risk level are given in Table 5. Only the values for the case B are given, calculation with actual focal distances, since the ones for the greater return periods are somewhat higher from the analog values for the Case A.

E (yr)	R(%)	I_{max} (° MCS)	$I_{max}-s$	$I_{max}+s$
100	10	8.16	7.65	8.67
100	20	7.94	7.49	8.40
100	30	7.79	7.37	8.21
200	10	8.32	7.76	8.88
200	20	8.14	7.64	8.65
200	30	8.01	7.54	8.48

Table 5. Maximum intensities I_{max} as function of exploitation period E and risk R

DESIGN SEISMIC PARAMETERS

As already said, the deterministic method was used to define the values which are absolute maximums and are not expected to be exceeded in any period of time. In this way, the horizontal acceleration was obtained of

$$a_{max} = 0.29 \text{ g,}$$

for macroseismic intensity

$$I_{max} = 8.6^{\circ} \text{ MCS.}$$

The probabilistic approach has been used to calculate the horizontal acceleration and earthquake intensity as the function of the return period, risk level and period of exploitation. Assuming the earthquakes of type S_1 i S_2 are defined by the 200- and 1000-year return periods, the below horizontal accelerations and intensities are obtained:

earthquake S_1	$I_{max} = 7.7^{\circ}$ MCS
	$a_{max} = 0.17$ g
earthquake S_2	$I_{max} = 8.2^{\circ}$ MCS
	$a_{max} = 0.22$ g

Since the structure is founded on a hard rock, the amplification effect of the top layers is not expected, the dynamic amplification factor (DEF) equals unity and the calculated maximum intensity has no increment. Therefore, the said parameters are also the final design values of the horizontal accelerations and macroseismic intensities.

CONCLUSION

The Peruća Dam site is located in the area of high seismic activity, both locally and regionally. Among the historical earthquakes, the most important are those from the below seismic sources:

- Sinjsko polje, the strongest earthquake: July 2, 1893, epicentral intensity IX° MCS, distance from the Peruća site 23 km and
- Svrljaja, the most important earthquake: November 11, 1990, epicentral intensity VII- VIII° MSK, magnitude 5.6, distance from the Peruća site 5 km.

At the Peruća site, the above two earthquakes had the horizontal accelerations of 0.16 and 0.15 g, and intensities of 7.1 i 7.9° MCS.

The Peruća Dam is located on the border of the regional structural unit of Dinaricum, in the fault zone which divides two overthrust and seismogenic structural units: Svrljaja (1) and Dinara-Kamešnica (2). For tectonic movements the deep rock mass subduction and superficial overthrusts are important. The recent activity of the regional stress causes pronounced tangential displacements in the frontal part of the overthrust structures and diagonal in the area with marked horizontal component along the favorably oriented faults. The Vrlika-Sinj zone faults are found on the surface around the Peruća Dam. In the encompassed area all the key zones are connected with the faults: Obrovac-Drniš-Klis, Split-Ploče, Muć, Dinara-Kamešnica, Mosor-Biokovo and Vrlika-Sinj.

The earthquake sources are associated with the seismotectonically active zones by singling out of four earthquake sources - Sinjsko polje, Svilaja, Mosor-Biočko and Dinara-Southern Velebit.

The engineering seismology investigations included calculation of maximum horizontal accelerations and macroseismic intensities. The calculation was conducted in two ways: deterministic approach on the basis of the seismotectonic investigation results, and the probability complying with the seismic risk concept.

The values obtained by the deterministic method are the absolute maximums which are not expected to be exceeded at any period of time. The horizontal acceleration obtained in this way

$$a_{max} = 0.29 \text{ g,}$$

and macroseismic intensity

$$I_{max} = 8.6^{\circ}.$$

The probabilistic approach has been used to calculate the horizontal acceleration and earthquake intensity as the function of the return period, risk level and period of exploitation. Assuming the earthquakes of type S_1 i S_2 are defined by the 200- and 1000-year return periods, the below horizontal accelerations and macroseismic intensities are obtained:

$$\text{earthquake } S_1 \quad I_{max} = 7.7^{\circ} \text{ MCS} \quad a_{max} = 0.17 \text{ g}$$

$$\text{earthquake } S_2 \quad I_{max} = 8.2^{\circ} \text{ MCS} \quad a_{max} = 0.22 \text{ g}$$

Since the dam is founded on a hard rock, the amplification effect of the top layers is not expected, the dynamic amplification factor (DEF) equals unity, and the calculated maximum intensity has no increment. Therefore, the said parameters are also the final design values of the horizontal accelerations and macroseismic intensities.

REFERENCES

- [1] Aljinović, B., Prelogović, E. & Skoko, D. (1990): Tectonic processes on the contact of the Adriatic Platform and the Dinarides in the area of the Northern Dalmacija. Proc. of the Inter. Conf. on Mechanics of jointed and faulted rock, Inst. of Mech/Tech. Univ. of Vienna, 179-182, A.A. Balkema, Rotterdam/Brookfield
- [2] Anderson, H. & Jackson, J., (1987): Active tectonics of the Adriatic region. Geophy. J.R.A.S., 91,937-983

- [3] Cornell, C.A. (1968): Engineering Seismic Risk Analysis. BSSA, Vol. 58
- [4] Gumbel, E.J., (1958): Statistics of Extremes. Columb. Univ. Press, New York
- [5] Herak, D., Herak, M., Sović, I. i S. Markušić (1991): Seismicity of Croatia in 1989 and the Kamešnica Mt. earthquake, Geofizika, Vol. 8, 83-99.
- [6] Herak M.J.(1986): A new concept of geotectonics of the Dinarides. Acta geol., YAZU, 16, 1-42, Zagreb
- [7] Herak, M. (1989): The magnitude-intensity-focal depth relation for the earthquakes in the wider Dinara region, Geofizika, 6, 13-21.
- [8] Herak, M., (1991): Dinarides, Mobilistic Approach on Genesis and Structure. Acta geol. 21/2, Priroda, 63, HAZU, 35-117, Zagreb
- [9] Jenkinson, A.F. (1969): Statistics of Extremes in Estimation of Maximum Floods. Techn. note no. 98, WMO, WMO-No. 223, T.P.126 Geneva
- [10] Kuk, V. (1982): On Maximum Earthquake Intensity. Master's Thesis, College of Mathematics, Zagreb
- [11] Kuk, V., Prelogović, E., Marić, K., Skoko, D., Allegretti I., Herak, M., Herak, D., Milošević, A., Ivančić, I., Sović, I., Jamičić, D., Aljinović, B., Markušić, S., Markovac, Z. (1994): Peruća Dam - Seismic Parameters Study. College of Mathematics, Zagreb
- [12] Magdalenić, A. (1971): The Cetina Catchment Hydrology. Yugoslav Karst; YAZU, 74, Zagreb
- [13] Makjanić, B. (1977): Applying the Theory of Extremes in Geophysics. Hidrometeorološki zavod RH, Zagreb
- [14] McGuire R. K. (1976): Seismic risk analyses, USGS, Open File Rep.
- [15] Milošević, A., Prelogović, E. & Herak, D. (1984): The Dinara Region Sismicity. Geol. vjesnik, 37, 205-215, Zagreb
- [16] Mišuš, P. (1973): Geologic - Tectonic structure and evolution of outer Dinarides and Adriatic area. B.A.A.P.G., 57/5, 913-929, Tulsa

- [17] Prelogović, E., Bahun, S. & Aljinović, B. (1990): Neotectonic relations in the evaporitic region among Bihać, Krin and Sinj. ARH. Inst. geol. istraž., Zagreb
- [18] Prelogović, E. & Kranjec, V. (1983): Geological development of the Adriatic area. Pomorski zbornik, 21, 387-405, Rijeka
- [19] Prelogović E., D. Skoko, V. Kuk, K. Marić, A. Milošević, M. Živčić, M. Herak, I. Allegretti, I. Sović (1985): Determining S1 i S2 earthquake characteristics on the Slavonija Nuclear Power plant Site, RGN i PMF, Zagreb
- [20] Skoko, D., Prelogović, E. & Aljinović, B. (1987): Geological structure of the Earth's crust above the Moho discontinuity in Yugoslavia. Geophy. J.R.A.S., 89, 379-382
- [21] Skoko, D. & Prelogović, E. (1988): Seismic potential of Yugoslavia territory. Proc. of the Ninth World Conf. on Earthquake Engineering, August 2-9, 1988, Vol. II, 163-168, Tokyo-Kyoto
- [22] Skoko, D. & Prelogović, E. (1989): Geological and seismic data required for maximum earthquake magnitude determination. geol. vjesnik, 42, 287-289, Zagreb
- [23] Skoko, D., V. Kuk, K. Marić, I. Allegretti, S. Markušić (1993): Instrument monitoring of seismic activity at the Peruća HPP site, Part I and II, Geofizički zavod PMF-a, Sveučilište u Zagrebu (manuscript)
- [24] Sponheuer W. (1960): Methoden zur Herdtiefenbestimmung in der Makroseismik, Freiburger Forschungshefte C 88, Akademie Verlag, Berlin.

THE PERUĆA DAM FLOODS ANALYSIS

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ABSTRACT

The Cetina River floods at the Peruća Dam profile have been defined by maximum discharges and hydrographs of various return periods. The main hydrologic features of flood runoffs from the karst catchment of the Cetina River are described. On the basis of data acquired by direct monitoring in the period 1961-1986, maximum discharges have been analyzed for wet season, October - May and dry season, June - September during year as well as the shape and volume of hydrographs. Isolated hydrographs in wet and dry season have been considered, and complex hydrographs (double) were analyzed separately for wet seasons. It has been proved that competent hydrographs of floods of various return periods could be devised in triangular shape.

INTRODUCTION

The Cetina river floods at the Peruća Dam profile are determined because they are required for sizing of the Peruća Dam discharge structures and the Peruća reservoir operation. Therefore, the maximum discharges should be specified and the relevant hydrographs prepared for flood waves of various return periods.

The available hydrological data on floods obtained by direct monitoring were analyzed. Special attention was paid to specific characteristics of the water runoff in karst regions, the Cetina catchment area being one of them.

MAIN HYDROLOGICAL CHARACTERISTICS

The Dinaric karst, which includes the Cetina catchment, determines its essential geomorphological forms and markedly orographic characteristics. The distance of the Cetina catchment from the sea is a natural barrier for the wet maritime streams with abundant atmospheric humidity, which makes conditions for condensation and formation of abundant rainfalls favorable [4].

Karst is characterized by a sparse hydrographic network, so about two thirds of permanent Cetina inflow is the result of continual groundwater inflow, mainly from the SW Bosnian karst [1].

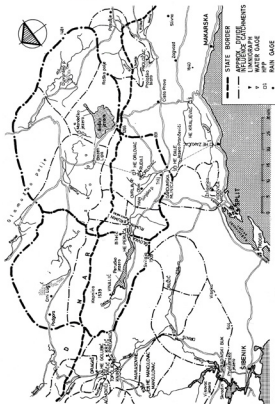


Figure 1. The Cetina catchment to the Gardunska Minica profile

The daily water inflows into the Peruća reservoir during the period 1961-1986, obtained from the Peruća HPP services, were available for the runoff analysis. The characteristic Cetina discharges at the Peruća Dam profile are as follows:

- minimum measured discharge: $Q_{\min} = 4 \text{ m}^3/\text{s}$
- mean discharge: $Q_x = 42.2 \text{ m}^3/\text{s}$
- maximum measured discharge: $Q_{\max} = 298 \text{ m}^3/\text{s}$

According to these data, the Cetina runoff at the Peruća Dam profile; for the period 1961-1986 was:

$$Q_{\min} : Q_x : Q_{\max} = 1 : 10 : 70 \quad (1)$$

Related to the water runoff variability in the Dinaric karst which is, according to [2], orientationally:

$$Q_{\min} : Q_x : Q_{\max} = 1 : 10 : 100 \quad (2)$$

the maximum Cetina discharge compared to the mean and minimum discharge is below the average orientational ratio from the (1) above.

The influence Cetina catchment area to the Gardunska Mlinica profile is shown in Fig. 1. The Cetina catchment area to the Peruća Dam profile towards the water divide is approximately $A = 1,380 \text{ km}^2$ [2]. The mean annual gross rainfall for the Cetina catchment to the Peruća dam is $P = 1,370 \text{ mm}$. The average annual effective rainfall is $P_e = 960 \text{ mm}$, so the average annual runoff coefficient is $c = 0.70$. The variations of mean annual discharge of the Cetina in the dam profile are generally within the range of values for karst, the coefficient of variation is $c_v = 0.24$. The distribution of the mean annual discharge of the Cetina in the dam profile shows slight negative skewness, the coefficient of skewness is $c_s = -0.21$.

The maximum recorded specific inflows from the Cetina catchment and its main tributaries, respective of the catchment area, are shown in the Fig. 2. It also shows the characteristic curves of the maximum recorded specific inflows with respect to the Croatian karst catchment areas [6]:

Upper Envelope:

$$q_{\max} = 25.89A^{0.514} \quad (3)$$

and the average relation in which, due to high dissipation A , and $q_{\max}$ the coefficient of correlation is very low, $r = -0.68$, so it should be used for orientation purposes:

$$q_{\max} = 5.76A^{0.446} \quad (4)$$

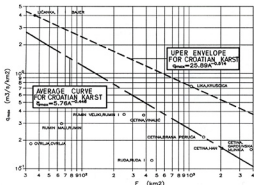


Figure 2. — Maximum recorded specific inflows from the Cetina catchment q_{max} , related to the catchment areas size, A , and the average curve and the upper envelope for the Croatian karst

The maximum recorded specific inflows from the Cetina catchment and its main tributaries are strongly affected by the reducing effect of the karst underground, so they are below the average curves for the Croatian karst [4]. The only exception is the furthest analyzed downstream profile Gardunska Mlinica on the Cetina to which the water runoff from the direct catchment is higher than in the upstream part of the Cetina catchment and its left-bank tributaries.

The mentioned characteristics, a comparatively low ratio of minimum, mean and maximum discharge, 1:10:70, high mean water runoff coefficient $c = 0.70$, and below average values of the maximum recorded discharges as compared to the Croatian karst catchments, document the strong effect of the karst underground on the Cetina water regime.

MAXIMUM DISCHARGES

The Dinaric karst rivers are typical for very pronounced dry and wet season during the year. The uninterrupted homogenous 26-year series of unrelated data for the wet period

October-May and dry period June-November were available for calculation of maximum discharges of various return periods. The maximum Cetina discharge series for the wet and dry season within the period 1961-1986 are graphically presented in Fig. 3. It should be noted that the maximum wet season discharges are also the maximum annual discharges for the period 1961-1986.

The graphical presentation in Fig. 3 clearly shows without any special analysis that the series are homogenous.

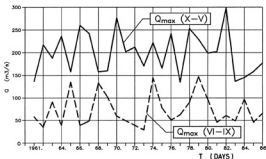


Figure 3. The maximum Cetina discharges at the Peruća dam profile; wet and dry season in the period 1961-1986

The analysis of daily inflows into the Peruća Reservoir has shown that there is no maximum discharge which would be preceded by another maximum discharge in a series. Therefore, the analyzed maximum discharge series should be considered independent of each other [7].

The analyzed period of 26 years, which is the number of data in each series, is a comparatively short period for statistical analysis. Therefore, it was examined whether the analyzed series are sufficiently long. The Kricky-Menkel equation was used for determination of the coefficient of variation error, σ_{cv} :

$$\sigma_{cv} = \pm \frac{c_v}{\sqrt{2(n-1)}} \sqrt{1+3c_v^2} \quad (5)$$

where:

- c_v - coefficient of variation for the analyzed discharge series
- n - number of series members

The criterion of D. Srebrnović that the hydrological series is not sufficiently long for the statistical analysis if $\sigma_{\alpha} = 0.10$ [3] was accepted.

The coefficient of variation for the wet period maximum discharge series is $c_v=0.23$, for the dry period maximum discharge series $c_v=0.49$. The corresponding coefficient of variation errors are: $\sigma_{c_v} = 0.035$ for the wet season discharge series, and $\sigma_{c_v} = 0.091$ for the dry season discharge series. Therefore, both considered series are sufficiently long for application of the mathematical statistics methods.

The various return period maximum Cetina discharges at the Peruća Dam profile for the wet and dry season have been calculated from the data presented in Fig. 3, on the basis of various distributions. The most suitable distribution for both series was the Gumbel distribution, columns 2 and 4, Table 1. The relevant maximum Cetina discharge distribution curves and the input data are graphically presented in Fig. 4.

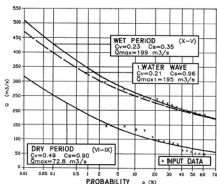


Figure 4. The various return period maximum Cetina discharges during the wet and dry season at the Peruća Dam profile

Return period (yrs)	Isolated flood wave				Double flood waves						
	Wet period October-May		Dry period June-November		Wet period October-May				Total volume $V_1 + V_2$ ($10^6 m^3$)	Difference -100	
					1 st flood wave		2 nd flood wave			$Q_{max} - Q_{max}$ ----- Q_{max} (%)	$(V_1 + V_2) - V$ $V_1 + V_2$ (%)
	Q_{max} (m^3/s)	V ($10^6 m^3$)	Q'_{max} (m^3/s)	V' ($10^6 m^3$)	Q_{max} (m^3/s)	V_1 ($10^6 m^3$)	Q_{max} (m^3/s)	V_2 ($10^6 m^3$)			
1	2	3	4	5	6	7	8	9	10	11	12
10,000	510	528	314	203	471	437	421	398	835	7.6	37
1,000	427	443	249	161	397	373	375	359	732	7.0	39
100	344	357	185	120	324	309	322	312	621	5.8	43
50	319	331	166	107	301	290	304	297	587	5.6	44
25	294	304	146	94.5	279	270	284	280	550	5.1	45
10	259	269	120	77.4	248	244	256	255	499	4.2	46
5	232	241	98.6	63.9	224	223	230	233	456	3.4	47
2	192	199	67.1	43.4	187	191	185	194	385	2.6	48
Distribution	Gumbel		Gumbel		Gumbel		log-Pearson 3				
Average	199		72.8		195		189				
c_s	0.23		0.49		0.21		0.26				
c_v	0.35		0.90		0.96		0.93				

Notations

- Q_{max} - max. discharge in wet season, max. annual discharge V - volume of an isolated flood wave in wet season
 Q'_{max} - max. discharge in dry season V' - volume of an isolated flood wave in dry season
 Q_{max} - max. discharge of the 1st flood wave in double flood wave V_1 - volume of the 1st flood wave in double flood wave
 Q_{max2} - max. discharge of the 2nd flood wave in double flood wave V_2 - volume of the 2nd flood wave in double flood wave

Table 1 The characteristic Cetina floods at the Peruća Dam profile

FLOOD WAVES

Isolated Flood Waves

The analysis of the recorded hydrographs of the Cetina flood waves has been carried out on the basis of the daily inflows in the Peruća reservoir in the period 1961-1986 [7]. With respect to long duration of flood waves and slow changes in inflow quantity, one day was accepted as a realistic time increment.

For the wet period of year, October-May, the average (set-up) time $T_p=8$ days, and the average flood wave duration, hydrograph basis $T=24$ days were determined. For the dry season, June-November, the average (set-up) time $T_p=5$ days, and the average flood wave duration $T=15$ days were determined.

The hydrograph shape for the Cetina isolated flood waves at the Peruća Dam profile during the wet and dry season can be approximated in a satisfactory manner by a triangle.

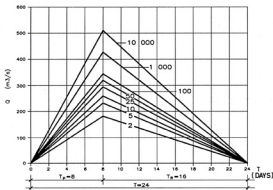


Figure 5. The various return period isolated Cetina flood waves for the wet period, October-May

Based on the approximate triangular shape, the average time of (wave set-up) and duration, and maximum discharges of various return periods, the hydrographs were obtained for isolated flood waves of various return period, for the wet season presented in Fig. 5 and for the dry season in Fig. 6.

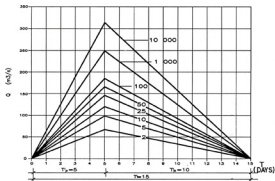


Figure 6. The various return period isolated Cetina flood waves for the dry period, June-November

The volumes of the isolated flood waves of various return period are given in Table 1: in column 3 for the wet season and in column 5 for the dry season.

Double flood waves

Due to large volume of its cavities, the karst underground in the hinterland of the Peruća reservoir has a reducing effect on the Cetina flood waves. This reflects in long wave (set-up) and recession times. For that reason, the flood wave after the intensive rainfall regularly lasts for more than 15 days.

The intensive rainfalls in the Cetina catchment are frequent during the wet season, when the hydrograph is in recession. In such case, the second flood wave comes immediately after the first, so they could be analyzed together as a double flood wave. The double waves have two maximum, peak discharges, usually within the interval of 15 to 30 days.

From the hydrographs of the mean daily Cetina discharges at the Peruća Dam profile during the period 1961-1986 the frequent occurrence of subsequent, namely double flood waves was determined [8]. Eighteen double flood waves were analyzed for the given period 1961-1986. The first and second flood wave were isolated along with their main elements - maximum and basic discharges, (set-up) times, hydrograph bases and volumes. The hydrograph shape has been analyzed separately. Two characteristic hydrographs of the double Cetina flood waves at the Peruća Dam profile are presented in Figs. 7 and 8.

All double flood waves which are analyzed because of their size and duration fall into the wet season, October-May.

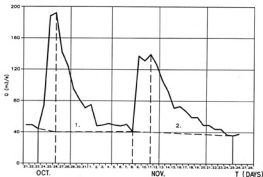
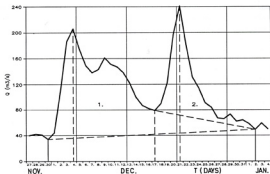


Figure 7. The double Cetina flood wave at the Peruća Dam profile
Oct. 23, 1964 - Nov. 25, 1964



**Figure 8. The double Cetina flood wave at the Peruća Dam profile
Nov. 30, 1976 - Jan. 2, 1977**

The double Cetina flood waves at the Peruća Dam profile for the period 1961-1986 are independent events, because they are separated by sufficiently long periods with low water inflows. Therefore, the inflows of the prior waves have actually no effect on the subsequent double flood wave inflows. This applies when the double waves are considered as the isolated entities. But each first flood wave within a double wave has certain effect on the second wave which follows it immediately. This characteristic is not a problem, because the first and the second flood wave within a double wave are analyzed separately.

When individual elements of the double Cetina flood waves at the Peruća Dam profile are analyzed, the basic and direct inflows are first separated for the first and the second flood waves. The average basic inflow of the first flood wave is $Q_{b1}=35.0 \text{ m}^3/\text{s}$, and of the second flood wave $Q_{b2}=40 \text{ m}^3/\text{s}$.

The series of maximum, peak discharges of the first and the second flood wave are homogenous and sufficiently long for statistical analyses: $n=18$; $c_1=0.21$; $\alpha_n=0.038$ for the first, and $n=18$; $c_2=0.26$; $\alpha_n=0.04$ for the second flood wave. Based on these data, the maximum discharges of various return periods as per different distributions were calculated

for both flood waves, within the double flood wave. As the most suitable, the Gumbel and the log-Pearson 3 distributions were selected for the maximum discharges of the first and the second flood wave, respectively. The maximum discharges of the various return periods are given in the column 6, Table 1 for the first flood wave, and in the column 8, Table 1, for the second flood wave.

The column 11, Table 1, shows different maximum discharges of various return periods for the isolated flood waves of the wet period from the column 2, and for the first flood wave, of the double wave, from the wet period from the column 6. The differences are small and amount to 7.6 percent for a 10,000-year return period. This is shown in Fig. 4, which contains the maximum discharge distribution curve for the wet and dry season along with the maximum discharge distribution curve for the first flood wave of the double wave.

The average set-up time for each wave is five days, and the average duration of each wave is 20 days, so that the average double flood wave duration totals 40 days.

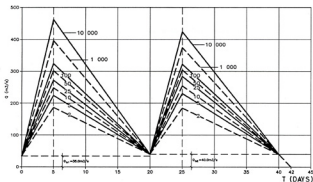


Figure 9. The various return period double Cetina flood waves at the Peruća Dam profile

Based on the analysis of the hydrographs for the recorded double Cetina flood waves at the Peruća Dam profile it has been concluded that their shape can well be approximated by a triangle. Using the accepted triangular shape, and based on the average basic discharges, the maximum discharges of various return periods and the average times of wave (set-up) and duration, the hydrographs were defined for the double flood waves of various return period for the Cetina at the Peruća Dam profile, as presented in the Fig. 9.

The volumes of the double flood waves of various return period are given in Table 1, in column 7 for the first flood wave, in column 9 for the second flood wave, and in column 10 for the total double flood waves, sum of the first and the second wave volumes.

The difference in volume of double and isolated flood wave is rather high and amounts from 37 % at 10,000-year return period waves, to 48% for the 2-year flood waves, column 12, Table 1. Therefore, the double flood waves should be accounted for during the wet season at the Peruća reservoir.

The Table 2. gives a comparison of the volumes of the Cetina double waves of various return periods at the Peruća Dam profile determined from the triangular hydrograph, values $V_1 + V_2$ in column 2, with the corresponding volumes of the double waves determined on the basis of the Pearson 3 distribution which is best suited to the calculation data, value V' in column 3.

Return period (yrs)	$V_1 + V_2$ (10 ⁶ m ³)	V' (10 ⁶ m ³)	$\frac{(V_1 + V_2) - V'}{V'}$ · 100 (%)
1	2	3	4
10 000	835	871	-4.1
1 000	732	736	-0.5
100	621	595	4.4
50	587	550	6.7
10	499	436	14
Distribution		Pearson 3	
Average		306	
c_1		0.32	
c_2		0.88	

Table 2 Comparison of the volumes of the various return period double Cetina flood waves at the Peruća Dam profile; triangular shape $V_1 + V_2$ and as per the Pearson 3 distribution V'

The difference between the volumes of the triangularly shaped double flood waves and the volumes determined as per the Pearson 3 distribution, column 4, table 2 for the flood waves of the 50 and 10,000-year return period are very small and do not exceed seven percent. Therefore, the triangularly shaped hydrograph is acceptable for the various return period double flood waves of the Cetina at the Peruća Dam profile.

CONCLUSION

The conducted analysis of the Cetina floods at the Peruća Dam profile have shown that the maximum discharges can be considered for the wet season, October-May, these also being the maximum annual discharges, and for the dry season, June-November. The maximum inflows from the Cetina catchment are strongly affected by the karst underground, therefore their value as compared to the other karst catchments is rather low, Fig. 2. The various return period maximum Cetina discharges at the Peruća Dam profile during the wet season are considerably higher than corresponding discharges during the dry season, Fig. 4.

The hydrographs of the isolated flood waves during the wet season are specially abundant in water as compared to the isolated flood waves during the dry season. The volumes of the flood waves during the dry season are in average 2.5 times higher than the volumes of the flood waves in the dry season.

In addition to the isolated floods waves the double Cetina flood waves often occur, practically always during the wet season, which are much higher than the isolated flood waves in the wet season by 37 to 48 percent, column 12, Table 1. The double Cetina flood waves hydrographs were obtained by their analysis for the Peruća Dam profile and various return periods.

The above analyses lead to the following conclusions:

1. The results of the maximum wet season discharge analysis, which are also the maximum annual discharges, namely the maximum 10,000-year discharge $Q_{max} = 510 \text{ m}^3/\text{s}$, column 2, Table 1 should be used in sizing the Peruća Dam discharge structures.
2. The below should be used in the reservoir operation:
 - during the wet season, October-May, the hydrographs of the double flood waves of various return periods, Fig. 9,
 - during the dry season, June-November, the hydrographs of the isolated flood waves of various return periods, Fig. 6.

REFERENCES

- [1] Baulčić I.: The Cetina River - Development of Relief and Water Circulation in karst, Institute of Geography, University of Zagreb, 1967
- [2] Herak M. *et al*: Karst Hydrology in Eight Countries of the of the Mediterranean Perimeter, Civil Engineering College of the University of Sarajevo, pp. 11-30, 1975
- [3] Sečhrenović D.: Flood Problems, Tehnička knjiga, Zagreb, 1971
- [4] Šepinac A.: Dinaric Karst Runoff, The Yugoslav Karst, Volume 6, JAZU, Zagreb, pp. 207-235, 1969
- [5] Žugaj R.: Average Discharges in the Croatian Karst, Hrvatske Vode Journal, pp. 521-534, 2 (1994) 8
- [6] Žugaj R.: Extreme Discharges in the Croatian Karst, Građevinar Journal, pp 749-763, 46 (1994) 12

Other sources:

- [7] The Peruća Dam, The Peruća Dam Remediation, The Cetina Hydrology - the Floods at the Peruća Dam Profile, Elektroprojekt Consulting Engineers Ltd., Zagreb, 1994
- [8] The Peruća Dam, The Peruća Dam Remediation, The Cetina Hydrology - Analysis of the Subsequent Flood waves at the Peruća Dam Profile, Elektroprojekt Consulting Engineers Ltd., Zagreb, 1995

GEOTECHNICAL INVESTIGATIONS AND DESIGN PARAMETERS FOR STABILITY ANALYSES OF THE PERUĆA DAM

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ABSTRACT

The concept, the extent and the results of complex geotechnical field and laboratory investigations in clay dam core are presented. The investigations were carried out a year after the damage.

The selection of design parameters included data collected during dam construction and control investigations performed after twenty years of dam exploitation.

The comparison of recent measurements and the selected range of parameters for clay core is also presented. It is demonstrated that the design values are adequately estimated.

INTRODUCTION

After the Peruća Dam explosion on January 29, 1993, and the provisional remediation of the dam damage and evacuation of water from the reservoir, the dam remediation design works were initiated. Comprehensive geotechnical and geophysical investigation works were planned in order to determine the condition of the clay core, gallery and the bedrock which are the key factors of dam functionality. The initial program planned the investigation and prospecting on the ends and in the central dam section. The data obtained by the visual inspection and the indices obtained immediately after the attempted dam destruction confirmed the solution based on excavation and new construction of the dam ends, so the initial investigations program was reduced to the central dam section.

The immediate threat of war operations prevented performance of investigations on the dam crest to be conducted in 1993. During that period, the gallery excavation and remediation was started, and the design documentation for the remediation of the entire

dam completed. The design documentation planned excavation and new construction of the ends, removal and reconstruction of the top 5-10 m of the central dam section, and construction of the impermeable cut-off wall on the dam core upstream end.

When the ceasefire was agreed on in April 1994, the excavation of ends and the top dam sections started. The investigation works were conducted from the elevation 353.0 m a.s.l., 10 m below the earlier dam top, in May and June 1994.

PURPOSE AND SCOPE OF THE GEOTECHNICAL INVESTIGATIONS

The purpose of the geotechnical investigations was:

- (a) To get the insight into the condition of the core in the drilling and sounding sites, in order to determine the possible major damage which might endanger the cut-off wall construction;
- (b) To obtain the core material tests results which would enable checking of the design clay parameters from the dam static and seismic stability analyses.

The investigations were conducted by a combination of the field methods, borehole with core drilling, standard and static penetration test, SPT, borehole permeability tests, geophysical methods, while the laboratory tests included a wide range of classification tests and the tests for determination of the soil mechanical parameters.

Layout of investigation points in the proximity of the future cut-off wall, between the stations 1+35 and 1+85, is such that a continuous testing along the core depth is conducted in 5 m intervals. The sounding boreholes were arranged in alternation with the SPT, and the data along the core depth obtained. The boreholes are extended into the rock underlying the dam, so the bedrock data exist for 10 m intervals.

The rough grid interconnecting within the core was realized by the seismic cross-hole method, also in 5 m intervals along the core depth. It has been estimated that this investigation "grid" can record larger areas of core damage or any major cracks or caverns.

The conducted investigations layout is presented in Fig. 1.

DESCRIPTION OF THE PERFORMED INVESTIGATIONS

Investigation boreholes. The investigation drilling and the tests on the boreholes were conducted by the Grouting and Special Toppings Department, Geotecnika, Zagreb.

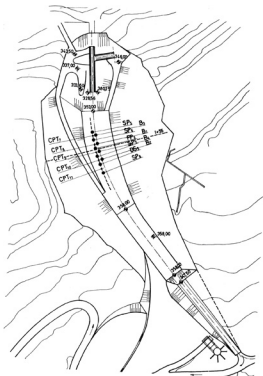


Fig. 1. The layout of geotechnical boreholes

The boreholes were drilled in May and June 1994, using the motor drills. The clay core was bored with single core drilling machine (146-101 mm), no drilling mud, in 1 m steps. The drilling in the rock was carried out with the core drilling machines T6S, with drilling mud.

While the boreholes were drilled and the impermeability tests conducted by the Legeranc method, the boreholes closed after a longer delay because of the clay swelling, so this method was not applied in permeability testing of the boreholes SP-6, SP-7 and SP-8.

The boreholes designated SP were used for standard penetration tests according to the plan, in 5 m intervals along the clay core depth. The disturbed clay material samples were taken along with the ones obtained by core drilling. The basic physical and qualification tests on those samples were conducted by the Laboratory of the Civil Engineering Institute of Croatia Zagreb, Split Division.

Static penetration test (SPT). The clay core investigation using the SPT was conducted by Geoteknika Zagreb. The Gouda equipment of the Dutch manufacturer N.V. Goudsche Machinefabriek was used, with the maximum allowable penetration force of 200 kN and drawing force of 260 kN. The equipment is of electric type, sized according to the standard, capable of continuous cone resistance and local sleeve friction measurements. The measurements were recorded using the analog electric recorder. The friction reducer set was generally used at a specified distance from the probe. The SPT equipment was not placed continuously; the penetrations were set on a 10 m section along the depth, with preliminary drilling, with the exception of the surface vicinity where the section was even 20 m long.

Geophysical prospecting. The geophysical prospecting was carried out by the MOHO Ltd. Zagreb in June 1994. The prospecting encompassed cross-hole testing and electrical logging in 6 boreholes. The cross-hole seismic wave was induced by a hammer after the standard penetration test, in 5 m intervals along the boreholes SP-6, SP-7, and SP-8. The monitoring was conducted from the previously drilled and cased boreholes SP-5, FP-3 and DS-5. The three-component geophones were used for the cross-hole test, and the additional geophones were used to complete the cross-hole method with the seismic polar method applied on the longitudinal clay core profile.

For this case, the most important are the results of the shear waves " v_s " propagation velocity through the core, as shown in Fig. 2. In the majority of the measuring points in the upper part of the profile the deviation from the predominant trend and expected quantitative results for the clay materials is noticed (higher v_s speeds at the 5 m depth and, according to the borehole SP-8 even to 20 m depth). Having in mind that at the depth of 10-15 m from the top the core is still comparatively narrow, and that the

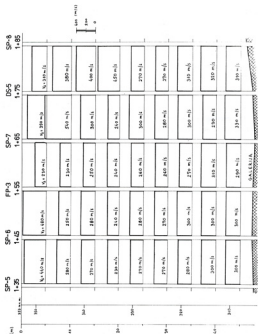


Fig. 2 Shear wave velocities in clay core by cross-hole method

boreholes are shifted from the core center towards the upstream shell, it might have happened that the seismic waves have found a shorter privileged passage through more stiff filter layers and the rockfill, therefore the average velocities v_s are somewhat higher than expected on low depths. The deviations at the borehole SP-8 are not covered by such a possible explanation.

Trial grouting of clay core. Within a framework of the geotechnical investigations, the clay core trial grouting was carried out by gravity grouting, boreholes B3 and B4 and the downward grouting procedure along with the drilling, boreholes B1 and B2. The trial grouting was carried out by Geotehnika Zagreb. The grouting was carried out on the dia. 66 mm boreholes, and during the grouting mix settlement the clay was swelling, which caused closing of the boreholes. The borehole B-2 suffered the grouting mix loss at pressure 1.5 bar at the station 10-15 m, and at pressure 3.0 bar at the station 15-20 m.

Laboratory testing of the clay core samples. The laboratory testing was carried out at the Geotechnical Department laboratory of the Civil engineering Institute of Croatia Zagreb, Split Division. The disturbed samples were subject to a large number of classification and index tests, granulometry, unit density, Atterberg limit, water content, swelling test, while the undisturbed samples were used to determine the clay strength, UU and CIU triaxial tests and deformation characteristics under static (oedometer) and cyclic loading (cyclic triaxial test at the laboratory of the Civil Engineering Faculty, university of Zagreb).

The swelling (swallowing) tests were conducted on the oedometer on an saturated sample with no vertical load, and the obtained swelling values ranged between 0.61 and 2.09%.

DAM CORE PROFILE DESCRIPTION

Generally, it has been noticed for the investigated part of the core that the clay materials had a comparatively high homogeneity, as confirmed by the description of the clay samples taken during drilling and reasonably matching results of the field, SPT, CPT, cross-hole and laboratory tests.

The heterogeneity caused by the damage was expected in the loose material zone with increased water content and low penetration resistance, or even caverns (holes) through which the drilling equipment would sink by its own weight. Such zones were not found in the investigated grid, which leads to a conclusion that possible core damage, if there was any, would primarily be local in character and smaller in scale.

The fact that, soon after explosion and reservoir emptying, the dam core sustained the pressure head approximately equal to the one of the half reservoir level with no major

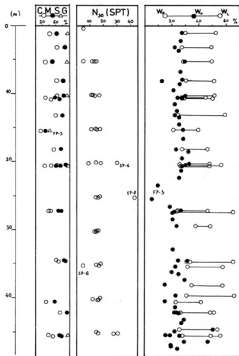


Fig. 3. Granulometric content, SPT blows and water content from all boreholes in clay core

seepage (the level during the gallery reconstruction when no accidents happened) indicates that there were no privileged passages established through the possible local damages which would cause subsequent unfavorable effect of the water. In the highest dam section the gallery height is about 4% of the core height, and its volume is under 1% of the clay core body volume (per metre of the dam height). It seems that the plastic core material filled in all the holes caused by mining of the gallery arch and adapted to the new conditions with no excessive fissures. The swelling (swallowing) characteristic of clay noticed during the investigation drilling and confirmed in the laboratory probably caused closing of the fine fissures in the sites of the lower local pressures.

Since the investigations were completed almost a year and a half after dam blasting, the results reflect the new core status.

The Fig. 3 shows the summary review of data on granulometric content, SPT blow count, Attenberg limit and water content for all the boreholes.

The granulometric review indicates that, generally, the material has over of 60-80% of silty and clayey particles, with the exception of the borehole FP-3 where a lens of the gravelly material was detected on the depth of 15 m, along with the gravel traces in the top borehole section.

The SPT blow count for the first 30 cm of penetration (not in accordance with the standard) gives a qualitative indication of uniformity along the core length and a mild trend of increase with the depth. The results which are significantly above the average are local, and probably the consequence of the encounter with the limestone cobble and gravel particles which are often reported in the drilling logs. The individual small values of a blow count on the top of the borehole and in depth, SP-8, may designate the local softening of the material caused by the drilling.

The clay water content along the borehole depth has been investigated in detail at the borehole FP-3. The common presentation shows that the results of other boreholes are within the determined data range. Generally, the values of the water content are between 20 and 30% and correspond with the water content range during the construction (Norveiller, 1983). Considerably lower results at the depth of 25 m are probably the consequence of the dried sample or, possibly, drained cohesionless material noticed at that depth in the borehole FP-3. The considerable increase in water content, over 40%, was noticed on only two samples from the borehole FP-3, at the core-gallery contact. Since the assumed explosion site is in the vicinity of this borehole, the increased water content could indicate local loosening of the clay core. The comparison with the Attenberg limit shows that, as a rule, the water content is on the verge of plasticity or somewhat lower along the entire borehole length.

The gravelly clay was noticed in the borehole FP-3 below the depth of 20 m; in places, the lens was even 60 m thick (under the limestone). It seems that this lens layer also reaches the 5 m distanced CPT-9 where a considerable increase in the core resistance was noticed at this depth. When the CPT measuring apparatus leaves the measuring range it is probably the consequence of the encounter of the probe with the limestone cobbles or larger gravel grains. Such grains or hard layers can deflect the CPT sound from the vertical. Since the preliminary drillings had been performed, the larger deviations have probably been corrected by this procedure.

The lens of the cohesionless permeable material was probably filled with water and its penetration during drilling and sounding established the relation and initiated seepage into the borehole FP-3. The very lens is located in the middle of the dam height, and probably does not present a direct consequence of the blasting. The explosion in the gallery would erupt the fragments of the grey concrete arch which had not been noticed, and the crest and shell zone settlement happened in the upper several meters of the dam in the direction of the reservoir rather than towards the core. It is similar with the limestone cobbles (white in color) which have been found in different depths and boreholes outside the assumed explosion site. The photographs of the excavation in ends also show occasional cobbles in core.

The shells were built by unloading of the rocks in thicker layers without compacting throughout the season, while the clay core construction was interrupted during the climatically unfavorable periods. For that reason, the constructed core level was below the elevation of the neighboring shells, and the individual stones could have rolled to the core as remain there. Therefore, it is not unrealistic to assume that the non-clay core ingredients have been present there since the initial dam construction.

The trial grouting has also indicated increased consumption of the material in individual stations, and the consolidation grouting within the remediation works has also shown increased consumption of material in the central core section. This procedure has generally decreased the core permeability and filled any possible cracks and lens, regardless of their causes. Consequently, it may be concluded that the carried out remediation measures, along with the construction of the cut-off wall in the central section, present a satisfactory and safe solution.

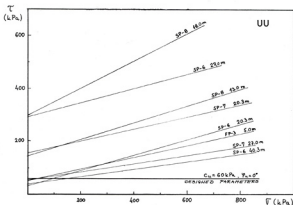


Fig. 4. The undrained strength parameters - measured and design values

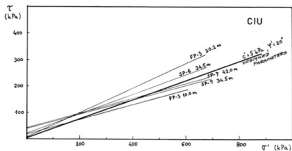


Fig. 5. The strength parameters in effective stresses - measured and design values

CLAY CORE PARAMETERS

Strength parameters. The material strength parameters are essential for the dam stability analysis by the method of limit states with the assumed failure mechanism (sliding surface). The laboratory researches included a number of triaxial tests, UU and CIU, while the undrained strength can be indirectly evaluated using the results of the SPT.

Strength parameters in total stresses (undrained strength) have been determined by the unconsolidated undrained, UU tests, and the summary presentation of the measurement results is given in Fig. 4. A considerably large range of results has been noticed as well as formation of groups with the undrained cohesion of about 50 kPa, 150 kPa and 300 kPa. The samples with the highest results had a somewhat lower water content, 17.5-20%, and higher compaction, dry density 1.74-1.83 t/m³, as compared to the other samples, $\gamma_d=1.5-1.6$ t/m³, $w_0>25\%$. Some of the samples with low parameter values have been reported as noncomplying with the standards.

The undrained strength determined from the CPT cone resistance by correlation is about 150-280 kPa, which is approximately matches the range of the mean results from the Fig. 4. However, according to the literature this correlation is not fully adequate for these comparatively saturated and comparatively overconsolidated materials.

With respect to the heterogeneity of the clay core compactness and humidity, it is only realistic to expect the wide range of the strength results. The selected design values $c_u=60$ kPa and $\phi_u=0$, also given in Fig. 4, are sufficiently conservative and encompass the most unfavorable cases of the measurement results. The strength parameters in effective stresses are determined by the consolidated undrained tests with the pore pressure measurement, CIU tests and a summary is given in Fig. 5. A somewhat smaller range of results is noticed. The measured cohesion ranges from 8-42 kPa, and the friction angles from 15.2 to 25.1°. With the respect to the expectations from the literature, the measured angles of 10-16° are too low for this type of material.

For analyses in effective stresses the parameters $c' = 5$ kPa i $\phi' = 20^\circ$ have been selected. The Fig. 5 shows that they are giving somewhat lower strengths than the ones measured in the area of the lower vertical stresses, while the calculated strengths in the region of higher stresses are within the average measurements. Since the safety factors are comparatively high for the deeper surfaces in the conducted analyses, such selection of parameters is considered satisfactory.

Dynamic parameters. The dam seismic stability analyses using the sliding block method include the stability calculations for the slopes and the dynamic response analysis. The previously specified parameters of undrained strength are used in the slope stability calculation by the boundary conditions method. The dynamic response analysis requests

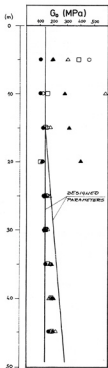


Fig. 6. Dynamic shear moduli " G_s " in clay core (measured and design values)

distribution of the initial shear moduli $G_0 = \rho v_s^2$, and nonlinear dependence of the moduli and damping on the shear cyclic deformation.

Based on the measured shear wave propagation velocity v_s and average density $\rho = 2.0 \text{ t/m}^3$, the shear modulus G_0 has been determined and presented in Fig. 6.

The calculated values of G_0 have been determined using the correlation $G_0 = 2,300 c_u$. The calculations include two modulus profiles along the depth - in the first, G_0 is constant, $G_0 = 138 \text{ MPa}$, $v_s = 263 \text{ m/s}$ and in the second, G_0 increases with the depth, $G_0 = 138\text{--}230 \text{ MPa}$, $v_s = 263\text{--}339 \text{ m/s}$. The Fig. 6 shows that the design and measured range of modulus are generally in good agreement but for the local deviation on the top of the borehole, as said earlier.

The nonlinear relations of modulus and damping with the shear strain have been tested on three samples using the cyclic triaxial test. Sample tests with measurement of strains directly on the sample (Hall sensors) show the model reduction in compliance with the applied relations from the literature, and the value of the measured maximum modulus is within the range of the geophysical measurements. The measured damping is somewhat higher than the one used in calculations, therefore the calculation is somewhat more conservative. The Fig. 7 shows analyzed results of a cyclic triaxial test.

It can be concluded from the above that the dynamic calculation parameters for the clay core have been selected realistically or conservatively, so it is considered that the conducted calculations meet the design requirements.

ROCKFILL PARAMETERS

The remediation works did not include testing of the rockfill material, so its parameters were evaluated according to the results collected during the initial dam construction (Norveiller, 1958) and the new information from the referenced literature.

Construction of the rock shells for the Peruća dam respected the than world practice in construction of rockfills (a relatively uniform granulation curve with some chippings, backfilling in high layers, water saturation). Such a practice (dumped rockfill) was notably changed in the late fifties and early sixties (Norveiller, 1983, Cooke, 1984), and the procedure of the stone unloading and dumping changed to the procedure of the compacted rockfill.

The lower section of the Peruća Dam rockfill was placed in 12 m thick layers, and the top section in 6 m thick layers. According to the descriptions, the downstream slope contained the finer grain stone, and the upstream slope the coarser grain stone.

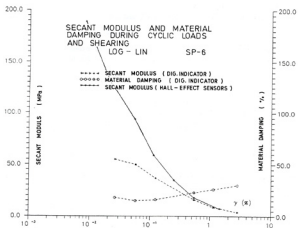


Fig. 7. The result of cyclic triaxial test on clay sample - secant moduli and damping

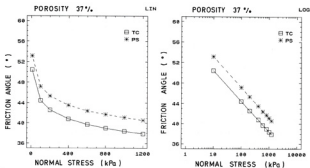


Fig. 8. Variation of rockfill friction angle with normal stress

The compacting of the rockfill was performed with the strong water jets (4 m³ of water on 1 m³ embankment).

The average granulometric composition of the stone from the quarry (Norveiller, 1958) shows that the maximum grain exceeds 500 mm, and that the fraction of the grain below 2 mm is less than 8%. The trial embankments had shown that the construction procedure might result in dry densities of 1.75 t/m³ to 1.88 t/m³.

The cohesionless materials of larger grain sizes are characteristic for **grain crushing**, and the resulting change in the mechanical behavior. The attempts to quantify and correlate this phenomenon with other measurable parameters are generally empirical in nature (e.g. Marsal 1973, Hardin, 1985). According to Hardin (1985), the Peruća Dam rock material has the breakage potential $B_R = 2.95$. The relative breakage B_R for the vertical stresses ranging from 200 to 1000 kPa and the horizontal stresses ranging from the isotropic consolidation to the passive pressure, ranged within the extremes from 1 to 15%. It has been estimated that the forecasted changes in the granulometric composition of the rockfill are modest in scope.

The strength of the cohesionless material depends on several factors, and the more important for the same mineral composition are compactness, granulometric composition and the value of the normal stresses.

The tests for the Peruća carried out on the pilot granite (Norveiller, 1958) conducted with variable maximum grain indicate increase in the friction angle with the increase in the maximum grain diameter, which indicated that the friction angle of 45° is realistic or even conservative option.

The detailed information on the Peruća Dam rock material strength can be obtained by the empirical approach of Barton and Kjaernsly (1981) which includes the impact of the basic material strength, size of an average grain in the rockfill, grain shape, material compactness and normal stresses. The effective friction angle (Φ') is determined from the relation

$$\Phi' = R \log (S/\sigma'_c) + \Phi_0$$

where:

- R - equivalent grain roughness dependant on the grain shape and origin, and material compactness, porosity, after building in
- S - equivalent grain strength dependant on uniaxial compression of an intact rock, average grain diameter, D_{50} and investigation type, triaxial compression or plane strain test
- σ'_c - effective normal stress
- Φ_0 - intrinsic friction angle, usually within the range of 25°-35°

The diameter of the average Peruća material grain was $D_{50}=150$ mm. Assuming the uniaxial limestone strength $\sigma_c=100$ MPa and average base angle $\Phi_b=30^\circ$, the effects of the compactness and normal stresses for triaxial compression, TC, and plane strain PS. The characteristic results for porosity $n=37\%$, density 1.75 t/m³, are shown in Fig. 8 for linear and log stress scale.

Crushing of grain under higher stresses decreases the average grain diameter, and subsequent settlement of the rockfill and filling of pores increases compactness of the rock material. With respect to the conducted analyses these effects should increase the material strength, but according to the evaluations this contribution should not be considerable. Additionally, such natural "compacting process" is neither controlled nor uniform, so it should not be taken into the account in conservative design. Therefore, the angle of 45° is selected for calculations for the range of normal stresses of $0 - 200$ kPa, and the friction angle of 41° for higher normal stresses.

Maximum shear modulus, G_0 . modulus at low strains has been determined from the empirical terms of Seed and Idriss for cohesionless materials (1970):

$$G_0 = 220 K_{3max} \sigma_0^{0.5} \text{ (kPa)}$$

where

- K_{3max} - dimensionless parameter dependant on the soil type and compactness
- σ_0 - mean effective pressure

According to the literature, the parameter K_{3max} used for gravel and stone materials is over 100, and for modern compacted rockfills it ranges between 150 and 250 (Gazetas, 1992). E.g. for seismic analyses of the Oroville high dam $K_{3max}=130$ was used, where the compacted rockfill had significantly smaller fractions than the Peruća rockfill. Since the Peruća rockfill compactness is probably poorer, the estimated parameter K_{3max} is within the range $K_{3max} = 100 - 150$.

The curves recommended in literature (Seed et al, 1985) were used for nonlinear relations of the shear modulus and damping with cyclic shear strain amplitude.

CONCLUSION

The geotechnical investigations conducted in the central clay core section of the Peruća Dam were undertaken in order determine the possible damage of the core which could have imperilled construction of the cut-off wall and in order to check the calculated soil parameters.

Within the investigated grid 5 x 5 m no major areas of loose material or large cracks and cavities were detected. The investigations indicated possible core damage in the immediate vicinity of the explosion site, and the local lens of the permeable coherent material in the central core section.

The core preparation by the consolidation grouting has probably filled the minor permeable zones and enabled safer performance of the impermeable cut-off wall which should provide the dam with the final safety.

The design calculation parameters for the clay core are selected realistically or on the safe side with respect to the investigation results. The rockfill parameters are estimated on the basis of the data collected during the dam construction and relevant literature.

REFERENCES

- [1] Barton, N., Kjaernsly, B. (1981)
Shear strength of rockfill. Journal of the Geotechnical Engineering Division, ASCE, Vol. 107, GT7, 873-891.
- [2] Cooke, J.B. (1984)
Progress in rockfill dams, 18th Terzaghi lecture, Journal of Geotechnical Engineering ASCE, Vol. 110, No. 10, 1381-1414.
- [3] Gazetas, G., Dakoulas, P. (1992)
Seismic analysis and design of rockfill dams: state-of-the-art. Soil Dynamics and Earthquake Engineering, Vol. 11, No. 1, 27-61.
- [4] Hardin, B.O. (1985)
Crushing of soil particles. Journal of Geotechnical Engineering, ASCE, Vol. 111, No. 10, 1177-1192.
- [5] Marsal, R.J. (1973)
Mechanical properties of rockfill. Embankment Dam Engineering, J.Wiley and Sons, New York, pp. 109-200.
- [6] Nonveiller, E. (1958)
The design and construction Peruća rockfill dam on the Cetina river, Proceedings of the IVth Congress on High Dams, Belgrade, pp. 210-217.

- [7] Nonveiller,E. (1983)
Rockfill dams - design and construction. University of Zagreb, Školska knjiga, Zagreb, p. 359
- [8] Seed,H.B., Idriss,I.M. (1970)
Soil Moduli and Damping Factors for Dynamic Response Analyses. EERC, Report No. EERC 70-10, University of California, Berkeley.
- [9] Seed,H.B., Seed,R.B., Lai,S.S., Khameneipour,B. (1985)
Seismic design of concrete faced rockfill dams, Concrete Face Rockfill Dams-Design, Construction and Performance, ASCE 1985, 459-478 (quoted per Gazetas, 1992)

SEISMIC STABILITY OF THE PERUĆA DAM

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ABSTRACT

The seismic stability was analyzed by the so-called "sliding block method" which includes the dam dynamic response analysis, determination of yielding accelerations of potential sliding bodies and the estimate of permanent dam displacement after the design earthquake.

The dynamic response was studied by variable-stiffness-shear-beam model including the parametric analyses over the wide range of dynamic stiffness properties of earth materials and appropriate nonlinear stress-strain behavior. The input seismic parameters included several real strong-motion records covering the wide range of frequencies. The yielding accelerations were determined by conventional slope stability method. The estimate of permanent displacements included the generation of appropriate diagrams using the same acceleration time-histories as in response analysis.

The same method was applied in analysis of seismic behavior of the dam during the strongest historical earthquake of 1990, using the original acceleration records from the stations close to the Peruća Dam site.

INTRODUCTION

The Peruća Dam is a rockfill dam with almost vertical, comparatively narrow clay core protected with multi-layer filters on both sides. The core material is high-plasticity clay. The dam is located in the Cetina River canyon. In layout, it is slightly concavely curved in the upstream direction. Until its intentional damage by the enemy early in 1993, the dam had a thirty odd years history of perfect operation. It might be interesting to mention that no dam of this type has suffered any considerable damage during stronger earthquakes.

Generally, the dam body must not collapse during the maximum probable earthquake because of the material liquefaction in its body or foundation, due to sliding along a

sliding surface, because of the loss of heightening above the reservoir level, or because of the uncontrolled seepage through the cracks. The issue of liquefaction is not present in the case of the Peruća Dam because the dam does not contain liquefaction susceptible material. The sliding along the potential sliding surfaces and the loss of the heightening above the water level shall be checked by the seismic analyses. The uncontrolled seepage through the cracks in the reconstructed dam section shall be prevented by adequate upstream and downstream filters. The upstream filter is intended to fill any possible cracks. The uncontrolled seepage from the damaged dam section shall be prevented by construction of the impermeable cut-off wall which is not subject to the progressive erosion in case of possible seepage through the cracks.

Today, the seismic analysis of the rockfill dams can make use of a number of procedures, from the very simplified to the very complex ones (ICOLD 1986, Gazetas, Dakoulas 1992). The currently predominant opinion is that the analyses based on the Newmark sliding block method (Seed 1979, Marcuson, Hadala, Franklin 1980) are sufficient and satisfactory for the dams which had not been built of the material which suffer considerable loss in strength under the seismic loads, or founded on such materials. Since neither the original nor the reconstructed Peruća Dam contains materials susceptible to liquefaction, the seismic analyses have been conducted by the Newmark sliding block method applied in the below steps:

- I Comparative dam seismic response analysis by the Ambraseys and Sarma procedure (1967), and applying the SHAKE program (Schnabel, Lysmer, Seed 1972). In the first procedure, the triangular dam cross-section is presented by the homogenous elastic beam deformed only by horizontal displacement ("shear beam"). In the second procedure, the dam triangular cross-section is also presented by the "shear beam" layered in a finite number of horizontal layers with shear stiffness, namely damping adapted to the average size of the shear strains. This enables approximate consideration of nonhomogeneous and nonlinear dam behavior.
- II Calculation of yielding horizontal accelerations causing the unit safety factors on the potential sliding surfaces in the dam body, applying the modified SSTABI program.
- III Calculation of permanent sliding body displacements applying the Newmark sliding block method (Newmark 1965, Seed 1979), based on the results of the steps I and II, as the supporting data for evaluation of the permanent dam deformations.

It is planned that the reconstructed dam sections shall be built of material of equal or better characteristics compared to the original dam material, and modelling of their arrangement in the dam cross-section has respected the final dam design.

The above analysis steps, together with adequate parametric analysis, have been followed in more detail in the central dam cross-section reconstruction, maximum height $h=60$ m.

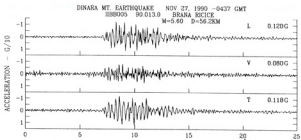


Fig. 1. The record of strong ground motion in Dinara mountain
 (Geophysical Department, Zagreb)

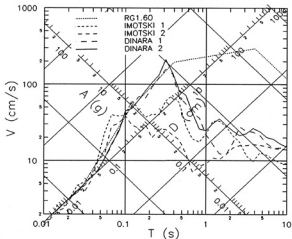


Fig. 2. Response spectra of recorded Dinara earthquakes

The analysis has also been performed on the cross-sections of various heights along the dam, as well as the analysis of the dam heightening effect on its seismic response. The dam seismic stability prior to the damage has also been evaluated.

In addition to the seismic analyses, the technical requirements for building in of the material into the reconstructed dam sections specify the measures which comply with the state-of-the-art practice in dam design in the seismic areas.

SEISMIC LOAD

The seismic load on the Peruća site was determined by the Andrija Mohorovičić Geophysical Department of the Natural Science Faculty of the University of Zagreb, 1994.

The maximum expected bedrock accelerations were obtained by the deterministic procedure for the three types of earthquake:

- earthquake on the dam site, magnitude $M=5.7$, maximum acceleration $a_{\max}=0.160$ g, maximum intensity $I_{\max}=8.0^{\circ}$ MCS
- close earthquake, epicentral distance - 7 km, magnitude $M=6.50$, maximum acceleration $a_{\max}=0.286$ g, maximum intensity $I_{\max}=8.6^{\circ}$ MCS
- remote earthquake, epicentral distance - 21 km, magnitude $M=6.6$, maximum acceleration $a_{\max}=0.197$ g, maximum intensity $I_{\max}=7.80^{\circ}$ MCS.

The maximum seismic acceleration $a_{\max}=0.22$ g for the return period $T=1,000$ years was obtained by the probabilistic method. These values correspond with the earthquake S2 while the maximum acceleration for the so called operating basis earthquake S1 is $a_{\max}=0.17$ g.

Many regulations request that the seismic stability of an object be inspected for the operating basis earthquake. It is not clear which boundary criteria for the operating basis earthquake should be met by the rockfill dam with the satisfactory level of stability for the maximum probable earthquake. Therefore, it is not common to conduct special operating basis earthquake analyses for the rockfill dam body, and the maximum value $a_{\max}=0.29$ g shall be used for further analyses.

Since, in addition to the maximum acceleration, the seismic response of the dam is considerably affected by a wide range of frequencies, several acceleration time-histories were selected for the analyses. The first group of accelerograms consists of the synthetic accelerogram compatible with the design spectrum RG 1.60 for the nuclear power plants (NEPH4), and the records on the historical Montenegro earthquakes of 1974 (IZIS, 1974).

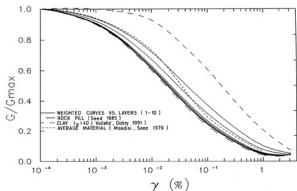


Fig. 3. The range of maximum shear moduli in clay core and upstream and downstream shells. The height of dam $H = 60$ m

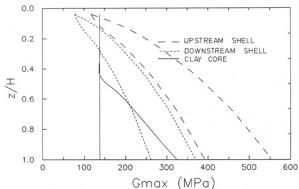


Fig. 4. The reduction of shear modulus with shear strain amplitude for clay and rockfill in comparison with weighted average curves for horizontal layers in dam model

- ULCINJ, April 15 or 16, 1979, W-E component, $a_{\max}=136.9 \text{ cm/s}^2$. The record of earthquake on the site.
- BAR, May 24, 1979, N-S component, $a_{\max}=198.5 \text{ cm/s}^2$. The record of earthquakes on the central distance from the site.
- VELIKI STON, April 15, 1979, N-S component, $a_{\max}=264.1 \text{ cm/s}^2$. The record of earthquakes with epicentral distance over 100 km.

The second group of accelerograms includes the earthquake records for the Peruća Dam site vicinity taken during 1990 on the Rāšica Dam. The longitudinal components of the below earthquakes have been selected, Geophysical Department archives:

- Imotski-Grude earthquake, April 3, 1990, $M=5.0$, $a_{\max}=0.113 \text{ g}$
- Imotski-Grude earthquake, August 27, 1990, $M=4.6$, $a_{\max}=0.074 \text{ g}$
- Mt. Dinara earthquake, November 27, 1990, 4.37 hours, $M=5.6$, $a_{\max}=0.120 \text{ g}$
- Mt. Dinara earthquake, November 11, 1990, 5.51 hours, $M=5.5$, $a_{\max}=0.069 \text{ g}$

These are the records of the medium to remote earthquakes. For the analysis, all the earthquakes are adjusted to maximum acceleration of $a_{\max} = 0.29 \text{ g}$.

The Fig. 1 presents a time record of one of the Dinara earthquakes, and the Fig. 2 shows their response spectra, also adjusted to $a_{\max}=1.0 \text{ g}$.

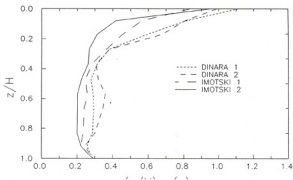
The response spectra for all used accelerograms reflect wide range of frequencies of historical earthquakes, while the synthetic accelerogram NEPH4 covers conservatively the whole range of relevant periods.

INSTALLED MATERIAL PROPERTIES

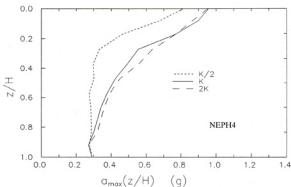
The design parameters of the dam material have been determined on the basis of the investigations conducted for the original design and evaluations from the referenced literature. With respect to possible variations, the sensitivity of results has been analyzed for a wider range of possible parameters. The below values have been selected for the characteristic dam sections:

Clay core:

- density: 2 t/m^3
- strength (undrained): $s_u=40\text{--}60 \text{ kPa}$ or $s_u=60 \text{ kPa}$ from the crest to the depth of 24 m, to increase linearly below that depth to $s_u=150 \text{ kPa}$ on the depth of 60 m (dam foundation)
- maximum shear modulus:
 $G_s=2,300 \cdot s_u$



**Fig. 5. Maximum acceleration vs. normalized dam height
for average stiffness K**



**Fig. 6. The influence of dam stiffness on maximal acceleration
- acc. NEPH4**

Rockfill:

- density 2.1 t/m³
- strength: a) $\Phi=45^\circ$ for depth to 20 m on the submerged upstream shell, namely for the depth to 10 m on the dry downstream shell;
b) $\Phi=41^\circ$ for greater depths;
- maximum shear modulus:
$$G_0 = 220 K_{\text{sum}} p^{0.5} \text{ [kPa]}$$

where p - mean effective stress, and $K_{\text{sum}}=100-150$

The Fig. 3 shows the range of maximum design shear moduli, separately for the submerged, upstream and unsubmerged, downstream dam shell and the clay core. The analysis used the weighted average moduli.

For changes in shear modulus and damping relative to the strain amplitude the curves of Vučetić and Dobry (1991) were selected for the clay core (with index of plasticity $I_p = 40$) and recommended rockfill curves (Seed et al, 1985).

The said modulus-strain curves are given in Fig. 4, along with the "average material" curve recommended by Makdisi and Seed (1979). The figure also shows the average curves for the ten horizontal layers weighed according to the participation of the rockfill and clay in the horizontal layer.

Other materials: Because of the low impact in the dam cross-section, the filters are included in the rockfill zone. The same relates to the cut-off wall which has not been modelled separately. The bedrock on which the dam is founded has the assumed shear wave velocity of $V_s = 1,000$ m/s and density of 2.4 t/m³.

DAM SEISMIC RESPONSE ANALYSIS

SHAKE Program. The SHAKE Program (Schnabel, Lysmer, Seed 1972) enables calculation of the "shear beam" response on the seismic input either in its base, bedrock or the bedrock outcrops. The "shear beam" can be randomly layered. The seismic input is given in the form of an accelerogram within the time domain. The calculation is conducted within the frequency domain in the complex area, which enables setting of the frequently independent damping. Nonlinear behavior of the material, presented as the dependence of the shear modulus and damping on the average shear strain amplitude, is taken into the account by the applicable iterative procedure. In calculations, the seismic input is set on the bedrock outcrop. Such approach, in addition to being more realistic

than the approach in which the seismic input is set in the dam foundation, also avoids the dubious resonant effects referenced to in the literature (Gazetas, Dakoulas 1992).

All analyses were based on the assumption that the reservoir is full.

The analysis of the model sensitivity to variable stiffness was carried out for the average stiffness K , double stiffness $2K$ and half stiffness $K/2$. The average stiffness K is defined as the mean value of the maximum shear modulus G for the estimated range of parameters, Fig. 3. The values $2K$ and $K/2$ encompasses wider range of stiffnesses than the shown ones.

The SHAKE Program analyses were conducted for all 8 accelerograms and the specified dam stiffnesses.

The average acceleration on the dam crest is:

- (a) for stiffness $K/2$ $a_{max}=0.7834$ g
- (b) for stiffness K $a_{max}=1.0238$ g
- (c) for stiffness $2K$ $a_{max}=1.1687$ g

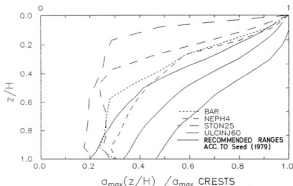


Fig. 7. The distribution of acceleration in dam body normalized with crest acceleration - stiffness K

The ratio of the average maximum acceleration of the dam crest and the bedrock ranges between 2.70 and 4.03. The calculated curves are quantitatively and qualitatively in conformance with the expected behavior of this type of dams (Gazetas and Dakoulas, 1992).

The Fig. 5 shows distribution of maximum accelerations vs. normalized dam height (depth) for the stiffness K and the other group of earthquakes; the Fig. 6 presents the impact of the design stiffness range for accelerogram NEPII4. The normalized, relative depth is the ratio of the dam depth z measured from the crest, and maximum dam height H .

The Fig. 7 shows the acceleration distribution per the dam height normalized with maximum crest acceleration for the average stiffness K . For comparison, the recommended design range is delineated (Seed, 1979). The results of this calculation give a somewhat more favorable distribution of the normalized accelerations for the upper dam section. It has consequently been appraised that the average curve of the recommended range is sufficiently conservative for the permanent displacements analysis.

Along with the detailed calculations conducted for the top dam cross-section, the seismic response was checked for the cross-sections of different heights along the dam. The highest cross-section in most cases gives the highest acceleration in the crest. The exceptions are the cross-sections 30-40 m high with approximately 5% larger crest acceleration than the highest cross-section, which is still within the limits of the total average results.

Ambraseys and Sarma Procedure. The Ambraseys and Sarma (1967) diagrams are often used for the dam seismic response analysis when the more complex calculation by numerical methods are not applied. Here, this procedure is used only for comparison with the SHAKE program.

The base period T_0 of the dam modelled as a homogenous triangular shear beam is

$$T_0 = 2.61 H/v_s$$

where:

- H dam height
- v_s average shear wave propagation velocity

The equivalent velocity v_s for nonhomogeneous cross-section shall be taken at the 1/3 of the dam height. The range of equivalent maximum shear moduli for this depth is 270-370 MPa. In addition to the initial reduction of the modulus to 40-50% of the initial value, the values within the range of $v_s = 235 - 304$ m/s are obtained, which gives the basic dam periods for the height of $H=60$ m within the range of $T_0 = 0.51 - 0.67$ seconds.

Correction for the third size has been evaluated according to the Gazetas and Dakoulas (1992). The ratio of the 3D and plane strain period for the trapezoidal canyon and the ratio of the crest length and height 5-6 is 0.82, so the corrected periods make $T_{0,cor} = 0.42-0.55$.

According to the results of the prior analysis, it may be estimated that the average damping in the dam is between 10 and 15%, so with the damping correction the final value of amplification in the dam crest $k(y) = 2.55$.

This value is approximately comparable with the average amplification factor obtained by the SHAKE program which ranges between 2.68-4.03. In this way, the Ambraseys and Sarma procedure would give a somewhat less conservative accelerations in the dam crest than the SHAKE program.

YIELDING ACCELERATIONS

According to the Newmark "sliding block" method (Newmark, 1965) the yielding acceleration of the sliding mass is the seismic acceleration of the base, namely adequate inertial force, which brings the total sliding body mass into the status of the boundary equilibrium, i.e. reduces the static safety factor to unity. The seismic dam analysis requests determination of the sliding mass acceleration on different heights along the dam cross-section since the seismic accelerations of the dam underlayer amplify along the cross-section height.

The yielding accelerations for the Peruća Dam analysis have been determined by the computer program SSTAB1 (Wright, 1974), and the corresponding horizontal force was entered by rotation of the coordinate system. The safety factors have been calculated for several horizontal forces and, by interpolation, the acceleration (force) determined for each sliding surface which gives the safety factor equal to unity.

The design profile of the reconstructed dam and selected circular sliding surfaces on the upstream and downstream slope are shown in Figs. 8 and 9.

The case with full reservoir has been analyzed since it is the most critical for the seismic loading.

The calculations were conducted with the strength parameters for clay $c_u=40$ kPa and for the rockfill $\phi=45^\circ$ to the stress of 200 kPa, namely $\phi=41^\circ$ for higher stresses. The selected angle of friction for filters $\phi=40^\circ$.

The calculation results are given in Table 1.

U P S T R E A M			D O W N S T R E A M		
sliding surface	safety factor	yielding acceleration	sliding surface	safety factor	yielding acceleration
1	2.368	0.228	1	1.794	0.336
2	1.708	0.172	2	1.848	0.389
3	1.783	0.193	3	1.989	0.498
4	1.972	0.210	4	1.767	0.338
5	2.423	0.280	5	1.992	0.470
6	1.697	0.160	6	2.051	0.639
7	2.011	0.200	7	1.551	0.225
8	2.534	0.250	8	1.662	0.284
9	1.474	0.100	9	1.740	0.327
10	1.753	0.131	10	2.019	0.465
11	2.147	0.197	11	1.862	0.403
12	2.321	0.280	12	1.850	0.414
13	1.788	0.174			
14	2.057	0.200			

Table 1 Safety factors and yielding accelerations for sliding surfaces

The static safety factors are more than 1.5, but for the shallow sliding surface on the top of the dam which has the safety factor 1.47, which corresponds to the safety factor of the surface parallel with the slope in the incoherent material. This sliding surface has also given the lowest yielding acceleration.

As expected, the yielding accelerations are lower on the downstream side since seismic force affects the total mass (soil and water), and the strength is dependent on the effective stresses which are lower than in the saturated downstream slope.

POTENTIAL PERMANENT DISPLACEMENTS

According to the Newmark's concept, sliding of a block down the slope starts at the moment when the base acceleration exceeds the block yielding acceleration, and ends when the velocity changes the sign. Sliding of the block back up the slope is not taken into

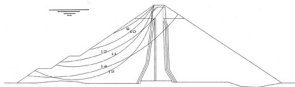


Fig. 8. Sliding surfaces on upstream dam side - the model for analyses of reconstructed dam



Fig. 9. Sliding surfaces on downstream dam side - the model for analyses of reconstructed dam

consideration, therefore such block displacement is also its permanent displacement. The next surpassing of the yielding acceleration shall cause a new downward block displacement, and the final permanent block displacement down the slope after an earthquake is the sum of all individual displacements.

If this concept is applied, the accelerograms of individual earthquakes can be analyzed and the cumulative permanent displacement calculated with respect to the ratio of yielding acceleration $k_y g$ and maximum earthquake acceleration a_{max} . The Civil Engineering College of the University of Zagreb developed a computer program ACCE for this purpose, and it was used for processing of all the accelerograms. The accelerograms were tailored to the maximum acceleration $a_{max} = 0.29 g$, so the obtained results can be directly applied in these analyses.

The results of the described double-integration time-related acceleration according to the sliding block method are shown in Fig. 10. It is clear that the accelerograms which were giving extreme results in the seismic response analysis have poorer cumulative effect.

The Fig. 10 shows the expected displacement range for the earthquake of magnitude $M=6.5$ according to Seed (1979). The synthetic record NEPH4 follows relatively well the upper limit of the expected range, while the records of the historical earthquakes show high dispersion.

The further analyses should take into account that these permanent displacements present the total downslope displacement of the block, so determination of e.g. permanent settlement should make account of the vertical displacement component only.

Evaluation of the permanent dam displacements. Determination of the permanent dam displacements after the assumed earthquake encompasses the results of all the prior analyses. The seismic response analysis gives the values of maximum accelerations, the limit state analysis method gives the yielding accelerations of the sliding masses, and the analysis (integration) of accelerograms gives a final permanent displacement dependant on the acceleration ratio.

Two cases have been analyzed by this procedure:

- (a) maximum historical earthquake at the dam site prior to damage,
 - (b) maximum expected earthquake for the reconstructed dam.
-
- (a) According to the Geophysical Department data, the strongest earthquake on the Peruća Dam site was in 1990, and the estimated maximum acceleration on the bedrock was $a_{max}=0.16 g$. According to the reports, the reservoir water become turbid, but there is no evidence that the dam suffered any damage.

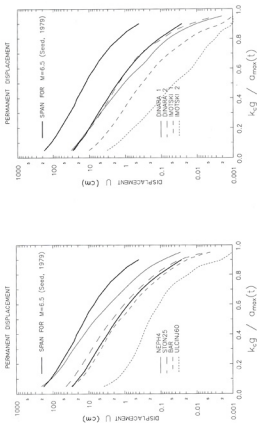


Fig. 10. Permanent displacements of sliding block vs. acceleration ratio for analyzed time histories

The analysis of the dam behavior during this event encompassed as follows: it can be concluded from the seismic response analysis that the ratio of the crest and bedrock acceleration is about 4.0 for the higher investigated stiffness. Since the lower seismic forces are accompanied with lower shear strains, therefore the modulus reduction and damping are smaller. Therefore, it is realistic to expect that the result for the higher expected stiffnesses is relevant for evaluation. With the acceleration ratio of 4, expected crest acceleration is 0.64 g. The yielding accelerations are specified in the Table 1, and the acceleration distribution as per the dam height has been taken for the extreme of the Seed's design range. For displacement evaluation the Mt. Dinara accelerograms which give the higher displacement values in the said set of records for the dam site vicinity were used and adjusted to the maximum acceleration of 0.16 g.

The extreme downslope displacement of the sliding surface on the top of the dam is 6 cm, and the corresponding design crest settlement would be 3.5 cm. This value is within the limits of accuracy for the entire procedure, so it may be evaluated that this displacement has no significant consequences on the dam body, as has been proven in reality.

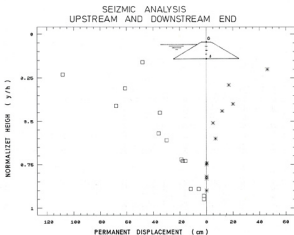


Fig. II. The extreme down slope permanent displacements of sliding bodies for reconstructed dam

This also leads to the conclusion that the entire "sliding block" procedure, applied with reasonable conservatism, does not give exaggerated or unrealistic results.

- (b) Evaluation of the permanent displacements for the reconstructed dam includes the below elements:

- The maximum dam crest acceleration of $a_{max}=1.03$ g is taken over from the seismic response analysis, as corresponds with the results for an average dam material stiffness. The mean value of the recommended design range of Seed (1979) has conservatively been selected for the acceleration distribution per the dam height;

U P S T R E A M			D O W N S T R E A M		
sliding surface	relative depth	permanent displacement	sliding surface	relative depth	permanent displacement
1	0.16	48	1	0.29	17
2	0.31	61	2	0.51	5
3	0.45	35	3	0.83	0
4	0.73	16	4	0.44	12
5	0.95	2	5	0.75	0
6	0.57	36	6	0.90	0
7	0.73	18	7	0.20	46
8	0.89	6	8	0.40	20
9	0.23	108	9	0.60	7
10	0.41	68	10	0.82	0
11	0.72	19	11	0.74	0.4
12	0.93	2	12	0.90	0
13	0.61	30			
14	0.89	12			

Table 2 Estimated permanent sliding mass displacements (cm)

- the yielding accelerations of individual sliding surfaces have been taken over from the Table 1;

- the permanent displacements have been read for the upper limit of the magnitude $M=6.5$ earthquake range.

The final permanent displacements calculated in this way are given in Table 2, and Fig. 11.

The maximum estimated downslope permanent displacement on the dam crest is 108 cm, and the corresponding permanent settlement is 59 cm.

The displacements on the upstream end are in average 2 times larger than on the downstream end where the extreme displacement is 46 cm.

The above results do not lead to the conclusion on the catastrophic consequences for the dam, but the local deformations are possible along with the displacements of individual sliding masses which may cause cracks in the dam body.

CONCLUSION

The seismic stability analysis of the reconstructed Peruća Dam has given the below results:

- in case of the maximum probable earthquake on the site, with maximum acceleration of a_{max} , the dam body might suffer damage due to the permanent displacements of the potential sliding masses;
- it is estimated that the global dam stability shall not be impaired due to the dam overtopping, because the maximum expected permanent dam crest settlement is about 60 cm and there is an ample freeboard between the maximum operating level of the reservoir and the crest; any possible dam crest damage is not considered significant for the global stability;
- the potential displacements of the sliding masses in the upper half of the dam may cause local damage in the slope and cracks in the dam body, but it is estimated that the planned construction of filters on the both core flanks, and the impermeable cut-off wall in the central section, shall sufficiently prevent any possible outbursts and seepage through the core.

REFERENCES

- [1] Ambraseys, N.N. and Sarma, S.K. (1967), The Response of Earth Dams to Strong Earthquakes. *Geotechnique* 17, No. 3, pp 181-213.
- [2] ICOLD (1986), Earthquake analysis procedures for dams, ICOLD Bulletin 52, Commission Internationale des Grands Barrages, Paris, 148 p.
- [3] Gazetas, G., Dakoulas, P. (1992), Seismic analysis and design of rockfill dams: state of the Art; *Soil Dynamics and Earthquake Engineering*, 11, 27-61.
- [4] Geofizički zavod (1994), The Peruća Dam - A Seismic Parameters Study, Andrija Mohorovičić Geophysical Department, Mathematical College, University of Zagreb
- [5] IZHS (1984), Corrected accelerograms and integrated ground velocity and displacement curves, IZHS Bull. No1, Skopje.
- [6] Makdisi, F.I., Seed, H.B. (1979), Simplified procedure for evaluation embankment response, *Journal of the Geotechnical Engineering Division, ASCE*, 105, GT12, 1427-1434.
- [7] Marcuson, W.F. III, Hadala, P.F., Franklin, A.G. (1980), Current methods of dynamic analysis for seismic stability of earth dams, *Engineering Geology*, 16 19-28.
- [8] Newmark, N.M. (1965), Effects of Earthquakes on Dams and Embankments. *Geotechnique* 15, No. 2, pp 139-160.
- [9] Seed, H.B. (1979), Considerations in the earthquake-resistant design of earth and rockfill dams, 19th Rankine Lecture, *Geotechnique* 29, No. 3, pp 215-263.
- [10] Seed, H.B., Seed, R.B., Lai, S.S., Khameneipour, B. (1985), Seismic design of concrete faced rockfill dams, *Concrete Face Rockfill Dams-Design, Construction and Performance*, ASCE 1985, 459-478.
- [11] Schnabel, P.B. Lysmer, J., Seed, H.B. (1972), SHAKE: A Computer program for earthquake response analysis of horizontally layered Sites, Report No. EERC 72-12, Earthquake Engineering Research Center, University of California, Berkeley, California.
- [12] Vučetić, M., Dobry, R. (1991), Effect of soil plasticity on cyclic response, *Journal of Geotechnical Engineering*, Vol. 117, No.1, 89-107.

- [13] Wright, S.G. (1974), SSTABL- A General Computer Program for Slope Stability Analyses, Dep. of Civil Eng., University of Texas at Austin

THE PERUĆA DAM STRESS-STRAIN ANALYSIS

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ABSTRACT

Construction of an impervious cut-off wall within the core of the damaged Peruća Dam was one of the dam remediation measures. The cut-off wall has been planned for cutting in into the dam foundation. Since during its exploitation the dam shall be exposed to strains, the interaction of the dam body and the cut-off wall is expected.

As required for the mechanical interaction analysis of the restrained cut-off wall bottom and dam body, a comprehensive calculations program was run encompassing the initial dam stress, removal of the top ten meters of the dam, cut-off wall construction, subsequent rebuilding of the top dam part, and reservoir impoundment after the dam remediation.

The Final Element Analysis Method (FEM) method was applied in calculations, and the material elastic/plastic behavior, nonlinear stress-stiffness relations, namely strength-effective stress relations in dam earth material were respected.

INTRODUCTION

Construction of an impermeable cut-off wall in the damaged Peruća Dam core has been planned as one of the dam remediation measures. The cut-off wall was conceived to be cut into the dam foundation. Since during the exploitation the dam shall be exposed to strains, the interaction of dam and cut-off wall is to be expected. A comprehensive analysis of the subject interaction has been performed within a study, and its basic results are briefed in the present paper.

The cut-off wall is a stiff structure within the dam body, restrained in the bottom section, in the point where it enters the dam concrete foundation or the hard bedrock. The cut-off

wall material, the plastic concrete, was to be designed so as to ensure impermeability under all design conditions the dam might be subject to during the future exploitation. In addition to the technological quality requirements, the cut-off wall must remain functional regardless of any possible dam strains.

As required for the mechanical interaction analysis of the restrained cut-off wall bottom and dam body, a comprehensive calculations program was run encompassing the initial dam stress, removal of the top ten meters of the dam, cut-off wall construction, subsequent rebuilding of the top dam part, and reservoir impoundment after the dam remediation. The calculations were conducted in compliance with the proposed dam remediation design solution, and all the possible variations of the dam body material stiffness and strength were taken into consideration as the set cut-off wall material stiffness and strength range. The Final Element Analysis Method (FEM) method was applied in calculations, and the material elastic/plastic behavior, nonlinear stress-stiffness relations, namely strength-effective stress relations in dam earth material were respected. The FLAC [1] computer program which enables solution of such complex strain problems in geotechnics was applied.

DISCRETIZATION BY FEM METHOD

Discretization by the FEM method was conducted on the central, the highest, dam cross section (60 m). A relatively dense final elements grid was used totalling 2000 elements for the entire dam profile.

In order to simulate the initial dam stresses, the dam construction in six 10 m high layers was simulated. Subsequently, removal of the top layer within the reconstruction works was simulated. The final simulation encompassed new construction of the top layer, additional heightening of the original dam by additional backfilling of the downstream slope, and the reservoir water level increase to the top elevation, assuming the cut-off wall construction has already been completed.

All the analyses used effective stresses, including the effect of hydrostatic pressures in the dam upstream shell and water flow through the dam core with the exception of the impervious cut-off wall segment.

MATERIAL MECHANICAL PARAMETERS

The mechanical parameters of the material have been taken over from the corresponding sections on Peruća Dam stress-strain and seismic analysis, and from the Duncan *et al* (1980) Report [2]. The Report was used for the stiffness parameters evaluation for the

nonlinear relation of the shell (rockfill) and core (clay) for which no data are available from the investigation works.

The earth material friction angle was calculated from

$$\varphi' = \varphi'_0 + R \log \frac{S}{p'} \quad (1)$$

where p' is mean effective stress in the dam body prior to introduction of the new loading increment.

The Young's modulus of elasticity is expressed by:

$$E = K p_a \left(\frac{p'}{p_a} \right)^n \quad (2)$$

where p_a is the atmospheric pressure.

The evaluated values of particular parameters are given in Table 1.

	Shells	Core
φ'_0 (°)	30	20
c' (MN/m ²)	0	0.005
R (°)	6	0
S (MN/m ²)	100	-
K (-)	200	50
n (-)	0.5	0.7
ν (-)	0.30	0.45
ρ (t/m ³)	2.07	1.97

Table 1 Soil parameters

φ'_0, R, S	parameters of the nonlinear strength law
c'	- cohesion
K, n	- Young's modulus parameters with respect to the mean effective stress p'
ν	- Poisson's coefficient
ρ	- saturated material density

The basic cut-off wall parameters - Young's modulus, compressive strength after 28 days, and cut-off wall bending strength, excluding the zone with tensile stresses, are given in

Table 2. The later value is calculated under the assumption that the full compressive strength of concrete is activated in the cut-off wall cross-section, and that the cross-section cannot take over the tensile stresses (the concrete has no tensile strength). The cross-section bending strength calculation took into consideration the cut-off wall tensile force effect. It consists of the cut-off wall dead load yield decreased by the seepage water buoyancy through the dam and the contribution of the dam strain caused by the upper reconstructed layer loading and unloading caused by the upstream dam shell saturation.

Young's modulus (MN/m ²)	300
Compressive strength(MN/m ²)	1.0
Cut-off wall bending strength (MNm/m') in cross-section	0.08

Table 2 Cut-off wall parameters

Based on the above assumed dam material parameters the horizontal shifts were calculated for the original non-reconstructed dam crest caused by the reservoir impoundment from the bottom to the maximum level. The obtained crest horizontal shift was about 6 cm. The measured crest shifts caused by abrupt reservoir impoundment were 1 to 4 cm, and the horizontal shifts during the 20 add years of dam operation would be 20 cm. The dam material stiffness parameters were varied in the parameter numerical analyses. Variation of parameter K to a half or double value from Table 1 resulted in the original dam crest shift of 14.5 and 0.6 cm. These calculations enabled the analysis of the effect of estimated soil parameters on the dam strains.

Also, the parameter analyses encompassed the range of possible cut-off wall parameters (a considerably more stiff and a considerably more soft cut-off wall than the one in Table 2).

The internal cut-off wall forces were simulated for the calculation purposes for the below dam body lifetime stages:

- 6 backfilling stages during the dam construction in 10 m high layers; removal of the top layer (dam reconstruction);
- cut-off wall construction, backfilling the last layer and heightening of the dam crest by additional filling of the downstream slope, and hydrostatic loading of the reservoir water;
- seismic loading, specified as the equivalent static force of 15% of the total weight.

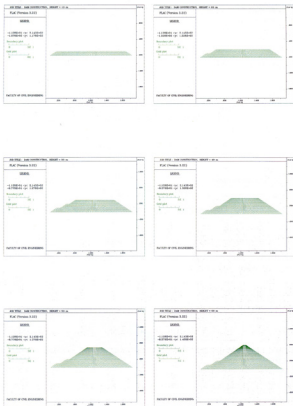


Fig. 1. Final elements grid for various construction stages

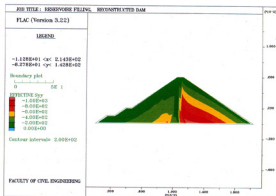


Fig. 2. Final stage - effective Syy vertical stresses [kN/m^2]

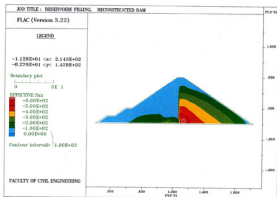


Fig. 3. Final stage - effective Sxx horizontal stresses [kN/m^2]

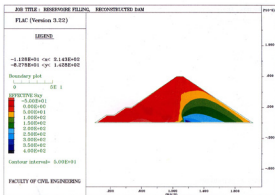


Fig. 4. Final stage - effective S_{xy} shear stresses [kN/m^2]

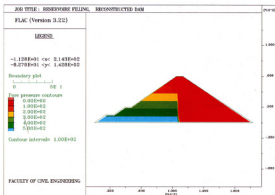


Fig. 5. Final stage - pore pressures [kN/m^2]

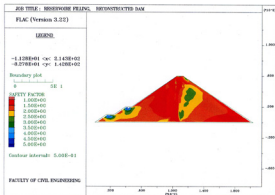


Fig. 6. Final stage - safety factors

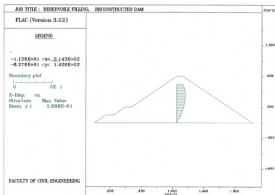


Fig. 7. Final stage - horizontal x cut-off wall displacement [m]

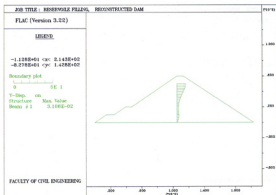


Fig. 8. Final stage - vertical y cut-off wall displacement [m]

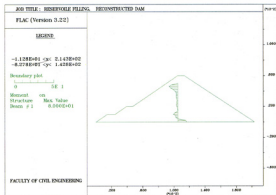


Fig. 9. Final stage - Cut-off wall bending moments [kNm/m¹]

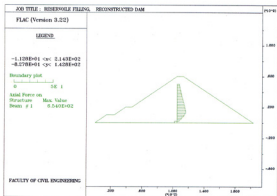


Fig. 10. Final stage - Additional longitudinal cut-off wall forces [kN/m^3]

NUMERICAL ANALYSES RESULTS

The characteristic results were presented in graphical form only for the material parameters from the Tables 1 and 2.

The Fig. 1 shows the final elements grids for various construction stages.

The Figs. 2-10 show the final stage results - reservoir impoundment for the designed reconstructed dam (with the cut-off wall and the new crest with the dam heightening).

The Figs. 2-6 show diagrams of the intervals of vertical S_{yy} , horizontal S_{xx} and effective shear S_{xy} stresses, pore pressures in (kN/m^2) and safety factors.

The safety factors are specified as the ratio of the average strength and the average mobilized shear stress for each grid element, thus they are not directly comparable with the safety factors from the boundary limit equilibrium relating to the entire sliding body.

The Figs. 7-10 show the horizontal (x) and vertical (y) cut-off wall shifts in (m), bending moment in (kNm/m^3) diagrams and the additional longitudinal (axial) forces in (kN/m^3).

PARAMETER ANALYSES RESULTS AND CONCLUSIONS

The basic parameter analysis conclusions, 21 altogether, may be rounded off as follows:

- Regardless of possible dam and cut-off wall material stiffness, the dam body safety factors are satisfactory, and its global stability shall not be imperilled throughout its lifetime.
- For all alternatives of the dam material stiffness and the cut-off wall material stiffness and strength, a plastic hinge occurs in the cut-off wall bottom section, i.e. its bending strength value is reached. The dam strains are causing the forced cut-off wall shifts which cause the plastic hinge occurrence in the fixed wall point at the dam foundation level. In all other points the cut-off wall bending moments are lower than the cross-section bending strength, regardless of the assumed stiffness and strength parameters.
- The transverse crack in the cut-off wall hinge was caused by the lack of the cut-off wall material tensile strength, and it goes for 80 cm from the upstream to the downstream cut-off wall edge. In the most extreme case, the crack opening is less than 5 mm.
- The parameter analyses indicate that any cut-off wall material softening, and consequently decrease in its strength, where the Young's modulus and strength ratios of 1000:1 to 300:1 are analyzed, does not eliminate the hinge phenomenon in its bottom section. Since the cut-off wall material should be of satisfactory strength in order to ensure protection against progressive erosivity of the cut-off wall material in the crack site, it is recommended to design the cut-off wall material so that its Young's modulus does not exceed 400 MN/m^2 , and the compressive strength after 28 days is not less than 1 MN/m^2 .

REFERENCES

- [1] - 1993), FLAC, User's Manual, Itasca Consulting Group, Inc., Minneapolis, Minnesota.
- [2] Duncan, J.M., Byrne, P., Wong, K.S., Mabry, P. (1980), Strength, Stress-Strain And Bulk Modulus Parameters For Finite Element Analyses Of Stresses And Movements In Soil Masses, Report No. UCB/GT/80-01, College of Engineering, University of California, Berkeley, California, p. 70

INVESTIGATIONS AND CONTROL OF MATERIAL PLACEMENT INTO THE DAM BODY

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SPLIT

ABSTRACT

The first part (A) of this paper includes an overview of the issues related to investigations, namely finding of material suitable for the Peruća Dam remediation. The investigations were focused on finding of sufficient quantities of material, particularly clay, respecting the strict design requirements, its unavailability in the dam vicinity, the information on the found material and comparison of its physical/mechanical characteristics with the design requirements.

The second part (B) of the paper presents statistical analyses of the results obtained by permanent and control testing of the clay material, and numerical analysis of results obtained by building in of the rock material and their comparison with the design requirements.

(A) MATERIAL DEPOSIT INVESTIGATIONS

INTRODUCTION

An interesting issue related to the Peruća Dam profile is why the decision was brought to use such a narrow clay core. The preliminary investigations for determination of the reservoir and dam location revealed that the quantity of quality clay material for construction of the impermeable clay core is insufficient. Therefore, the core itself offers the answer.

The same situation was faced during the dam remediation. The Peruća Dam Remediation Final Design specified that all excavated quality material from the upper dam section and the left and right flanks shall be carefully sorted and deposited on suitable spoil areas in the dam vicinity, and their physical/mechanical properties preserved to the maximum possible degree. In that way all the quality and usable material was available for dam reconstruction.

The above philosophy was confirmed as correct, because 23,000 m³ of the upstream rip-rap and material used for the upstream and downstream shells was saved. And, what is the most important, it included 10,000 m³ of clay. The filter material was not saved because it was degraded and mixed with the material from the shells and clay from the core.

The physical/mechanical characteristics of the excavated material had generally been known, but the additional laboratory tests were conducted in order to determine the required quality and characteristics of particular types of material, and set the construction criteria.

The material excavated from the existing dam body and dumped in the immediate dam vicinity was used to start the dam remediation works. In this way, the time was bought for finding the rest of required material, particularly clay.

MATERIAL DESIGN CHARACTERISTICS AND REQUIREMENTS FOR PLACEMENT INTO THE DAM BODY

The material backfilled, namely built in into the dam body includes:

- **Clay** - natural coherent nondispersive material of adequate plasticity, compressibility and very low permeability in compressed state;
- **Filter material** - incoherent rock material of adequate grain-size composition, of natural origin, sand and gravel, or obtained by blasted rock crushing;
- **Coarse grain rock material** - material for shells of adequate grain-size composition, obtained by blasting and subsequent sorting;
- **Rock blocks** - large rock blocks for protection of the upstream slope obtained by blasting;
- **Clayey rock material** - rock material mixed with binding clay of natural origin and obtained by mixing of clay with the crushed stone of adequate grain-size composition.

The said material was expected to meet the general requirements, i.e. no traces of organic material and salts soluble in water, while the specific requirements were those related to physical and mechanical characteristics and methods, namely standards of testing, which shall be discussed below.

Clay Core Material

The clay of intermediate to high plasticity (CI/CH) was used for the clay core construction. Its characteristics were expected to meet the following criteria:

- grain-size composition $35\% < 0.002 \text{ mm}$ $75\% < 0.06 \text{ mm}$
- liquid limit $43\% < W_L < 60\%$
- index of plasticity $20\% < I_p < 38\%$
- compaction, modified Proctor $W_{opt} = 17.24 (\%)$ $\gamma_d = 15.5-17.5 (\text{kN/m}^3)$
- built-in clay compaction $\gamma_{d \text{ min}} = 0.95$ γ_d , modified Proctor
- building in of organic clays and silts (OL, OI, OH and M), organic waste, humus, roots and similar was not permitted, along with the materials of high compressibility and the one which would change their physical/mechanical characteristics with time.

Clay Core Protection Filter Material

In addition to the above, the filter material used for the clay core was to meet the below grain-size composition:

GRAIN DIAMETER (mm)	%
60	100
20	65-100
6	40-65
2	15-40
0.6	7-25
0.2	0-13
<0.06	0

The material was also expected to meet the Sherard requirement that for a 15% grain $DF_{15}=0.7-1.5 \text{ mm}$, i.e. that the unit weight of the built-in material is 0.7% of maximum unit weight obtained in laboratory.

Shells Rock Material

The material used for the upstream and downstream shells was blasted and treated rock with specific density of $2,500 \text{ kg/m}^3$ and the below grain-size composition:

GRAIN DIAMETER (mm)	%
600	95-100
60	55-70
6	20-35
2	10-25
0.06	0-5

Additionally, the attention was paid that the maximum fraction diameter does not exceed 500 mm, and the unit weight of the built-in material is not less than 20.0 kN/m³.

Upstream Slope Rip-Rap

The material used for the upstream slope rip-rap is hard, high density, durable and mechanically resistant rock of minimum specific density of 2,500 kg/m³. Such a material had to contain large rock fractions and stone blocks not exceeding about 1.0 m in diameter, while the grain-size composition was expected to be:

BLOCK DIAMETER (mm)	%
1000	70-100
600	50-70
200	25-40
60	10-20
20	0-10

Dam Crest Clayey Gravel

The material planned for construction of the dam crest was to meet, along with the above, the following grain-size composition requirement:

GRAIN DIAMETER (mm)	%
60	100
2	60
0.06	25
0.002	10

The compacted material unit weight was expected to meet the criterion of 18.0 kN/m^3 .

Testing Standards

Since there are no adequate applicable Croatian regulations and standards, use of all international standards such as DIN, BS, ASTM, and JUS was allowed, as well as use of regulations which are in compliance with the Act on Taking-over the Standardization Act.

INVESTIGATIONS AND TESTING

Clay Core Material

The clay material quantity saved after removal of the dam ends and upper section made only one third of the total material quantity required for new the dam core construction. Therefore, about $20,000 \text{ m}^3$ of clay of equal or similar characteristics was still needed.

Finding of additional quantity of clay posed a large problem because of its limited quantity in the area. The deposit distance from the site, deposit operation cost-effectiveness, access roads and transportation had also to be born in mind, and the specified design physical/mechanical characteristics met.

In addition to the above facts, the situation in which the entire Peruća and Satrić area was a front line additionally slowed down investigation works. The risk of left or intentionally planted mines was high. Particularly risky area was the Alebić Kula zone and Peruća-Prolići village on the way to the Peruća Dam, where the artillery was situated during the combat actions. Therefore, the visual inspection of potential sites was carried out with permission and assistance of the Croatian Army. Before approaching any site, the inspection and cleaning of the area was requested from the engineer corps, after which the permission was granted for access to that site. Although no planted mines were found, these precautions had to be undertaken for the sake of safety of the investigation team.

Deposit Site Selection

Based on the above criteria, the Satrić deposit was selected as a potential site within a wider neighborhood, Fig. 1. Its air distance from the Peruća Dam is approximately 3.0 km. The visual inspection of the area resulted in selection of three sites (Vardića Kuće, hamlet of Prolići and village Lijići) which, according to geological indices, had the highest potential quantity of material. The remaining area was assessed as unsuitable because it is arable land near the family houses.

According to the Clay Deposit Investigations Program which specifies the in situ and laboratory test, the field investigations were initiated.

The field investigations in the above sites started late in April 1994, and they included drilling of eleven investigation boreholes. The investigation results were positive only for the Vardića Kuće site where the clay was of adequate characteristics and in substantial quantities, although the intercalations included excessive organic and silty material. The quantity of about 8,000 m³ of clay was confirmed, which was still not sufficient for the dam core completion.

The results for the other two sites were negative, and the quantities were both insufficient and costly for exploitation, therefore the investigations moved towards the farmland off the Satrić area. Visual inspection was conducted of the complete Satrić area, hamlets of Peruća and Prolići, Hrvace, even a part of the Sinjsko polje. Based on visual prospecting and with support of the local community, another eight potential sites were determined.

Further prospecting did not include drilling - it was carried out with machinery, by digging prospecting pits. In this way the deposits in the Hrgovići hamlet, the access area downstream the Peruća, and Prolići hamlets were inspected, prospected and classified. The Hrgovići site had very small quantity of high-plasticity clay (approx. 800 m³), while the clay on other sites, Peruća and Prolići, was generally of intermediate and high plasticity, but the presence of silty material with fairly high quantity of organic matter made them nonconforming to the set criteria.

In the meantime, while the prospecting of the said sites was under way, unexpected problems with the land owners in Vardići arose. They requested very high payments for exploitation of clay from their land although they had been promised that their plots shall be recovered after the remediation works. The three-month investigations gave no result. Another four months had been spent on reinvestigations of the Satrić area (Lokve, Naplavi, Majčiči, Alebića Kuće). The reservoir site, controlled by the Croatian army, was checked along with the Sinjsko polje by the Cetina River. No positive results were obtained. When we almost gave up, the local population informed us about the Zorica site approximately 10 km far from the Peruća Dam, in the direction of the town of Sinj.

Its proper site is a hill slope above the hamlet of Zorica. The deposit itself had the surface area of some 10,000 m².

Although in a low spirit, we started with visual prospecting, and then with excavation of prospecting pits.

The in situ and laboratory tests were controlled by a soil mechanics supervisor, on the basis of the collected data.



Figure 1. Broader investigated area with prospected sites

In Situ Investigations

The in situ investigations were conducted late in November, as they included excavation of the prospecting pits to the depth of 3.50 m. Altogether 7 pits were excavated and marked SJ-1 to SJ-7. The pit size was approx. 2.50 x 1.20 x 3.50 m, with the exception of the pit SJ-6 which was excavated to the depth of 0.60 m. The field AC classification of material run concurrently with excavations, and the sketches were prepared of the pit shape and depth and of any changes in material layering. The arrangement of the pits is given in Fig. 2.

Soil Composition and Characteristics

Based on the results of the conducted in situ AC classification and the laboratory tests it was determined that the humus layer in all the investigated sites of the Zorica deposit was underlain with high-plasticity clay (CH), but for the SJ-6 pit site where the tufa was encountered already on the depth of 0.50 m. The prospected clay was of different colors, from light yellow, clay brown and gray, to black. The classification of excavated material confirmed that, in almost all sites, the clay had slight intercalations of gray silt where the clay broke into lumps. The silt intercalations were 1-2 mm thick, irregular, and did not affect characteristics of the complete clay mass.

After laboratory confirmation of the soil composition and characteristics, slightly over 20,000 m³ of clay was determined. The exploitation of the available quantity had to be very careful so that the clay shortage would not be suffered. This almost happened when the clayey gravel, fine uniform silt and even coal were encountered in one part of the deposit. The problem was soon solved by slight widening of the deposit.

Laboratory Tests

From all the prospecting pits, but for SJ-6, the material samples were taken in quantities sufficient for laboratory tests. Large samples of disturbed material were taken from the pits SJ-1, SJ-2, SJ-3, SJ-4, SJ-5 (two samples) and SJ-7. They were tested for mechanical and physical characteristics of soil according to applicable regulations, and the below tests were performed:

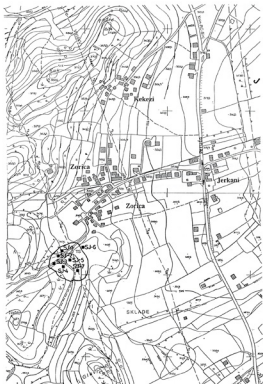


Figure 2. Zorica deposit map

- moisture content	W _o (%)
- Atterberg limit of plasticity	W _o , W _p , W _L , I _p (%)
- grain-size composition	-----
- specific density	ρ (gr/cm ³)
- relative porosity and modulus of compressibility (oedometer)	e, M _v (MPa)
- impermeability	k (m/s)
- triaxial shear test	c (kPa) i φ (°)
- optimum compaction per standard Proctor	W _{opt} (%) i γ _c (kN/m ³)
- organic matter	(%)
- erosivity - Pin Hole test acc. to Journal of Geotechnical Engineering Division	

First, the laboratory tests were conducted for physical characteristics of soil layers, such as the moisture content, specific density, grain-size composition, Atterberg limit of plasticity, and organic matter percentage, along with the mechanical characteristics according to standard and modified Proctor test.

For all other tests the material was prepared so that its moisture content was optimum according to the modified Proctor tests.

Two different layers, according to their physical appearance, were detected in prospecting pit SJ-4, so the samples were prepared for testing by mixing of these two materials in ratio approximately equal to the natural relationship.

Swelling of all tasted materials ranging between 6 and 16% was recorded during the triaxial shear test, oedometer and erosivity tests.

When the samples for the triaxial shear test were taken, it was noticed that the bond between the layers is comparatively poor. The samples were 8 cm high, and layers 3 cm thick. This was almost certainly the consequence of preparation of samples according to the modified Proctor test, since the optimum moisture content is comparatively low related to comparatively high energy input. This is only a remark on its possible effect on the laboratory results.

The samples tested on the oedometer have first been completely saturated with water, and the tests were conducted after the swelling process had been completed.

The erosivity tests (pin-hole) were also conducted on samples prepared according to the modified Proctor tests. No numerical data were obtained since the swollen material closed the water passage hole within a very short period of time. Thus, the additional nonstandard tests were conducted. When the first test was completed, namely when each

sample was completely saturated (12 tests were conducted), a standard hole was again made in the sample and the pin-hole test repeated, under the maximum water column pressure.

Such tests have shown that the water flow through the samples was invariable, the water was clear, no turbidity was detected, namely no signs of corrosion.

After the tests had been completed, the samples were checked in detail and no defect was detected.

Zorica Deposit Exploitation

Since the deposit is located on a hill slope, the only way of its exploitation was the so called frontal excavation. The clay was taken from the deposit front side using the excavator, and directly loaded into the truck. The material was transported to the construction site, and sometimes to the spoil area. The favorable fact for this method of work was that the moisture content of the clay at the deposit was somewhat higher than optimum. The moisture was mainly lost during transportation and building in, so no major problems arose during construction works. When the moisture content was too high, the clay was dried out at the construction site or spoil area. The clay transportation intervals depended on weather conditions and construction schedule for individual dam elements. The Photo 1. illustrates clay excavation at the Zorica deposit.

Comparison of Clay Design Characteristics and Laboratory Test Results for the Zorica Deposit

The laboratory clay test results obtained for the Zorica deposit before the deposit exploitation started have been compared with the design requirements. The comparison of the Zorica deposit samples test results has shown that the clay characteristics fully complied with the design criteria, and were even better regarding the plasticity, as shown in the plasticity and grain-size diagrams, Fig 3 and 4.

The enclosed diagrams are either individual or summary presentation of the laboratory tests conducted in order to get the Zorica deposit exploitation approval. The presented laboratory results were taken for the basic requirement to be met during building in of the clay.



Photo 1. Clay excavation at the Zorica deposit

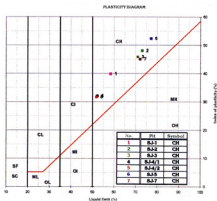


Figure 3. Zorica deposit clay plasticity diagram

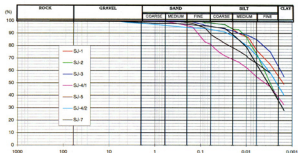


Figure 4. Zorica deposit clay grain-size diagram

Clay Core Filter Material

As already said, the excavated filter material was not saved, so it had to be replaced in full quantity of approx. 15,500 m³.

Since the broader Peruća Dam area did not dispose of adequate natural rock material of natural origin, but in the than occupied zone from which the material had been used during the dam construction, the activity has been initiated to find the material in the existing quarries in the Split-Peruća direction. All the quarries in the area were checked, because the filter material was to meet the design physical/mechanical characteristics and the grain-size composition requirements, and the manufacturer was expected to guarantee production of required quantities. The Srinjine and Dugobabe quarries were singled out as the ones meeting the requirements. They are owned by the same company, therefore the supply was guaranteed.

When the first filter material shipment arrived on the site, the field control grain-size analyses were conducted which confirmed the manufacturer's certificates and guarantees, as shown in the enclosed grain-size curves.

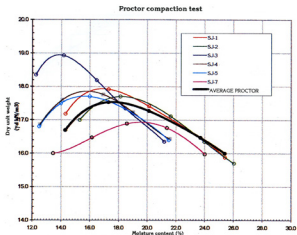
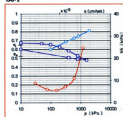


Figure 5. Modified Proctor compaction test diagram

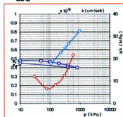
PIT	SJ-1	SJ-2	SJ-3	SJ-4	SJ-5	SJ-7	AVERAGE MODIFIED PROCTOR
W_{opt}	17.76	18.97	14.35	15.70	21.40	19.65	17.97
γ_d	17.99	17.81	18.96	18.01	16.67	16.97	17.40

The minimum and maximum compaction tests were carried out in the field laboratory in order to achieve the unit weight of the filter material of 0.7 of the maximum unit weight compaction obtained in laboratory. The laboratory results were: maximum compaction 24.16 kN/m^3 , and minimum unit weight 18.83 kN/m^3 for the material moisture content 4.81% and 2.11%. Therefore, any compaction between these values meets the design requirements.

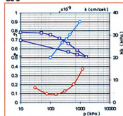
SJ-1



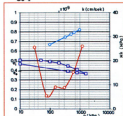
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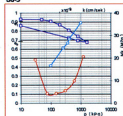
SJ-3



SJ-4



SJ-5



SJ-7

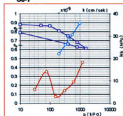


Figure 6. Diagrams of oedometer compressibility with permeability

Upstream and Downstream Shell and Upstream Slope Rip-Rap Rock Material

When the dam upper section and ends were excavated, all the rock material was dumped at the old quarry some 300 m upstream the dam. The excavated material was disposed by dumping down the quarry slope, which caused its segregation. From this segregated material, larger blocks were sorted out for the upstream slope rip-rap, while the finer and cleaner fractions were taken for construction of the dam shells. Some of this quality rock material was also built in into the dam, but its quantity was not sufficient for the dam completion, particularly because of the planned dam heightening. The other necessary quantity was obtained by blasting in the existing quarry.

The quarry area contains the Upper Cretaceous limestone, poorly to medium fractured, generally of vertical direction, and the fractures filled with red clay. The front view of the quarry shows that the top zone of some 15 m is more fractured and weathered, while the lower zone of some 20 m contains a thick layered compact limestone. For that reason the quarry was used in two stations, for the upstream and downstream shell.

The exploitation design was prepared for the quarry reopening. It specified optimum explosive consumption with respect to the required grain-size.

All the rock material required for the Peruća Dam was taken from the same quarry. The rock from that quarry was suitable for remediation because of its known physical/mechanical characteristics checked and confirmed during the Peruća Dam lifetime. Therefore, no special testing of the physical/mechanical characteristics of this material was required for remediation.

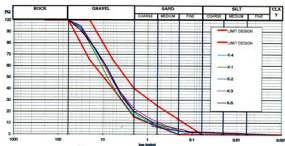


Figure 7. Design limits and the initial control of the Srelnjine and Dugobabe quarries filter material grain-size composition

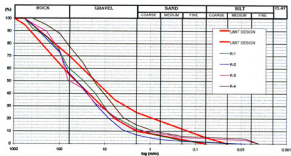


Figure 8. Shell design limits and initial grain-size control



Photo 2. Quarry exploitation

The Photo 2 shows the quarry exploitation method, the weathered and compact zones, and the grain-size curve, Fig. 8, of the prior initial grain-size analysis confirming the exploitation and explosive consumption optimization design.

(B) MATERIAL BUILDING IN CONTROL

INTRODUCTION

The Perúa Dam Remediation Final Design and the Material Quality Control and Assurance Program specified quality control during the material placement by **permanent** and **control** tests.

The permanent tests, which were in the Contractor's scope of works, were conducted after the material placement in a particular dam element on a daily basis; the control tests were conducted on a random basis as checks of permanent tests, and they were conducted by the Owner's Supervision Team.

TESTING PROGRAM

For all the in situ and laboratory tests specified by the design documentation the type, testing standards and test sample quantity were stipulated. Therefore, all tests were sorted out according to the type of material, sampling quantity, type of test and applicable standards.

The permanent and control tests included the below classification and mechanical tests:

Permanent tests

Clay core

-	unit weight	1 pc/ 25 m ²
-	moisture content	1 pc/ 25 m ²
-	standard Proctor test	1 pc/3000 m ³
-	grain-size composition	1 pc/ 500 m ³
-	Atterberg limit	1 pc/ 500 m ³
-	specific gravity	1 pc/3000 m ³
-	organic matter	1 pc/1000 m ³

Filter material

- grain-size composition 1 pc/ 300 m³
- unit weight 1 pc/ 50 m²

Shells

- grain-size composition 1 pc/10000 m³
- unit weight 1 pc/ 5000 m²

Control tests**Clay core**

- unit weight 1 pc/ 250 m²
- moisture content 1 pc/ 250 m²
- standard Proctor test 1 pc/3000 m³
- grain-size composition 1 pc/1500 m³
- Atterberg limit 1 pc/1500 m³
- specific gravity 1 pc/3000 m³
- organic matter 1 pc/3000 m³
- carbonates 1 pc/3000 m³
- compression and impermeability 1 pc/3000 m³
- triaxial strain test 1 pc/3000 m³

Filter material

- grain-size composition 1 pc/ 500 m³
- unit weight 1 pc/ 250 m²

Shells

- grain-size composition 1 pc/30000 m³
- unit weight 1 pc/15000 m²

CLAY BUILDING IN CONTROL**Introduction**

Two sorts of clay were used for the dam clay core, the so called "old" and "new" clay.

Because of the earlier described problems with the required quantities of clay, the imperative request during excavation of the dam upper section and ends was to save as much clay from the existing core as possible. The clay was disposed in the immediate vicinity of the dam ("old" clay). The "new" clay is the clay from the Zorica deposit.



Photo 3. "Old" clay spoil area



Photo 4. Clay core compacting

Since all the relevant material characteristics were laboratory tested, the clay placement started in July 1994. The "old" clay from the spoil area on the site was used first, Photo 3.

The clay placement lasted until October 21, 1994 when the left flank reached the elevation of 345.80 m a.s.l.

In order to give a final definition of the clay placement criteria, a trial field was prepared according to the instructions of the soil mechanics supervisors. The trial field was placed in the immediate vicinity of the dam, on the operating platform, and clay transport, spreading and compacting was carried out by mechanical equipment used for subsequent works on the clay core. By variations in layer thickness and number of roller runs (vibratory roller, Bitelli Ghilbi C100) the data were obtained which lead to the conclusion that the best results are achieved with 30-35 cm thick loose layer, with 8 runs in the same track, and vibrating action. The compacting speed was limited to 5 km/h.

The majority of laboratory tests were carried out on the site, in field laboratories set near the dam, and equipped for the dam material placement monitoring.

The results of the permanent and control tests indicated that the "old" clay is of intermediate to high plasticity (CI/CH), reddish brown, and the other characteristics were within the below limits:

- specific density	$\rho = 2.6 - 2.72 \text{ (kg/dm}^3\text{)}$
- liquid limit	$W_L = 45.77 - 54.33 \text{ (\%)}$
- limit of plasticity	$W_P = 25.40 - 29.58 \text{ (\%)}$
- index of plasticity	$I_P = 18.99 - 24.75 \text{ (\%)}$
- carbonate	$\text{CaCO}_3 = 2.98 - 8.50 \text{ (\%)}$

For illustration of the achieved quality of works, and later analysis and presentation of the results, an example of statistical analysis of the **moisture content and unit weight test for the "old" clay built in into the left dam end between September 15 and 30, 1994 is given.**

The numerical analysis of results of 167 samples shows **mean optimum moisture content** determined in laboratory of $W_{opt}=23.20(\%)$ while the average achieved in-situ moisture content was $W_f = 23.07(\%)$, which gives the difference of $-0.13(\%)$, so the dispersion of results is evaluated as low. The graphical presentation in Fig. 9 shows that in 95% cases the moisture content of clay built in during that period was between -4 and $+4(\%)$ of the mean optimum moisture content determined in a laboratory.

The same sample was used for analysis of the results obtained for **dry unit weights**. The numerical analysis indicates that the average in-situ dry unit weight was $\gamma_d=16.34 \text{ (kN/m}^3\text{)}$ which, compared with the optimum $\gamma_{opt} = 16.28 \text{ (N/m}^3\text{)}$ gives the

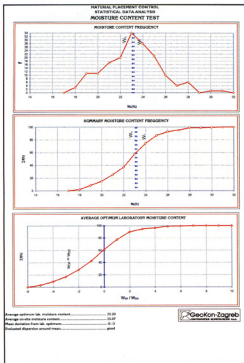


Figure 9. Statistical analysis of moisture content control and permanent test results during placement of the "old" clay late in September 1994

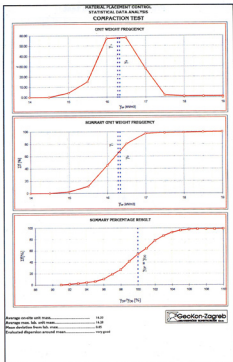


Figure 10. Statistical analysis of unit weight control and permanent test results during placement of the "old" clay late in September 1994

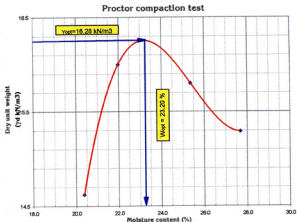


Figure 11. Referenced modified Proctor test for a set of "old" clay data

difference of 0.06 (kN/m³). Therefore, the dispersion of results around the mean is evaluated as very low. The graphical presentation in Fig. 10 shows that in 97% cases the compaction of the clay built-in during that period was over 95% of the modified Proctor test value (design building-in criterion). The Fig. 11 shows a diagram of the referenced modified Proctor test for this group of results. Based on the above said, it may be concluded that the clay placement during this period was of high quality and in compliance with the design requirements.

Since November 22, 1994 the exploitation of the "new" clay from the Zorica deposit and its subsequent building in started. The building in lasted until December 16, 1994, when the poor weather conditions stopped the works. The works continuation was planned for March 1995, but the unfavorable weather conditions and the situation at the deposit

delayed the works until May, when the work started to develop more seriously and was occasionally interrupted by intensive rain.

The permanent and control tests for the "new" clay were conducted under the same conditions as for the "old" clay, only the moisture content checks were more frequent with respect to the situation at the deposit; the additional prospecting pits were dug, and the samples were taken for full material identification analysis.

It was determined by permanent and control laboratory tests that the "new" clay was mainly of high plasticity (CH) with some minor parts of the deposit where the clay of intermediate plasticity (IC) was predominant. The clay from this deposit varies in color, from light yellow, clay brown and gray, to black. Other characteristics were within the below limits:

- specific density	$\rho = 2.6 - 2.74 \text{ (kg/m}^3\text{)}$
- liquid limit	$W_L = 39.97 - 77.10 \text{ (\%)}$
- limit of plasticity	$W_p = 18.50 - 30.90 \text{ (\%)}$
- index of plasticity	$I_p = 19.26 - 52.40 \text{ (\%)}$
- carbonate	$\text{CaCO}_3 = 1.08 - 6.66 \text{ (\%)}$
- organic matter	$0.00-0.42 \text{ (\%)}$

For illustration of the achieved quality of works, and later analysis and presentation of the results, an example of statistical analysis of the moisture content and unit weight test for the "new" clay built in into the left and right dam flank during May 1995 is given.

The numerical analysis of results of 148 samples shows mean optimum moisture content determined in laboratory of $W_{opt} = 18.94(\%)$, while the average achieved in-situ moisture content was $W_T = 17.81(\%)$, which gives the difference $-1.13(\%)$, so the dispersion of results is evaluated as very low. The graphical presentation in Fig. 12 shows that in 95(%) cases the moisture content of clay built in during that period was between -4 and $+2(\%)$ of the mean optimum moisture content determined in a laboratory.

The same sample was used for analysis of the results obtained for dry unit weights. The numerical analysis indicates that the average in-situ dry unit weight was $\gamma_d = 17.56 \text{ (kN/m}^3\text{)}$ which, compared with the optimum $\gamma_{opt} = 17.81 \text{ (kN/m}^3\text{)}$, gives the difference of $0.25 \text{ (kN/m}^3\text{)}$. The dispersion of results is evaluated as very low. The graphical presentation in Fig. 13 shows that in 92% cases the compaction of the clay built in during that period was over 95% of the modified Proctor test value (design building-in criterion).

The Fig. 14 shows a diagram of the referenced modified Proctor test for this group of results. Based on the above said, it may be concluded that the clay placement during this period was of high quality and in compliance with the design requirements.

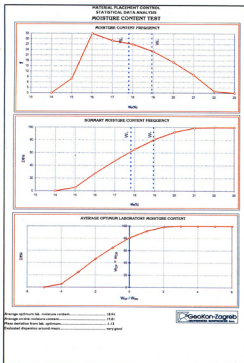


Figure 12. Statistical analysis of moisture content control and permanent test results during placement of the "new" clay in May 1995

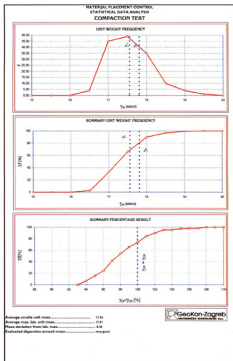


Figure 13. Statistical analysis of unit weight control and permanent test results during placement of the "new" clay in May 1995

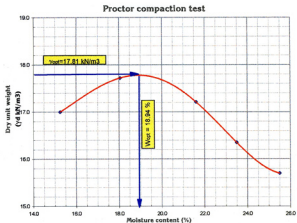


Figure 14. Referenced modified Proctor test for a set of "new" clay data

The mechanical characteristics of the built-in clay shall be tested after the construction has been completed, during drilling for installation of monitoring instrumentation.

ROCK MATERIAL BUILDING IN CONTROL

Clay Core Filter

Grain-Size Composition

The filter material grain-size composition was determined in compliance with the standard U.B1.018, and was used for classification, identification and determination of physical and

mechanical characteristics of the crushed rock material for construction of filter zones. The grain-size analyses were conducted using the dry sieving method on standard soil mechanics sieves with the minimum mesh opening of 0.10 mm.

During the remediation works, until the moment this paper is being prepared, over sixty tests have been conducted. The grain-size analysis of the samples has shown that the subject crushed material from the Dugobabe and Srinjine quarries meets the design requirements. During construction, the filter grain-size composition monitoring indicated minor deviations from the specified requirements in some batches delivered to the on-site spoil area. The filter grain-size composition had to be corrected. Such cases were corrected by the process engineer, on the site, and the delivered material was corrected with finer grain-size crushed material mixing with already laid filter material.

The enclosed grain-size diagram includes the grain-size composition limits set for the actually placed material. There is a clear deviation from the specified values. The average grain-size curve has also been given for all sievings, Fig. 15, and it shows that the average grain-size analysis for the filter material satisfies the requirements set for placement into the filter layers.

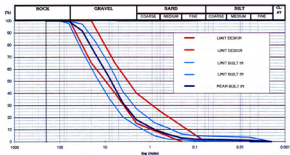


Figure 15. Filter material grain-size composition limits diagram
- design and as-built status



Photo 5. Backfilling downstream filter trench

Unit Weight

Depending on the technical conditions, the filter material was placed in approximately 50 cm thick layers, sprinkled during the dry period, and compacted with the vibratory roller (Binelli, Ghibli C100). Where the filter trench was narrower the large roller could not be used, so the small vibratory roller (Dispack, CG 11) or the vibratory compactor (Wacker, DS 72 Y) were used instead. The number of runs depended on a type of used compacting machinery. The analysis of samples taken from the trial section lead to the conclusion that the design compaction of the filter material is achieved in six runs over the treated surfaces using the compaction roller, five runs using the small vibratory roller, and four runs when the large vibratory roller is used. The said number of runs meets the requirement for compaction of 70% of maximum compaction achieved during the preliminary testing of filter material, $\max \gamma_c = 24.16 \text{ (kN/m}^3\text{)}$, $\min \gamma_c = 18.83 \text{ (kN/m}^3\text{)}$, $W_{opt} = 3.40(\%)$.

The unit weight tests were carried out according to the standard U.B1.012, and the test results were used for the filter zone layers compaction control.

The unit weight was tested in wet and dry state. So far, over two hundred and seventy tests were conducted.

The average wet unit weights were:

Left end, upstream: $\gamma_w=22.45$ (kN/m³), min. $\gamma_w=19.58$ (kN/m³), max. $\gamma_w=23.91$ (kN/m³)

Left end, downstream: $\gamma_w=22.69$ (kN/m³), min. $\gamma_w=20.54$ (kN/m³), max. $\gamma_w=23.84$ (kN/m³)

Right end, upstream: $\gamma_w=22.33$ (kN/m³), min. $\gamma_w=20.21$ (kN/m³), max. $\gamma_w=23.80$ (kN/m³)

Right end, downstream: $\gamma_w=20.90$ (kN/m³), min. $\gamma_w=16.97$ (kN/m³), max. $\gamma_w=22.46$ (kN/m³)

The average dry unit weights were:

Left end, upstream: $\gamma_d=21.86$ (kN/m³), min. $\gamma_d=19.13$ (kN/m³), max. $\gamma_d=23.34$ (kN/m³)

Left end, downstream: $\gamma_d=22.05$ (kN/m³), min. $\gamma_d=19.83$ (kN/m³), max. $\gamma_d=23.89$ (kN/m³)

Right end, upstream: $\gamma_d=21.76$ (kN/m³), najm. 19.91 (kN/m³), najv. 23.93 (kN/m³)

Right end, downstream: $\gamma_d=20.37$ (kN/m³), najm. 16.47 (kN/m³), najv. 21.88 (kN/m³)

Moisture Content

The sample moisture content at the spoil area and in-situ was determined according to the standard U.B1.012. The sample moisture content is evaluated in order to obtain the data required for optimum sprinkling and quality compacting of the filter material. To that end, the built-in layer was additionally sprinkled when the moisture content was not satisfactory (optimum).

For the in-situ filter samples the below **moisture content averages** were obtained:

Left end, upstream: $W_O=2.74(\%)$, min $W_O=1.16(\%)$, max. $W_O=5.14(\%)$

Left end, downstream: $W_O=2.86(\%)$, min $W_O=1.42(\%)$, max. $W_O=4.99(\%)$

Right end, upstream: $W_O=2.53(\%)$, min $W_O=1.51(\%)$, max. $W_O=4.43(\%)$

Right end, downstream: $W_O=2.62(\%)$, min $W_O=1.61(\%)$, max. $W_O=3.62(\%)$

Shell

The rock material was placed in maximum 1.0 m thick layers. The layers are rolled with vibratory rollers (Bittelli, Ghibli C100). The analysis of the unit weights achieved on the trial section leads to the conclusion that the requested compaction is achieved with minimum five roller runs over a layer.

Grain-size Composition

The filter material grain-size composition was determined in compliance with the standard U.B1.018, and was used for classification, identification and determination of physical and mechanical characteristics of the rock material for construction of upstream and downstream shells. The grain-size analyses were conducted using the vernier calipers for large diameter grains, and sieving for the grain size 40.0 cm and below.

Eighteen grain-size analyses were completed by the mid-July 1995. The rock material samples for the grain-size analyses were taken from the pits for measurement of the dam body shell unit mass.

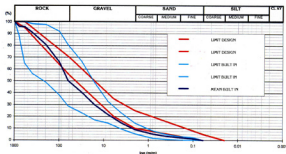


Figure 16. Shells material grain-size composition limits diagram - design and as-built status

The enclosed grain-size diagram in Fig. 16 shows that the grain-size curves for the tested samples correspond to a certain degree within the design limits. The analysis results lead to the conclusion that the deviations could have been caused by unintentionally unfavorable sampling. However, the soil mechanical and hydraulic characteristics of the newly build

shell sections are such that the function of the upstream and downstream shell constructed of available (tested) rock material shall not be imperilled.

Unit Weight

The unit weight tests were carried out according to the standard U.B1.012, and the test results were used for the upstream and downstream shell layers compaction control.

The unit weight tests were conducted in wet and dry state. During the remediation works on damaged sections and construction of the new parts of the shells fifty-two tests have been conducted.

The obtained wet unit weights were:

Left end, upstream: $\bar{\gamma}_w = 22.49$ (kN/m³), min. $\gamma_w = 21.09$ (kN/m³), max. $\gamma_w = 24.03$ (kN/m³)

Left end, downstream: $\bar{\gamma}_w = 22.41$ (kN/m³), min. $\gamma_w = 21.13$ (kN/m³), max. $\gamma_w = 24.17$ (kN/m³)

Right end, upstream: $\bar{\gamma}_w = 20.53$ (kN/m³), min. $\gamma_w = 19.61$ (kN/m³), max. $\gamma_w = 23.93$ (kN/m³)

Right end, downstream: $\bar{\gamma}_w = 22.19$ (kN/m³), min. $\gamma_w = 20.94$ (kN/m³), max. $\gamma_w = 23.92$ (kN/m³)

Downstream shell widening:

$\bar{\gamma}_w = 22.41$ (kN/m³), min. $\gamma_w = 19.26$ (kN/m³), max. $\gamma_w = 24.23$ (kN/m³)

The obtained dry unit weights were:

Left end, upstream: $\bar{\gamma}_d = 21.79$ (kN/m³), min. $\gamma_d = 20.34$ (kN/m³), max. $\gamma_d = 23.84$ (kN/m³)

Left end, downstream: $\bar{\gamma}_d = 21.53$ (kN/m³), min. $\gamma_d = 20.47$ (kN/m³), max. $\gamma_d = 22.98$ (kN/m³)

Right end, upstream: $\bar{\gamma}_d = 21.32$ (kN/m³), min. $\gamma_d = 19.63$ (kN/m³), max. $\gamma_d = 23.93$ (kN/m³)

Right end, downstream: $\bar{\gamma}_d = 21.23$ (kN/m³), min. $\gamma_d = 20.08$ (kN/m³), max. $\gamma_d = 22.78$ (kN/m³)

Downstream shell widening:

$\bar{\gamma}_d = 21.54$ (kN/m³), min. $\gamma_d = 19.64$ (kN/m³), max. $\gamma_d = 23.53$ (kN/m³)

The obtained data indicate the shell layers compacting was of good quality.

Moisture Content

Bearing in mind the design requirements, the layers were always sprinkled with 100 l of water per cubic meter of built-in material. The exceptions were the periods of extreme

rainfall, when additional sprinkling was not necessary, and the dry summer periods when the required quantities of water were not easy to provide.

The on-site sample moisture content was determined according to the standard U.B1.012. The sample moisture content is determined for evaluation of rockfill water saturation and quality compacting of the rock material.

The below on-site moisture contents were obtained for the soil samples:

Left end, upstream: $\overline{W}_O=4.33(\%)$, min $W_O=2.68(\%)$, max. $W_O=5.32(\%)$

Left end, downstream: $\overline{W}_O=4.05(\%)$, min $W_O=3.25(\%)$, max. $W_O=5.62(\%)$

Right end, upstream: $\overline{W}_O=4.20(\%)$, min $W_O=2.61(\%)$, max. $W_O=5.24(\%)$

Right end, downstream: $\overline{W}_O=4.55(\%)$, min $W_O=3.76(\%)$, max. $W_O=6.14(\%)$

Downstream shell widening: $W_O=4.00(\%)$, min $W_O=2.77(\%)$, max. $W_O=6.83(\%)$

CONCLUSION

The statistical analysis of 167 samples of clay built-in during September 1994, and 148 samples of clay built-in in May 1995 leads to the conclusion that the design requirements have been met.

The filter material placement control conducted on 270 samples has shown that the filter layers placement has also met the design requirements.

The shell material was controlled on 52 samples. The results have shown that no sample had the unit weight below the design 20 kN/m^3 .

DAM CORE GROUTING

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ZAGREB

ABSTRACT

The primary objective of the core grouting was filling in the cracks and cavities generated in core by the explosion in the dam, in order to prevent slurry losses and caving in of the cut-off wall trench sides.

The grouting was performed using the cement-bentonite slurry with the rheological characteristics and grouting pressure selected so as to enable consolidation of the majority of the clay core.

The results of the grouting confirmed the assumption that the major damage was caused in the central dam section. They also revealed that the damage was highly dispersed, unrelated, and of local character, both lengthwise and heightwise the dam. This confirms correctness of the accepted remediation alternative which includes construction of the plastic concrete cut-off wall as the imperious dam element in those dam sections where it is not reconstructed.

The core grouting completely fulfilled its function since the cut-off wall was constructed without any problems with stability of the sides during the trench excavation.

The core was consolidated by grouting, and together with the cut-off wall it shall be both impermeable and static dam element transmitting the loads to the downstream shell.

GROUTING PURPOSE

The accepted solution of the cut-off wall construction in the damaged clay core provides for impermeability of a 261.15 m long central dam section, namely of about 60% of its total length. The cut-off wall varies in depth between 12.9 and 51.5 m, it is 1 m thick and its surface area is 8,505 m².

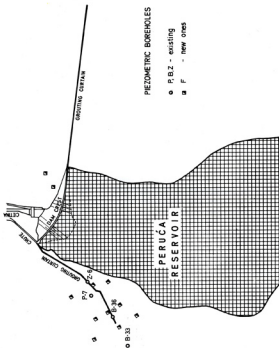
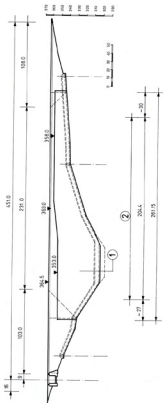


Figure 1. Layout



KEY

1. CUT-OFF WALL
2. GROUTED CORE SECTION

Figure 2. Longitudinal section

Since the stability of the trench sides is secured with slurry, there was a real risk of loosing the slurry through the explosion-induced cracks in the clay core and caving-in of the trench. Therefore, it was decided to fill the cracks in the dam section in which the cut-off wall was to be constructed with the cement-bentonite slurry, and thus consolidate the subject clay core segment, lower the slurry loss risks, increase the shear resistance and consequently improve the cut-off wall construction safety.

The plan was to perform the clay core grouting on about 204 m long stretch, namely on about 78% of the cut-off wall length, Figs. 1 and 2. The grouting was not planned in the left and right end, on 27 and 30 m length, respectively, because the dam remediation in these segments was planned to include excavation and backfilling, so on the half of the surface the cut-off wall was to be integrated into the newly constructed dam core.

Further, the analysis of the dam settlement monitoring indicated that the dam entered the stage of secondary consolidation. The settlement in the left cut-off wall end during the previous three months of monitoring had totalled 2 to 3 mm, namely only about 0.005 % of the dam height, while no settlement was recorded in the right dam end during that



Photo 1. Clay core at the deepest excavated part of the left dam end with destroyed gallery

period. With respect to these data, it could have been concluded that there were no major cracks in these dam core stretches, therefore no extensive slurry losses should have been expected and the cut-off wall construction should have been carried out under normal conditions.

After excavation works for the dam reconstruction have been completed in its left and right ends, the above assumption was confirmed since the external visible core surfaces were compact and no cracks were noticed which would cause slurry loss, Photo 1.

GROUT SLURRY CHARACTERISTICS AND GROUTING PRESSURE

A stable grout slurry has the characteristics of the Bingham fluid. With it, the grouting extent does not depend only on the crack width and grouting pressure, but also on the initial tangential resistance of slurry, " τ_0 ", which has to be suppressed if fluid is to start flowing. The slurry flow depends on its dynamic coefficient of viscosity, " η ".

Since the grouting was aimed at core consolidation, the rheological characteristics of the slurry should have been such that the grouting extends to the majority of core at the pressure which approximately corresponds with the effective vertical stresses in the dam core. This grouting pressure has been selected because of the fear that higher pressure could disturb the balance of disintegrated clay core achieved after blasting of explosive. Additionally, higher grouting pressure could cause leakage of slurry through larger core cracks into the filter layers and dam rockfill shell. This would have caused difficulties in defining the completion of the grouting process against the common criteria of the slurry consumption at a specified pressure during a given period of time.

The planned grout slurry consisted of 80% cement and 20% bentonite, diluted with water at the mix ratio (dry matter:water, by weight) of 1:2.5 to 1:2.

The Portland cement characteristics:

density	$\rho = 3.12 \text{ kg/dm}^3$
Blaine value	3,508 cm^2/g
setting, start	140 min
setting, end	220 min
compressive strength after 28 days	44.8 MPa

The bentonite characteristics:

density	$\rho = 2.587 \text{ kg/dm}^3$
liquid limit	$W_L \text{ \%} \dot{C} = 514$
plastic limit	$W_p \text{ \%} \dot{C} = 41.55$
index of plasticity	$I_p = 472.45$



Photo 2. Clay core grouting

The laboratory tests of rheological and physical/mechanical characteristics of the slurry at the mix ratio (dry matter:water) of 1:2.5 and 1:2, have given the below results:

mix ratio (dry matter:water)		1:2.5	1:2
Marsh viscosity [s]	ϕ 4 mm	42	∞
	ϕ 10 mm	7.8	16.6
Fannell dynamic viscosity	η [mPas]	8	18.0
Initial tangential resistance	τ_0 [Pas]	16.5	57.0
Gel strength [Pa]	after 5 sec	8.5	22.5
	after 10 min	12.5	30.0
Decantation volume	after 5 min	0	0
	after 240 min	0	0

Density	ρ [kg/dm ³]	1.235	1.286
Setting, start	[hours]	18	18
Setting, end	[hours]	30	27
Compressive strength after 28 days [Mpa]		0.9	1.57

It was planned to perform grouting in 5 m high borehole sections, the deepest section being on the depth of 45 to 50 m. At unit weight of core material between 16 and 17 kN/m³, the effective vertical stress at this depth is approximately 800 kN/m². Maximum grout slurry extent radius is calculated from the Wittke equation

$$\max R = \frac{\sigma}{\tau_0} p_{\max} + r_0$$

where

$\max R$ [m]	-	maximum slurry extent radius
a [m]	-	crack width
τ_0 [Pa]	-	initial tangential stress of slurry
$p_{\max}$ [Pa]	-	maximum grouting pressure
r_0 [m]	-	borehole radius



Photo 3. Clay core grouting

The maximum grout slurry extent radius is 4.88 m and 48.51 m for the crack widths $a = 1 \cdot 10^{-4}$ m and $a = 1 \cdot 10^{-3}$ m, respectively, when the mix ratio is 1:2.5, $\nu_0 = 16.5$ Pa, grouting pressure $p_{\text{gr}} = 800$ kPa, and borehole diameter 60 mm. When the slurry of the highest density with the mix ratio of 1:2 is used, the initial tangential stress is $\tau_0 = 57$ Pa, and the pressure and borehole diameter are the same, the maximum grouting extent radius for the crack widths $a = 1 \cdot 10^{-4}$ m and $a = 1 \cdot 10^{-3}$ m is 1.43 and 14.06, respectively. Since the clay core width in the lowest dam segment is about 23 m, the obtained results are considered satisfactory.

For the top grouting sections, at the depth of 1 to 5 m, the assumed grouting pressure is approximately 100 kN/m². At this pressure, the maximum grout slurry extent radius for mix ratio 1:2.5 is 0.61 m for the crack width $a = 1 \cdot 10^{-4}$, and 6.11 m for the crack width $a = 1 \cdot 10^{-3}$. These results have also been considered acceptable, although the core width at this elevation is about 8 m. However, the cut-off wall axis is set 4 m upstream the dam axis, therefore the grout slurry extent at the assumed pressure is sufficient.

The obtained results have proven that the rheological and physical/mechanical characteristics of the grout slurry of the foreseen composition and the mix ratio (dry matter : water), at the grouting pressure ranging between 100 - 800 kN/m², are satisfactory. Based on these data, the triangular boreholes arrangement was designed, with 1.4 m borehole distance in the dam axis direction, Fig. 1, which corresponds with the grout slurry extent radius for the mix ratio 1:2 and the crack width $1 \cdot 10^{-4}$ m for the highest dam section on the foundation plane.

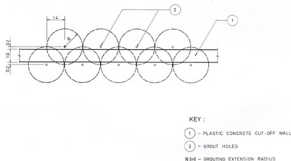


Figure 3. Grouting boreholes arrangement

GROUTING CRITERIA

It was planned to perform the grouting in 5 m high sections in upwards direction. Within the range of pressures for the highest and the deepest borehole sections of 100 kN/m^2 and 800 kN/m^2 , respectively, the grouting pressures have been determined for the interim sections relative to the effective vertical stress in the core, curve 2, Fig. 2, and to the pressure of the grout slurry with mix ratio of 1:2.5 and density of 1.235 kg/dm^3 , curve 1, Fig. 2. This has been achieved so that the grout slurry pressure was increased using the grouting pump, i.e. to the borehole depth of 15 m for 50 kN/m^2 , to the depth of 30 m for 100 kN/m^2 , to the depth of 45 m for 150 kN/m^2 , and to the depth of 50 m for 200 kN/m^2 , curve 3, Fig. 4.

The grouting completion criteria were defined so that each section was to be grouted under the specified pressure until the grout slurry consumption fell to 0 l in 20 min, namely until the full saturation, or if this had not been achieved, to the slurry consumption of maximum 20 l in 20 min. In case the grouting activity had been completed at the consumption of $\leq 20 \text{ l}$ in 20 min, the grouting of the upper section should have been conducted at the pressure used for the preceding lower section.

In addition to the primary, the secondary boreholes were planned. Due to the lack of experience with this type of works, the criteria for the secondary boreholes were not specified. They were planned for the grouting sections where the grout slurry consumption was so high that it could have been supposed that all cracks were not filled.

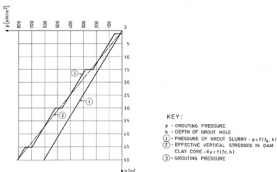


Figure 4. Grouting pressure v.s. depth of grout hole

GROUTING RESULTS ANALYSIS

Along the 204 m long grouting route 147 primary boreholes 4,927 m long, and 24 secondary boreholes 861 m long, namely totally 171 boreholes 5,788 m long were made. The total consumption of the grout slurry was 591,156 l, and the average consumption was 102.13 l/m of borehole. Since 1 l of slurry contains 0.35 kg of dry matter, about 207 t of cement and bentonite was used for grouting.

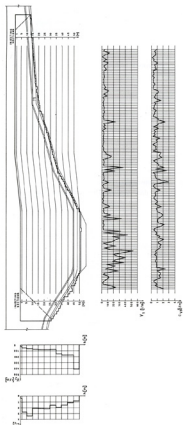
Based on the average consumption of the grout slurry in the boreholes, V_1 $\bar{c}_1$ /m of borehole⁶, Fig. 5, the grouted area may be divided into the **highest central and the remaining part of the dam**.

Although the average grout slurry consumption per 1 m of the borehole varied in about 50 m long **central dam section** between $V_1 = 13.48$ l and $V_1 = 534.45$ l, the prevalent consumption was exceeding 100 l per meter of borehole as recorded on about 70% of all boreholes. The slurry consumption variation coefficient as per individual sections, " c_{v1} ", was for these boreholes about 1.0, which indicates comparatively uniform consumption of slurry in borehole sections. This is evident from the review of the grout slurry consumption as per the dam sections, Fig. 6.

For the borehole with consumption $V_1 = 534.45$ l/m, the consumption as per section varies between 75 l/m at the depth of 1 to 5 m, and 1,291 l/m at the depth of 30 to 35 m, with the coefficient of variation $c_{v1} = 0.7$. Such consumption is the consequence of large dam core damages, as confirmed by the measurement of settlements after the explosion which in this section were about 1.55 m, namely 27% of the dam height.

For the borehole with the lowest average consumption of $V_1 = 13.48$ l/m, the consumption per sections varies from 6 l/m at the depth of 5 to 10 m, to 30 l/m at the depth of 40 to 45 m, while the coefficient of variation for this borehole is only $c_{v1} = 0.53$. The measured settlements in this part were about 0.40 m or 7% of the dam height.

The average consumption in the **remaining grouted part of the dam core** is considerably lower, and the transition is discontinuous. The average grout slurry consumption exceeds 100 l/m on 12% of boreholes only. The maximum consumption is $V_1 = 370.24$ l per meter of borehole and the coefficient of variation is $c_{v1} = 2.24$. However, the average consumption in this borehole is the result of an exceptionally large consumption of the grout slurry in the borehole section from 5 to 10 m, i.e. 2,329 l/m, this being the maximum consumption on a single section along the entire grouting route. Since the consumption at the remaining six sections of this borehole ranges between 12 and 35 l/m, these data explain the coefficient of variation $c_{v1} = 2.24$.



KEY:

- V_g [m³/m²] - mean injected grout volume per meter wall
- σ_g [m³/m²] - standard deviation
- V_{g1} [m³/m²] - mean injected grout volume per meter wall
- σ_{g2} [m³/m²] - standard deviation

Figure 5. Mean injected grout volume

Such an analysis is fully applicable to any other borehole within the entire grouted dam section in which high average consumption of the grout slurry and high coefficients of variation have been recorded. They are the result of extremely high consumption of the grout slurry in only one or two grouting sections on a borehole, while the consumption in other borehole sections is considerably lower.

The conclusions based on this analysis are confirmed by the data on the grout slurry consumption in particular borehole sections, Fig. 6.

The minimum average grout slurry consumption was also recorded at this part of the grouting route, i.e. $V_1 = 11.33$ l per meter of borehole, with the coefficient of variation $c_1 = 1.17$. The values of the average consumption as per individual borehole section range between 4 and 63 l/m, but eight out of ten sections have the consumption below the average.

The grouting results lead us to the conclusion that the dam core in this part suffered considerably lower damage than the central section. Although the sections with comparatively high grout slurry consumption were recorded, they are not interconnected as was the case in the central dam section. The settlement data also confirm more moderate damage compared to the central section. The settlements ranged from 0.03 m or 1‰ of the dam height in the flanks, to 0.53 m or 9‰ of the dam height in the joint with the central section.

The results of the analysis of average grout slurry consumption as per the sections on the same grouting depths, V_2 Šl/mČ, Fig. 5 indicate comparatively small changes in consumption relative to the section depth for the sections from 1 to 30 m of depth, i.e. $V_2 \approx 50$ l/m for the upper section, and $V_2 \approx 80$ l/m for the section from 25 to 30 m. However, the coefficient of variation of the average consumption varies from $c_2 = 1.58$ to $c_2 = 3.44$. This is the consequence of the extremely high consumption of the grout slurry in particular sections, while the consumption in other sections at the same grouting height is considerably smaller, Fig. 6.

With increase in depth, the average consumption per grouting section increases and varies between 155 and 175 l/m for the depths of 30 to 45 m. The average consumption for the deepest section of 45 to 50 m is $V_2 = 415.04$ l/m.

The coefficients of variation decrease with the increase in the depth. For the grouting section at the depth of 45 to 50 m it is $c_2 = 0.86$. This is the consequence of more uniform average consumption of the grout slurry in individual boreholes.

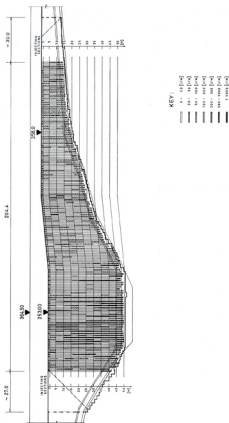


Figure 6. Injecting sections consumptions

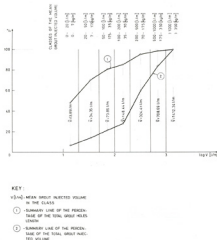


Figure 7. Classified mean volume of injected grout

Although it is difficult to make general conclusions based on the average values, the increase in average consumption of the grout slurry with the grouting depth, accompanied by the decrease in coefficient of variation, shows that the dam clay core damage increases with the depth the maximum being in the lowest central dam section. This should have been expected, since the largest quantity of explosive was probably blasted in this grouting gallery dam section.

The dispersion and the dam core damage level are well illustrated by the grout slurry consumption as per sections which have been classified according to the average consumption, Fig. 6, as well as by the share of particular classes within the total borehole length, curve 1, Fig. 7, and the total grout slurry consumption, curve 2, Fig. 7.



Photo 4. Gallery reconstruction with steel arches, shotcrete and grouting - detail

The diagram in Fig. 7 leads to a conclusion that the average consumption at 81% of the total borehole length was less than 100 l/m, namely 35 kg/m of dry matter, with the share in the total quantity of consumed grout slurry of 22%.

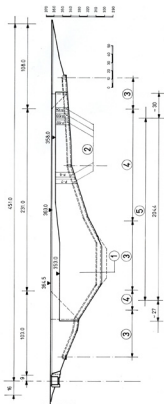
The average consumption of 100-500 l/m, namely 35-175 kg/m of dry matter is recorded on 15% of the total borehole length on which 38% of the total grout slurry was consumed.

On the remaining 4% of the borehole length, with the average consumption exceeding 500 l/m or 175 kg/m of dry matter, over 40% of the grout slurry was used.

CORE GROUTING SUCCESS WITH RESPECT TO THE CUT-OFF WALL CONSTRUCTION

No problems related to the abrupt slurry loss arose during the cut-off wall construction, only its seepage was somewhat more intensive in the gallery, in the sections marked in Fig. 8.

This should be attributed to the below.



KEY

1. CUT-OFF WALL
2. SLURRY SEEPAGE POINTS
3. GALLERY RECONSTRUCTION BY NEW CONCRETE LINING
4. GALLERY RECONSTRUCTION BY STEEL ARCHES, SHOTCRETE AND GROUTING
5. GROUTED GALLERY SECTION

Figure 8. Sections of the slurry seepage in the gallery

In the subject sections, the gallery reconstruction included supporting of the old cracked concrete lining with steel arches, placement of a layer of shotcrete with double steel mesh reinforcement, and grouting with the cement-bentonite suspension behind the concrete lining for filling in of cracks and cavities and realization of the impervious joint between the lining and core clay, namely bedrock.

The cut-off wall construction activities related to the gallery reconstruction did not include grouting behind the gallery lining, so the cracks and cavities in the old concrete lining were only partly filled in from inside the gallery, applying the layer of the shotcrete.

Since the slurry loss was recorded during the trench excavation in the bedrock for the cut-off wall it was assumed, based on the analysis of the gallery reconstruction works completed so far, that the loss was caused by the dynamic load of the hydraulic trench cutter on the bedrock and partly reconstructed gallery concrete lining. The hydraulic trench cutter damaged the bedrock, the base joint of the bedrock and clay core, while the cracks in the gallery concrete lining reopened after being partly closed with the shotcrete.

The hydrostatic pressure of the slurry in the cut-off wall trench was sufficiently high to cause the slurry seepage into the gallery through the contact surfaces between the bedrock and clay core, the gallery concrete lining and the bedrock, namely clay core, and through the reopened cracks and voids in the gallery concrete lining, Fig. 9.

In compliance with the above assumption, grouting was started behind the gallery concrete lining in order to completely fill in the cracks and cavities and realize the impermeable joint along the entire outside perimeter with the clay core and bedrock.

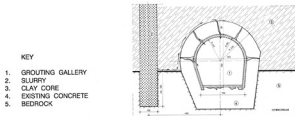


Figure 9. Slurry seepage in the partially reconstructed gallery

The excavation for the cut-off wall trench followed the gallery lining grouting with no problems related to the slurry loss. This confirmed the assumptions about the reasons for the slurry penetration into the gallery.

No caving in of the trench sides was noticed during performance of works, therefore it can be concluded that the dam core grouting in the cut-off wall construction site completely fulfilled its function both regarding slurry loss prevention and improvement of the trench stability.

CONCLUSIONS

The grouting results confirmed the assumption drawn from the settlement monitoring that the major dam core damage was caused in its top central section.

However, the data on the grout slurry consumption indicate highly dispersed, unrelated and local character of damage both lengthwise and heightwise the dam.

This particularly relates to the right part of the grouted site where the extremely high grout slurry consumption was recorded in the borehole sections in the vicinity of which the consumption was considerably low. The lower consumption was not recorded only in the adjacent boreholes, but also in the sections of the same borehole immediately below or above the section of high consumption.

This fact confirms that the selected design solution, with the plastic concrete cut-off wall as a new impermeable element for the dam section which is partly or completely not reconstructed, was correct.

Since the cut-off wall construction was completed with no major difficulties with respect to the slurry losses and practically no difficulties with trench stability, the core grouting completely fulfilled its purpose. The grouting additionally consolidated the core which, together with the cut-off wall, shall be a reliable impermeable and static dam element transmitting the load to the downstream shell.

REFERENCES

- [1] "Filling materials for watertight cut-off walls", ICOLD Bulletin 51, International Commission on Large Dams, Paris, France, 1985
- [2] Lombardi, G., "Grouting design and control using the GIN principle", Water Power & Dam construction, June 1993
- [3] Nonveiller, E., "Grouting - Theory and Practice", Školska knjiga Zagreb, 1989

PLASTIC CONCRETE CUT-OFF WALL CONSTRUCTION

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ABSTRACT

The plastic concrete cut-off wall was constructed within the Peruća Dam clay core. It is 261.15 m long, between 12.9 and 51.5 m deep, and total surface area is 8,505 m².

The design mechanical characteristics of the plastic concrete are: 28-day compressive strength $> 1 \text{ MN/m}^2$, Young's modulus of elasticity $\leq 400 \text{ MN/m}^2$, Darcy's coefficient of filtration $< 10^{-4} \text{ m/s}$. These characteristics were met by the plastic concrete of the below composition per 1 m³: 112 kg of cement, 35.6 kg of bentonite, 102 kg of rock flour, 1,527 kg of aggregate with variable grain size (0-4 mm), and 337 l of water.

The cut-off wall trench was excavated with grab and hydraulic trench cutter, and the concrete was placed by underwater concreting using the contractor procedure.

The cut-off wall was constructed in four months.

INTRODUCTION

The Peruća Dam remediation project included construction of a plastic concrete cut-off wall within the central dam section. The cut-off wall was intended to take over the function of an impermeable dam element instead of the damaged clay core. The cut-off wall was constructed between the stations 0+87.70 and 3+44.00, along the dam axis.

The joint between the reconstructed dam sections in the left and right end and the section which includes the cut-off wall as an impermeable element is designed so that the cut-off wall is min. 2 m long structure incorporated in the clay core of the reconstructed sections.

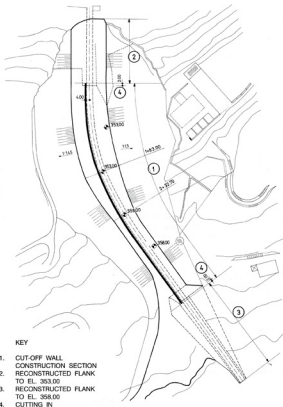


Figure 1. Dam layout with cut-off wall



Figure 2. Dam longitudinal section along the cut-off wall axis

Since it is designed so that this joint is in the slope with ratio 1:1, the said criterion relates to the lowest reconstructed sections.

Vertical to the dam axis, the cut-off wall is placed 4.0 m downstream the grouting gallery axis. The cut-off wall is 1.0 m wide.

Maximum design cut-off wall depth is 50.4 m, and minimum 12.90 m. According to the design, the cut-off wall was to be minimum 2 m cut into the bedrock, namely the concrete slab in the central dam section, Fig. 3.

Based on the calculations for the cut-off wall behavior under various loadings the below mechanical characteristics of the cut-off wall material were specified by the design:

- compressive strength $> 1 \text{ MN/m}^2$
- Young's modulus of elasticity $< 400 \text{ MN/m}^2$
- impermeability coefficient $< 10^{-8} \text{ m/s}$.

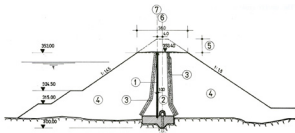
In the dam section where 10 m high part was to be removed, the cut-off wall was constructed from the operating platform on El. 353.40, and where 5 m high part was to be removed the works were carried out from the platform on El. 358.40. In the transitional ramp, between the elevations 353.40 m a.s.l. and 358.40 m a.s.l., the cut-off wall is step-like, and the steps are 1 m high, Figs. 1 and 2.

The operating platform consists of a 40 cm of compacted rock material. The clay core is protected with geotextile and thus separated from the gravel layer.

The reinforced concrete guide walls, used as the guides for the excavation equipment, are 25 cm wide and 1.25 m high. The guide walls shall remain permanently built in the dam body, Fig. 4.

The cut-off wall trench is mainly excavated in the damaged dam clay core. The excavation in some segments in the top several meters encompassed the upstream filter layer. To prevent the slurry losses through the core cracks during excavation, the consolidation grouting was performed along the cut-off wall trench route prior to excavation works.

In order to provide for the contact between the cut-off wall and the bedrock, to and achieve the best impermeability, the cut-off wall joint with the bedrock and the gallery concrete was to be grouted and a bond realized with the existing grout curtain. The grouting works had been planned to be carried out from the reconstructed inspection gallery.



KEY

1. CUT-OFF WALL
2. CLAY CORE
3. FILTER LAYERS
4. ROCK FILL
5. RECONSTRUCTED DAM PART CREST
6. DAM AXIS
7. CUT-OFF WALL AXIS

Figure 3. Typical dam cross-section with cut-off wall

KEY

1. CLAY CORE
2. CUT-OFF WALL
3. GUIDE WALLS
4. OPERATING PLATFORM
5. GEOTEXTILE

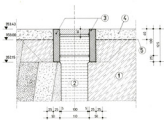


Figure 4. Cut-off wall top section with guide walls - detail

The quality control included the following activities:

- the preliminary testing of the slurry and fill component materials: cement, bentonite, water, sand, gravel, the slurry itself, green and hardened plastic concrete and concrete for guide walls;
- construction testing and control: trench plumbness control, aggregate moisture content, material dosing in a concrete mixing plant, plastic concrete 7-, 28- and 90-day compressive strength, tests of consistency, impermeability, erosivity, and guide wall concrete; samples are taken for a particular quantity of built-in cut-off wall material;
- completed cut-off wall testing, in two stages:

first stage: the cut-off wall integrity is tested: geometry, homogeneity of placed material, quality of joints between individual elements and mechanical characteristics of placed material, i.e. compressive strength and impermeability; the boreholes are drilled in the cut-off wall body and testing samples taken;

second stage: the cut-off wall impermeability is tested while the dam is under full hydrostatic pressure; the permanent impermeability monitoring system is used.

The Impermeable Cut-off Wall design was prepared by Elektroprojekt Consulting Engineers Zagreb, 1994.

PRELIMINARY TESTS AND FINAL SELECTION OF THE CUT-OFF WALL CONSTRUCTION PROCEDURE AND MATERIAL COMPOSITION

The cut-off wall design specified that the Contractor shall detail the selected construction procedure in the Construction Design. The selection was to be made between a two-stage or single-stage procedure, and the cut-off wall material composition was to be determined. Plastic concrete for the two-stage procedure and self-hardening cement-bentonite suspension for the single-phase procedure.

The Geotechnika-Bauer Ltd. joint venture was appointed as Contractor. The company was established solely for the Peruća Dam Remediation Project by Geotechnika Zagreb and Bauer Spezialtiefbau, Schöbenhausen, Germany.

The Contractor selected the two-stage procedure, excavation followed by plastic concrete placement. The procedure was expected to render the design mechanical characteristics of the cut-off wall material.

The drawback of the single-phase procedure is that it requests trench with the self-hardening slurry that can hardly maintain the required viscosity during the excavation, which could last up to 3 days, and still meet the design quality requirements after hardening.

All preliminary laboratory quality control of the component material and determination of optimum plastic concrete mix were conducted by Geoexpert-IGM Zagreb, with expert assistance and in cooperation with the Bauer Spezialtiefbau experts. The preliminary testing of the following component materials was carried out: cement PC 30z 45 S, produced by Dalmacijacement, crushed aggregate and rock flour from the Srinjine quarry, Bentoplast bemonite produced by INA-Petrokemija Kutina, and the Cetina river water samples. All tested component materials were suitable for the plastic concrete mixing.

The slurry is prepared of 11.1% suspended bentonite, and with the mix ratio (bentonite: water) of 1:8.

REV. RATE	1	2	3	4	5	6	7
COMPONENTS							
CEMENT	50	100	150	200	250	300	350
SAND	50	100	150	200	250	300	350
GRAVEL	50	100	150	200	250	300	350
ADDITIONAL	50	100	150	200	250	300	350
WATER	1	2	3	4	5	6	7
WATER-CEMENT RATIO	1	2	3	4	5	6	7
GREEN PLASTIC CONCRETE CHARACTERISTICS							
CONSISTENCY	1	2	3	4	5	6	7
PLACEMENT	1	2	3	4	5	6	7
HARDENED PLASTIC CONCRETE CHARACTERISTICS							
COMPRESSIVE STRENGTH	1	2	3	4	5	6	7
COMPRESSIVE STRENGTH	1	2	3	4	5	6	7
TENSILE STRENGTH	1	2	3	4	5	6	7
MODULUS OF ELASTICITY	1	2	3	4	5	6	7
COEFFICIENT OF THERMAL EXPANSION	1	2	3	4	5	6	7

Figure 5. Plastic concrete testing results

The preliminary testing of plastic concrete resulted in the optimum ratio of individual components. The tests were carried out on mixes of differently dosed concrete, bentonite and aggregate. Seven different mixes were subject to testing of fresh mass consistence, 7- and 28-day compressive strength, density, modulus of elasticity, and erosivity, Fig. 5.

Based on the tests on green and hardened fill, the following mix ratio for 1 m³ of plastic concrete was selected (mix ratio No. 2 from the Table):

cement	112 kg
bentonite	35.6 kg
rock flour	102 kg
aggregate 0-4 mm	1,527 kg
water	337 l
water-cement ratio	3.01
green mass consistence (slump test)	52-60 cm

When the mix is prepared in situ the component dosing should be modified according to the material characteristics and weather conditions.

SITE ORGANIZATION

The majority of the slurry and plastic concrete mixing facilities is placed on the operating platform on El. 358.0, upstream the dam axis, in the right flank. Here, the slurry mixing plant was set, with the bentonite storage, slurry desanding plant with tanks for fresh, trench and waste slurry, and the slurry pump which conveys the slurry to the excavation site.

The slurry mixing plant was also used for mixing of plastic concrete bentonite suspension which was conveyed to the agitator, and further to the concrete mixing plant. The cut-off wall plastic concrete was prepared in Konstruktor Split 60 m³/h mixing plant.

The slurry mixing and desanding plants were connected with the slurry piping set along the dam to the excavation and cut-off wall concreting sites. As the work progressed, the piping was extended.

The hydraulic trench cutter, grab and crawler crane worked in the cut-off wall trenching site. The material excavated by grab was removed by trucks to the spoil area in the right flank, same as the solid material from the slurry desanding plant.

The water necessary for slurry and concrete mixing was pumped from the Cetina, downstream the power house, into the tank, and furthered to the mixing plant.

The power supply was from the main on-site transformer, Fig. 6.

The cut-off wall construction works were conducted from the right to the left dam end.

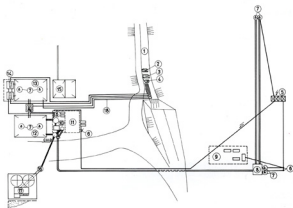


Figure 6. Site organization

KEY

- | | |
|----------------------------------|--|
| 1. OPERATING PLATFORM ON THE DAM | 10. SLURRY MIXING PLANT |
| 2. GRAB C-50 | 11. BENTONITE STORAGE |
| 3. HYDRAULIC TRENCH CUTTER BG-30 | 12. FRESH SLURRY TANK |
| 4. CRANE BS-108 | 13. TRENCH SLURRY TANK |
| 5. MAIN 630 KVA SUBSTATION | 14. SLURRY DESANDING PLANT |
| 6. POWER CABINET | 15. WASTE SLURRY TANK |
| 7. PUMPS AND WATER SUPPLY SYSTEM | 16. BENTONITE SUSPENSION AGITATOR |
| 8. WATER TANK | 17. 60 M ³ /H CONCRETE MIXING PLANT |
| 9. SITE OFFICES | 18. SLURRY CONVEYANCE PIPING |

CUT-OFF WALL TRENCH EXCAVATION

Excavation Machinery

The excavation was performed by two types of machinery: kelly-bar grab and hydraulic trench cutter.

The kelly-bar grab KRC2 on a crawler C-50 was used only for preliminary cut-off wall trenching to the depth of 3-4 m, rarely to 10 m, in order to form the space for the slurry circulation during the hydraulic trench cutter excavation and achieve the depth needed for submerging of the cutter centrifugal pump used for removal of the excavated material. The grab was not used for deeper excavation because it does not allow maintenance and control the trench plumbness on greater depths.

Therefore, the majority of excavation works was performed using the BC 30 hydraulic trench cutter on a HS 870 crawler crane, with a HDS 80 winch system suspending cables and hoses, manufactured by Bauer, Photo 1. The basic elements of this system are the crawler crane and the trench cutter (excavator).

The crawler crane is specially designed to support the cutter. It is driven by a 430 kW hydraulic Diesel motor. The crane is aligned so as to move in parallel with the cut-off wall trench, Fig. 7.

The cutter has a heavy steel frame, 15.4 m high, 2.8 m long and 2.4 m wide, two hydraulic motors driving two cutting wheels, and centrifugal pump with the excavated material removal hoses. The trench cutter weight is about 32 tons.

The hydraulic motors rotate the cutter wheels in opposite directions. The wheels fragment and loosen the material and mix it with the bentonite suspension which is in the trench. The centrifugal pump sucks the mixed slurry and excavated material and conveys it by the piping to the slurry desanding plant, Fig. 8.

The cutter performance depends on its power which results from its weight (thrust force) and the wheel torque (2x81 kNm). These components are interrelated - when their H/V ratio (H-horizontal component, V-vertical component) is high, the wheels turn but do not penetrate into the soil. When the H/V ratio is low, the wheel teeth cut into the rock or soil, but do not move, Fig. 9. The optimum BC-30 cutter output is achieved by fine balance of the H/V ratio.

The cutter can reduce the rock material size to 30-40 mm. Special dynamic effect devices are used to crush the rock into fragments sufficiently fine to pass through the pump inlet. For that reason the wheel speed can be increased to 45 rpm. The wheels are fitted with elastic flexible shock absorbers protecting the teeth against failure.

KEY

1. CRANE
2. CUTTER WHEEL
3. WINCHES SUSPENDING CABLES AND HOSES

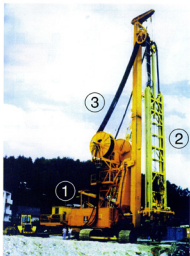


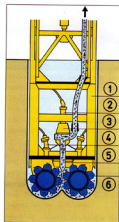
Photo 1. Hydraulic trench cutter BC-30

To avoid that the trench bottom becomes grooved, the tip-side teeth were used, so the complete trench bottom was removed, Fig. 9.

The trench plumbness is ensured by two separate vertical adjustment devices - the rough adjustment is conducted using the hydraulic winch of the main crane, and fine adjustment using the hydraulic cylinder inside the cutter. This enables precise control of the cutter teeth loading.



Figure 7. Trench cutter position and movement during excavation



KEY

1. STEEL FRAME
2. HYDRAULIC MOTOR
3. CENTRIFUGAL PUMP
4. HYDRAULIC MOTOR
5. PUMP INLET
6. CUTTING WHEELS

Figure 8. Trench cutter elements

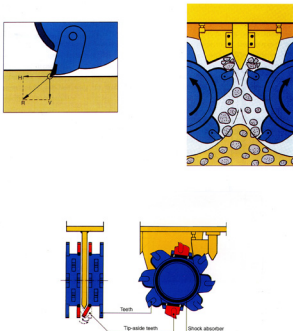
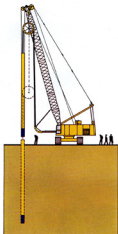


Figure 9. Cutting wheels - detail

In order to achieve the trench plumbness the cutter is suspended so as not to induce the forces which could affect the trench excavation. Automatically controlled cables and hoses maintain the verticality of the resultant of all forces acting on the cutter. Additionally, the cutter is fitted with electronic inclinometer which controls the cutter deviations from the vertical axis in two vertical directions to 0.2%, Fig. 10.

Figure 10.
Cutter suspending method



Slurry Mixing, Conveyance and Desanding System

The bentonite suspension has a double role during the cut-off wall trench excavation. First, it holds up the trench walls preventing the loss of equipment, its trapping and stoppage of works. Second, it enables hydraulic conveying of excavated and broken material from the excavation site to the desanding plant.

The slurry mixing, conveyance and desanding is shown in Fig. 11.

Slurry mixing plant 1 consists of two bentonite mixers (BM 1000 and BM 3000) with total capacity of 4,000 l. The prepared mix is conveyed by the pipes into the fresh slurry tank 2 equipped with pumps for slurry circulation in the tanks. The main pump 4 conveys the fresh slurry into the cut-off wall trench. Saturated with the trench spoil, clay and pieces

of rocks and concrete, the slurry is conveyed by the cutter centrifugal pump to the slurry desanding plant 5, type BE 500, with desanding capacity 500 m³/h. The solids from the trench are separated 6 and provisionally dumped near the plant to be later removed by truck to the spoil area, and the regenerated slurry, enriched with the particles of clay from the trench, is conveyed to the trench slurry tank 3. From here, the slurry is repumped into the piping and conveyed to the excavated trench. The trench slurry is permanently enriched with the fresh slurry repumped from the tank 2 to maintain the specified characteristics.

During the cut-off wall concreting, the concrete is first placed on the trench bottom, and the slurry is displaced towards the surface with increase in the concrete level. Once on the surface, the slurry is pumped back to the desanding plant and into the tank 3. After a number of cycles, the slurry loses its specified characteristics, gets saturated with clay particles and concrete components. Such slurry is transported into the waste slurry tank 7 and removed from the system by piping 8.

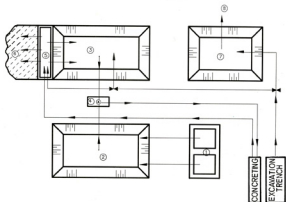


Figure 11. Slurry mixing, conveyance and desanding system

The slurry desanding plant, Fig. 12 and Photo 2, consists of two identical units functioning as an integrated plant.

The slurry saturated with the trench spoil, clay, particles of rock and concrete, reaches the intake pipe 1 and vibrating sieve 2 which separates fractions above 5 mm and chunks of clay. The sieved slurry is diverted to two identical desanding plants, which increases the desanding capacity. The slurry is forwarded to the tank 3 to be repumped 4 into the cyclone 5 where the fine particles are separated from the slurry. Through the cyclone outlet 6 the particles are released onto the vibrating sieve 7 separating them from the rest of the slurry.

The desanded slurry is taken from the cyclone over the spillway 8 into the tank 9, and hence into the trench slurry tank. The automatic float 10 controls the tank level and keeps it constant through the desanding process.

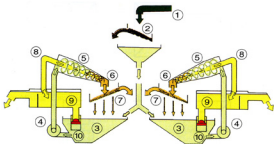


Figure 12 Slurry desanding plant BE 500 operation scheme



Photo 2. Slurry desanding plant BE 500 with the trench slurry tank

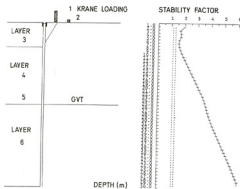


Figure 13. Trench stability factors

The computer stability calculation was conducted for the maximum trench length of 7.5 m, and maximum depth of 50 m.

The trench wall stability coefficient was determined from the relation of the slurry hydrostatic pressure and soil pressure increased by the dynamic loads incurred by the hydraulic trench cutter operation.

The stability factor was calculated for each meter of the trench depth, to its bottom, and the results indicate that the coefficient in all layers was above the specified 1.10, Fig. 13.

The diagram shows that the minimum stability coefficient of 1.541 is determined on the depth of 6.0 m from the trench top, and the maximum of 6.098 at the bottom.

The below slurry mix ratio proposed by the Contractor was accepted:

bentonite	40 kg
water	985 l
slurry density	ρ 1.05 g/cm ³

The slurry quality was permanently controlled during the excavation and before the cut-off wall concrete placement to make sure that the slurry retains its specified properties in circulation. The control included laboratory tests of the slurry temperature, density, pH, Marsh viscosity, liquid limits and sand content (%), Fig. 14.

Working Procedure

The first stage of the cut-off wall works was construction of an operating platform from which the trench was excavated with the cutter. The 10-12 m wide platform was made of 40 cm thick compacted rock placed above the geotextile protecting the dam clay core.

The platform was also used for construction of a guide canal and guide walls. The guide canal was used to guide the trench cutter when the excavation along the cut-off wall route is carried out, and as the trench slurry circulation duct. The guide walls are reinforced concrete, 1.25 m high and 0.25 m wide, so they also served for stabilization of an open trench on its top.

The working procedure is illustrated by the Fig. 15.

The preliminary excavation to the depth of 3-4 m was carried out using the kelly-bar grab. The basic excavation element is "excavation unit", and concrete placement unit is "panel". Length of one panel may include one or more excavation units. The maximum length of the primary panel is 7.0 m for the trench stability reasons, and in order to minimize the number of panels and thus increase the cut-off wall impermeability.

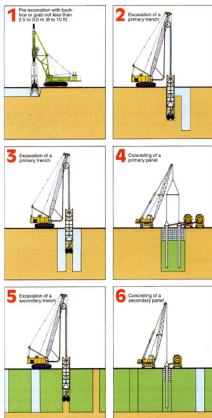


Figure 15. Cut-off wall construction procedure

Figure 16.

Secondary panel construction

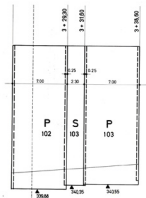
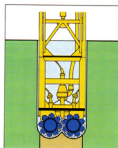


Figure 17.

Primary and secondary panel lengths with cutting

Panel = R187				Time 07:49				Date 06.11.98			
Depth 40 Startpoint				1				2.2 m			
Depth 40 Endpoint				1				32.4 m			
Total Millingtime				1				6:08 h			
Total Regeneration Time				1				0:12 h			
I	Depth	I	X-Deviation	I	Y-Deviation	I	Millingtime	I	Plans	I	
I	(m)	I	(mm)	I	(mm)	I	(sec)	I	Mr.	I	
1	0	1	+0	1	+0	1	0	1	0	1	
1	1	1	+0	1	+0	1	0	1	0	1	
1	2	1	+0	1	+0	1	0	1	0	1	
1	3	1	+0	1	+0	1	0	1	0	1	
1	4	1	+0	1	+0	1	0	1	0	1	
1	5	1	+0	1	+0	1	0	1	0	1	
1	6	1	+0	1	+0	1	0	1	0	1	
1	7	1	+0	1	+0	1	0	1	0	1	
1	8	1	+0	1	+0	1	0	1	0	1	
1	9	1	+0	1	+0	1	0	1	0	1	
1	10	1	+0	1	+0	1	0	1	0	1	
1	11	1	+0	1	+0	1	0	1	0	1	
1	12	1	+0	1	+0	1	0	1	0	1	
1	13	1	+0	1	+0	1	0	1	0	1	
1	14	1	+0	1	+0	1	0	1	0	1	
1	15	1	+0	1	+0	1	0	1	0	1	
1	16	1	+0	1	+0	1	0	1	0	1	
1	17	1	+0	1	+0	1	0	1	0	1	
1	18	1	+0	1	+0	1	0	1	0	1	
1	19	1	+0	1	+0	1	0	1	0	1	
1	20	1	+0	1	+0	1	0	1	0	1	
1	21	1	+0	1	+0	1	0	1	0	1	
1	22	1	+0	1	+0	1	0	1	0	1	
1	23	1	+0	1	+0	1	0	1	0	1	
1	24	1	+0	1	+0	1	0	1	0	1	
1	25	1	+0	1	+0	1	0	1	0	1	
1	26	1	+0	1	+0	1	0	1	0	1	
1	27	1	+0	1	+0	1	0	1	0	1	
1	28	1	+0	1	+0	1	0	1	0	1	
1	29	1	+0	1	+0	1	0	1	0	1	
1	30	1	+0	1	+0	1	0	1	0	1	
1	31	1	+0	1	+0	1	0	1	0	1	
1	32	1	+0	1	+0	1	0	1	0	1	

Figure 18. Trench excavation computer printout

When the grab excavation is completed, the excavation with the trench cutter unit is continued to the design depth.

The primary panel excavation units are done first, and they consist of three trenches, i.e. 2.8 m long side trenches and 1.4 m long central trench. When the excavation for several primary panels is completed, the panels are concreted. When the concrete reaches its minimum required strength, the excavation for secondary panels starts. They are the primary panels joining elements.

The primary panels are 2.3 m apart, and the secondary panel is 2.8 m long, thus the trench cutter cuts into the neighboring concreted primary panels for 25 cm on each side. This results in a quality joint between the panels, because the secondary panel concrete is placed directly on a coarse surface of primary panels, Figs. 16 and 17.

One of important requirements for the trench was maintenance of its plumbness, both longitudinal and transverse to the cut-off wall axis, with the allowed 0.2% deviation from

the vertical. This results in the cut-off wall continuity in the panel joints. Such an operation was possible due the digital inclinometer the trench cutter is fitted with, which displays the deviations from both axes during excavation and the actual trench depth. When an excavation unit for a particular panel was excavated, the computer record printout was made containing all the relevant excavation data: works start time, initial depth and final depth in (m), total excavation duration, slurry regeneration time, x- and y-deviation from the vertical (cm) for each meter of depth, excavation duration (min) for each meter of depth, Fig. 18.

These records verified actual depth of each panel trench before the approval for its concreting was given.

When the excavation was completed, the cutter centrifugal pump at the trench bottom pumped the trench slurry from the bottom and replaced it with the fresh slurry. Thus, the requirement that the trench bottom should be clean prior to concrete placement, i.e. free from the slurry deposit, is satisfied. The foundation joint cleanliness prior to concrete placement was checked so that the regenerated slurry sample was taken from the trench bottom and checked for solids and sand. The maximum allowable content was 7%, and the maximum recorded content was 3%.

CUT-OFF WALL CONCRETING

The preliminary test results were used to determine the cut-off wall plastic concrete mix ratio and its consistence which would be suitable for the way of transportation and concrete placement procedure.

Having in mind that the concrete for one cut-off wall panel is placed continuously, and no interruptions in the operation are allowed, the concrete was mixed in a mixing plant located in the vicinity of the concrete placement site. The concrete was transported with 6 m³ mobile mixers.

Placement of the cut-off wall concrete is the most delicate operation on the cut-off wall, since any mistake could cause major damage and imperil the basic function of this structure, namely impermeability.

The cut-off wall was constructed using the underwater concreting with the contractor pipe. The basic characteristic of this method is that the concrete is compacted by its own weight rather than by vibrations.

Therefore, in addition to the design mechanical characteristics, the concrete must be fluid, have the capacity of flow out and compacting by its own weight, be segregation and

flushing resistant, and easy setting. The fluidity must be sufficient for the concrete to fill in all the cavities while its level raises as the consequence of difference in levels in the contractor pipe and trench. The excessive water in the concrete mix causes segregation, namely flushing, of cement slurry and aggregate and formation of nests in the cut-off wall body. The consequence is a porous concrete sensitive to water flow. The contractor pipe procedure requests slower concrete setting because of possible interruptions in the concreting procedure.

The role of the contractor pipe is to prevent flushing, segregation and soiling of concrete, because the concrete is lead to the bottom of the excavated panel trench and the new concrete is permanently added into the already placed concrete.

The contractor tube is smooth on the inside, the wall is 8 mm thick, and fitted with efficient assembly tools. The tube segment is 4 m long. A tremmie is placed on the top of the pipe, and the concrete is feed directly from the mixer so that the concrete flows down from less than 2.0 m. The concrete placement rate should be minimum possible, so that the trench walls do not cave in. Prior to concreting, the trench bottom must be cleaned by the trench slurry regeneration.



Photo 3. Panel concreting

The contractor pipe is lowered to the trench bottom before the concreting starts. Two pipes were used for concreting of the primary panels, and one pipe for the secondary panels. During concreting the pipes were supported by the BS 108 B crawler crane, which was also used for assembly and disassembly of the pipe segments, Photo 3.

If the pipe foot does not reach the trench bottom, the first concrete might be flushed or soiled by slurry because the first concrete does not raise the panel surface but rather covers the trench bottom and walls. Such concrete does not render quality contact surfaces with the bedrock and side panels, therefore the trench bottom must be cleaned prior to concreting. The concrete is released in the pipe foot, displacing the slurry and raising within the trench. The concrete is compacted by its own weight. When the concrete flow is interrupted the balance is achieved of the concrete in the pipe and in the trench. The concrete level in the pipe is higher than the level in the trench because of the friction between the concrete and the internal pipe walls. The addition of concrete into the tremmie raises the concrete level in the trench.

As the concreting progresses the friction increases and the level difference prevents the concrete flow down the pipe. The contractor pipe is lifted, the top segment is removed immediately below the tremmie. The specified 1.5 m of pipe must remain in concrete to prevent mixing of the new concrete with slurry and soiled layer of concrete on the top. Dismantling of segments is continued until the trench is filled with concrete.

The panel is concreted to the level of some 30 cm under the guide canal top, so that the room is left for the slurry circulation.

To prevent inclusions of the slurry and cured in material in concrete, the concrete flow should be continuous, namely the contractor pipe should never remain empty and should not be disturbed or taken out of the concrete. Thus, it is strictly forbidden to stop concreting process within a panel.

As we have already said, the primary panels are concreted first, several panels in a row, and after excavation the secondary panels are concreted between the primary ones. The concrete displaces a layer of soiled concrete mixed with slurry which remains in the top 30 cm after the concreting process is completed.

When the cut-off wall concreting is completed, the soiled layer is removed by a trench cutter and new concrete is placed instead to the guide canal top.

CUT-OFF WALL CONSTRUCTION

The contract for the cut-off wall construction works was signed with the Croatian/German joint venture Geoteknika-Bauer Ltd. on August 8, 1994. The joint venture was organized

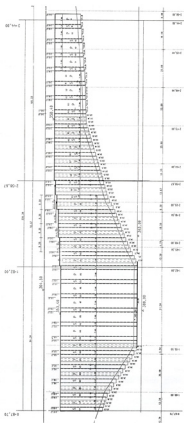


Figure 19. Cut-off wall division into primary and secondary panels

specially for the Peruća Dam remediation project. The Bauer personnel and machinery were engaged on excavating and Geotehnika on concreting. The deadline for completion of the cut-off wall construction works was February 1, 1995.

The preparations started immediately after the contract had been signed and the equipment shipped to the construction site. The on-site preparations started on September 19, 1994, including the platform, mixing plant, workshops and slurry tanks construction.

The first ten meters of the guide wall was concreted on September 30, 1994.

The preliminary excavation with grab, and the subsequent excavation with the hydraulic trench cutter started on October 10, and the first panel, P1, in stations 303.77 to 310.77 was concreted on October 13, 1994.

The works started from the right dam end where the upper section was removed to the El. 358.00 m a.s.l. because at that moment the backfilling works were not completed in the left and right dam ends to the El. 353.00 m a.s.l. and 358.00 m a.s.l. respectively.

For that reason, the excavation progressed to the station 310.77 in the left end to enable that the backfilling of the dam ends to the planned elevations be completed in the meantime. Additionally, the limiting factor for progress of the cut-off wall works was that the excavation and remediation of the caved-in gallery in its bottom part and towards the left end was under way. This left no time for completion of the contact grouting around the gallery lining. Since the gallery route runs only 4.0 m upstream the gallery axis, the abrupt slurry loss from the excavated cut-off wall trench through possible cracks and caverns in the clay could imperil the excavation crew.

This possibility was minimized by consolidation grouting along the cut-off wall trench route, during which no major cracks or caverns were encountered.

However, during excavation for the panel P-7, stations 247.97 to 254.97, during excavation of the second side trench in the rock, the slurry was lost from the inspection gallery in the section where the cracked primary concrete lining was remediated with shotcrete. The abrupt lowering of the slurry level in the trench caused minor caving-in of the trench walls immediately under the guide walls. Since the side trenches had already been excavated, the decision was brought to concrete the panel, while the central trench would be excavated later and concreted as the secondary panel.

The slurry losses were also suffered during the excavation for the panel P-6 and two panels in the right dam end. Since the slurry loss conditions recurred, and the slurry penetrated the gallery during the excavation in the bedrock and parts of the gallery remediated with the shotcrete over the old lining, it was concluded that the slurry was being lost through the joint between the bedrock and clay core, leaking into the gallery through the cracks in the old concrete gallery lining. The joint between the bedrock and clay core had probably been damaged in explosion and due to the cutter dynamic shocks during the bedrock excavation. These shocks have probably reopened the cracks in the gallery concrete.

For the above reasons, the contact grouting of the gallery concrete lining was immediately initiated, particularly in the section in front of the cut-off wall trench, in order to fill in the cracks between the core and the lining. When the contact grouting was completed, no further slurry losses were recorded.

In the meantime, the right dam end backfilling was completed and the cut-off wall construction started. The works progressed towards the left dam end.

The inspection gallery remediation was successfully completed by the end of November 1994. It was followed by backfilling of the left dam end to the EL. 353.00 m a.s.l., so the conditions for the cut-off wall completion in the left end were provided.

The last meters of the guide walls were completed on December 16, 1994. The cut-off wall construction started in the new backfilled dam section in the left end. Since the excavating for the cut-off wall run for several meters through the newly formed upstream filter, the slurry was lost during the excavation for the panel P-24. Excavation of this unit was completed under continuous adding of slurry into the trench, and it was concreted as the shortened primary panel of 2.8 m.

Although the hydraulic trench cutter had several minor failures, the excavation for the cut-off wall was completed on January 10, 1995, when the last panel was concreted. The cut-off wall top cleaning and additional concreting to the top of the guide canal was completed on January 12, 1995. The deadline was beaten for almost a month.

The plastic concrete mechanical characteristics testing during the construction was performed by Civil Engineering Institute of Croatia, Split Department:

- compressive strength: a sample cylinder was taken for each 25 m³ of the cut-off wall; 20% of samples were tested for 7-day strength, 75% for 28-day strength, and 5% for 90-day strength;
- consistence slump test for each concreted panel,
- impermeability: one sample for each 50 m³ of concrete.

Geosexpert-IGM Zagreb was engaged on erosivity testing (pin-hole test). A sample was taken for each 50 m³ of concrete.

The cut-off wall plastic concrete quality control was contracted with the Materials Department, Civil Engineering Faculty of the University of Zagreb. The control tests scope was approximately 10% of the total plastic concrete permanent tests conducted by the Contractor. The plastic concrete quality control results are covered by a special paper.



Photo 4.
Cut-off wall trench
excavation by hydraulic
trench cutter



Photo 5.
Hydraulic trench cutter
wheels

BASIC CUT-OFF WALL CONSTRUCTION DATA

The cut-off wall consists of 28 primary and 28 secondary panels, Fig. 19. It is 261.15 m long, and runs between the stations 86.82 and 347.97 along the cut-off wall axis. The design length is per the dam axis.

The maximum depth is 51.5 m in the central section, and minimum 12.9 m in the right end. It is cut in for 2.5 m into the bedrock or concrete on its entire length, and for 4 m in the locations where the bedrock is steep.

The relevant cut-off wall data are:

-	total surface area	8,505 m ²
-	cut-off wall excavation quantity, including excavation for 25 cm thick joints between the panels	9,282 m ³
-	cut-off wall plastic concrete quantity	9,335 m ³
-	guide wall concrete quantity	172 m ³
-	quantity of plastic concrete for the cut-off wall top section	265 m ³

The Contractor kept the daily logs and submitted daily reports on excavation and panel concreting. The panel record includes the excavation unit designation and station, excavation start and completion, trench length and depth, interruptions of excavation and concreting works, concreting start and completion, volume of excavated trench and placed concrete, and any possible incidents, slurry losses, machinery failures, etc.

The report analysis could render the information on the machinery productivity during the excavation for the cut-off wall in clay and bedrock, or concrete, on different excavation depths and for different purposes - for primary or secondary panels.

Average excavation rate in the existing clay core:

-	primary panel side trenches, l=2.8 m	22.5 m ³ /h
-	primary panel central trenches, l=1.4 m	22.0 m ³ /h
-	secondary panels, l=2.8 m	29.6 m ³ /h

Average excavation rate in the newly backfilled clay:

-	primary panel side trenches, l=2.8 m	36.2 m ³ /h
-	primary panel central trenches, l=1.4 m	24.3 m ³ /h
-	secondary panels l=2.8 m	44.8 m ³ /h

Average excavation rate in the bedrock or concrete:

- primary panel side trenches, $l=2.8$	6.5 m ³ /h
- primary panel central trenches, $l=1.4$ m	5.3 m ³ /h
- secondary panels $l=2.8$ m	7.9 m ³ /h

The excavation time in clay and bedrock changes with the depths, so at the beginning it may be only several minutes, while on 48 m 20 min/m is necessary.

Having in mind total excavation duration for a single primary and secondary panel, the below rates were achieved:

- excavation for primary panels	18 m ³ /h
- excavation for secondary panels	25 m ³ /h

The excavation rate per a working shift (8 hours) is calculated to be:

- existing clay excavation per shift	176-237 m ³
- bedrock excavation per shift	42-63 m ³



Photo 6. Top of the constructed cut-off wall in the right dam end

When the excavation time is added the time necessary for the trench cutter removal, alignment and pulling out from the excavation site, the average time necessary for excavation for the largest panels was:

- primary panel (51 m deep, 350 m³) 3 shifts
- secondary panel (51 m deep, 140 m³) 1 shift

The panel concreting rate primarily depends on the mixing plant output of about 60 m³/h, less the time necessary for occasional pulling out and removal of the contractor tubes.

The average panel concreting rate was 36 m³/h, so the concreting of the largest 370 m³ panels was completed in an extended shift of 12 hours.

CONCLUSION

The plastic concrete cut-off wall was constructed in the Peruća Dam clay core. It is 261.15 m long, which makes about 59% of the dam length. It consists of 28 7.0 m long primary panels, and 28 2.8 m long secondary panels. The cut-off wall depth varies between 12.9 m in the right dam end and 51.5 m in the central dam section. Total cut-off wall surface area is 8,505 m².

1 m³ of plastic concrete contained 112 kg of cement, 35.6 kg of bentonite, 102 kg of rock flour, 1,527 kg of aggregate with the grain size 0-4 mm, and 337 l of water. The plastic concrete mechanical characteristics were: 28-day compressive strength ≥ 1 MN/m², Young's modulus of elasticity ≤ 400 MN/m², Darcy's coefficient of filtration $< 10^{-8}$ m/s.

The cut-off wall trench was excavated using the grab and hydraulic trench cutter with the average output of 22 to 30 m³/h in clay, and 5 - 8 m³/h in bedrock and concrete.

The average concreting output was 36 m³/h.

The Geoteknika-Bauer joint venture completed the cut-off wall, including the on-site preparatory works, in four months.

LABORATORY TESTING OF THE CUT-OFF WALL PLASTIC CONCRETE

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ABSTRACT

Results of preliminary and control laboratory testing of the cut-off wall plastic concrete are presented. Compression and tensile stress tests were made, and erodibility and water permeability measured. The stress-strain characteristic behavior as a result of unconfined compression test was approximated by method of nonlinear regression analysis with hyperbolic function. It was shown that the modulus of elasticity depends on stress and strain and that it could be calculated if just the compression strength was known. The bending tests implicated that, due to plastic behavior of plastic concrete, the nonlinear stress distribution along the sample cross-section must be taken into account when tensile strength is calculated. The "Pin-hole" test was new designed to cover specific requests in the structure.

INTRODUCTION

The cut-off wall plastic concrete is intended to provide for designed controlled seepage level during the dam lifetime [1]. The dam must additionally meet a number of other requirements and criteria such as strength, strain, and longevity. The plastic concrete characteristics subject to testing are defined by the cut-off wall design documentation [2], i.e.

- 28-days compressive strength shall be more than 1.0 MPa
- 28-days compressive strength 10% samples may have the less than 1.0 MPa
- 7-days compressive strength should be more than 65% of the 28-days compressive strength
- Young's modulus of elasticity shall not exceed 400 Mpa

- coefficient of impermeability shall not be more than 10^{-8} m/s
- 10% samples may have the coefficient of impermeability between 10^{-8} m/s and 10^{-7} m/s.

After completion of preliminary laboratory tests [4] the Contractor, Geotehnika-Bauer, proposed and the dam Architect Engineer accepted the below plastic concrete fill mix ratio:

cement	112,0	kg
bentonite	35,6	kg
rock flour	102,0	kg
0-4 mm aggregate	1,527,0	kg
water	337,0	l

The internal control of the bentonite suspension was performed by the Geoexpert laboratory for the Contractor. The internal control of aggregate, green plastic concrete consistence, mixed plastic concrete and plastic concrete samples taken from the cut-off wall was conducted by the Civil Engineering Institute of Croatia, Split Division. Under the contract signed with the Development and Engineering Division of the Croatian National Electricity the external control was performed by the Materials Department of the Civil Engineering Faculty in Zagreb.

The laboratory samples were tested during mixing and pouring of the plastic concrete, and the samples were taken from the completed cut-off wall body by sawing during the integrity and impermeability tests on the entire cut-off wall.

The standard preliminary and control tests of the plastic concrete samples indicated that its behavior was different from that of the plain concrete. This behavior should be described as a guideline for other similar tests. The plastic concrete has the compressive strength which is higher than in standard geomechanical materials (clay and similar), but is still relatively low as compared with the compressive strength of plain concretes. Therefore, the majority of procedures specified by the standards for concrete and clay had to be modified. The paper gives the results of laboratory tests conducted on external control samples carried out during the cut-off wall construction [5, 6, 7].

TEST PROCEDURES

The modulus of elasticity was tested according to the ISO recommendations and U.M1.025 standards. The samples were ϕ 15 cm 30 cm high cylinders. To prevent occurrence of irregular concentrated stresses caused by irregular contact surface, the sample bases were topped with the quick-setting cement. The modulus of elasticity and strength were tested on a crushing machine with the measuring force of 150 kN, Photos 1 and 2, equipped with additional electronic force transducer and inductive sensors (LVDT) for strain measurements. The stress rate was 1 MPa/min. The force and strains voltage signals are transmitted to the computer through an AD converter, which enables continuous recording of force-strain diagram.



Photo 1.



Photo 2.



Photo 3.

The **density and compressive strength** testing was carried out according to the U.M.020 standard. The samples were cured so that they were constantly kept in humid atmosphere (after hardening in mold, they were wrapped in wet flannel cloth and stored in a plastic bag). The cylinders ϕ 15cmxh=15cm, ϕ 15xh=30cm, and the cubes with 15cm and 10cm edges were tested. The cubes were beforehand topped with the cement paste in order to achieve the satisfactory surface levelling. The majority of samples were tested according to the testing program, after 28 days, and some samples were tested after 90 days or after more than 6 months. A separate sample series was subject to testing of the effect of partial drying out: the cylinders ϕ 15cmxh=15cm from the same concrete batch of the same age were left for 4 hours in the laboratory ambient conditions (20°C and 60% relative humidity).

The **impermeability** is tested according to the U.B.034 standard, using the falling pressure method, Photo 3. After 28-days of curing, the cube-shaped samples placed during the cut-off wall concreting were cut into the ~ 4 cm high slabs which were subsequently formed into ϕ ~ 10 cm cylindrical samples. The majority of tests were carried out so that the water was flowing in the plastic concrete placement direction, as commonly used per the above standard. For comparison, the four samples were prepared so that the water flow was perpendicular to the sample placement direction, which is in better concordance with the cut-off wall exploitation conditions.

The **erosivity** was tested according to the procedure described in the Professor Nonveiller's book [8], with certain modifications agreed upon with the Architect Engineer, required in order to achieve the water flow of approximately 3 m/s and subsequently 6 m/s. The apparatus, Photo 4, was tailored to the tested material and testing conditions in the structure. The samples were cured in the same way as for all other tests, namely constantly wrapped in the flannel cloth and stored in PE bag. The sample size is ϕ 10cmxh=7 cm, a ϕ 4 mm hole is drilled through the sample center (due to sand grain size and water flow rate, along with the poor plastic concrete cohesiveness). The sample is placed in the apparatus. The pressurized water is forced through the sample hole at high rate.

The **bending strength** was tested according to the U.M.010 standard. The size of the prism-shaped samples was 10 cm x 10 cm x 40 cm.

The samples of the **concrete placed** in the plastic concrete cut-off wall have been taken two times: (1) in the beginning of the works, from the panel No. P 003, with profiles of 100 mm and 60 mm. The 100 mm profile samples have been taken by dry sawing using the rotary core drill; (2) during the cut-off wall concrete drilling for the dam integrity and the entire cut-off wall impermeability testing. A large number of samples were rejected already during the inspection because they were defective, and the defects were caused by the sampling procedure. Only three samples from the cut-off wall body, with 60 mm profile, were used for testing. All other samples of larger profiles suffered such disturbances during the sawing from the cut-off wall body that they cannot be taken as representative.

Photo 4.



RESULTS

The preliminary sample tests have already shown that the stress-strain relation is highly nonlinear and that the ascending and descending curves of the subsequent loading and unloading are different, Fig. 1.

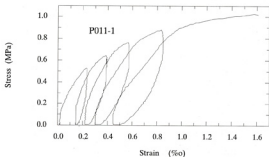


Figure 1. Typical stress-strain curve during the sample "training"

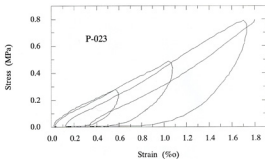


Figure 2. Stress-relative strain curve under repeated loads
for a preliminary loaded sample with strength over 60%
The back creeping is noticeable

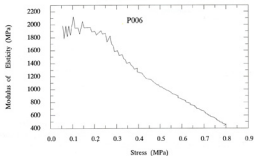


Figure 3. Changes in "modulus of elasticity" vs. stress

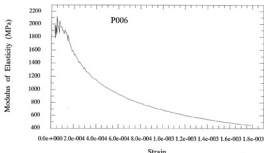


Figure 4. Changes in "modulus of elasticity" vs. relative strains

It has also been noticed that when the sample is loaded to a certain stress which is below ~ 60% of the strength, and then unloaded to zero, the repeated loading shall again result in a nonlinear stress-strain relation. On the contrary, after the preliminary loading > 60% of strength the approximately linear stress-strain relationship is obtained, Fig. 2. It has been noticed that this almost boundary stress increases with the increase in sample strength.

The back strain of a sample during a certain period of time after the unloading has also been noticed, Fig. 1. It may be assumed that the stress-strain relationship changes with time, namely that it is the function of the time-related stress and permanent stress rate. Since the main task of the compressive stress-strain measurement was to determine the modulus of elasticity, it was immediately clear that the modulus changes with stress, and with the relative strain within certain boundary values, Figs. 3 and 4.

The shape analysis of the measured stress-relative strain relations reveals that the obtained experimental data can be described with the below simple hyperbolic function:

$$\sigma = \frac{f_c \cdot \varepsilon^n}{\varepsilon_0 + \varepsilon^n} \quad (1)$$

where: σ - stress, f_c - compressive strength, ε - relative strain, n - exponent, and ε_0 - constant strain in zero with no physical meaning (parameter of hyperbola). By applying the method of nonlinear regression analysis on measurement data, the coefficients f_c , ε_0 and n were calculated for each sample, Fig. 5.

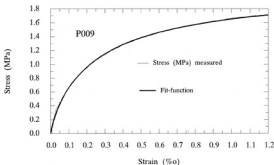


Figure 5. Experimental data and theoretical stress-relative strain curve (fit function)

The correlated compressive strength experimental and theoretical values shown in Fig. 6 confirm accuracy of the assumed function (1).

The mean value $n(\text{mean}) = 0.908$ of all the samples with 5% standard error, Fig. 7, was obtained from the analyzed data for exponent n using the linear regression, and n was approximated as unity. Therefore, the equation (1) is simplified:

$$\sigma = \frac{f_c \cdot \varepsilon}{\varepsilon_0 + \varepsilon} \quad (2)$$

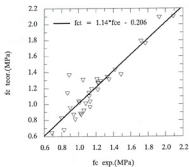


Figure 6. Relationship of measured and theoretically calculated f_c (compressive strength)

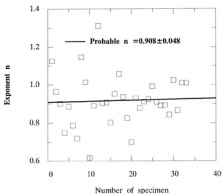


Figure 7. Mean exponent n for all measurements

The constant ϵ_0 , which may be correlated with the experimentally obtained compressive strengths, Fig. 8, is missing for definition of the stresses (2). Using the linear regression ϵ_0 o f_{exp} , the function $\epsilon_0(\%) = 0.2943 f_{exp} + 0.024$ has been calculated. Since the strength changes within the approximate boundaries of 1 to 2 MPa, ϵ_0 shall consequently be between 0.2 and 0.6 %.

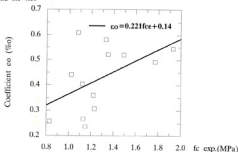


Figure 8. Relationship of coefficient ϵ_0 and experimental compressive strength

By substitution of known (calculated) coefficients f_c i ϵ_0 , the explicit form may be obtained of the stress-relative strain relation, and thus of the relationship between the modulus of elasticity and stress or relative strain.

$$E = \frac{f_c - \sigma}{\epsilon_0} \quad (3)$$

$$E = f(\epsilon) = \frac{f_c \cdot \epsilon_0}{(\epsilon_0 + \epsilon)^2} \quad (\text{tangent modulus}), \quad (4)$$

$$E = f(\epsilon) = \frac{f_c}{\epsilon_0 + \epsilon} \quad (\text{secant modulus}), \quad (5)$$

In other words, the corresponding modulus of compression for any required relative strain or stress can be calculated from the known coefficients f_c i ϵ_0 . The function of modulus according to (3, 4 and 5) for the material for which the stress-strain curve has not been

measured but the compressive strength is known to be between 1 and 2 MPa, may be obtained if the parameter ϵ_o , Fig. 6 is determined beforehand. Therefore, the value of the compression modulus is not specified for the tested material, but it rather varies depending on the stress and relative strain within a rather large interval (e.g. Figs. 3. and 4.). Therefore, the plastic concrete quality, in addition to the modulus of elasticity (in this case $E \leq 400$ MPa), should also be specified by adequate stress or relative strain. In the subject case, when only the maximum model is specified, it is possible to calculate the corresponding stresses and relative strains for each tested sample from the equations (3, 4 and 5), provided the specified maximum module is within the defined range of functions (3, 4 and 5), which means $\sigma_{400} < f_c$ and $\epsilon_{400} < \epsilon_{\text{failure}}$. Thus, substituting $E=400$ MPa (tangent) into (3 i 4) gives

$$\sigma_{400} = f_c - 400 \cdot \epsilon_o \quad (6)$$

$$\epsilon_{400} = \sqrt{\frac{f_c \cdot \epsilon_o}{400}} - \epsilon_o \quad (7)$$

or if $E=400$ MPa secant is substituted in (5) it gives:

$$\epsilon_{400} = \frac{f_c}{400} - \epsilon_o \quad (8)$$

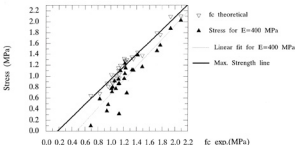


Figure 9. Stress interval (between curves) within which the modulus of elasticity is less or equal to 400 MPa

The condition $E \leq 400$ MPa requests $\sigma \geq \sigma_{400}$ and $\epsilon \geq \epsilon_{400}$. Therefore, the compressive modulus of elasticity shall be below the set limit of 400 MPa only for the stress exceeding σ_{400} and relative strain exceeding ϵ_{400} which is, according to the diagram in Fig. 9, very close to the material compressive strength point within a comparatively narrow stress interval, but a quite broad relative strains range. The area between the dotted and solid line in Fig. 9 presents the stress interval of approximately 0.1 MPa, within which it applies that the modulus of elasticity is less or equal to the set value of 400 MPa. The similar analysis may be conducted for relative strains.

The results of the compressive strength testing are presented in Table 1, and for comparison the Table 2. is given with the results of the contractor's internal analysis. The Table 3. contains the test results for the $\phi 59$ mm cylinders taken from the cut-off wall plastic concrete.

Size days	$\phi 15 \times 15$ 28 days	$\phi 15 \times 30$ 28 days	$\phi 15 \times 15$ 90 days	$\phi 15 \times 15$ wet	$\phi 15 \times 15$ 28 days, dry	$\phi 15 \times 30$ >180 days
No. of samples	30	29	4	5	5	27
arithmetic means	1.24	1.11	2.09	1.21	1.37	1.88
standard deviation	0.29	0.30	-	-	-	0.43
coefficient of variation	0.23	0.27	-	-	-	0.23
density, t/m ³	2.09	2.10				

Table 1. Summary of the plastic concrete external control compressive strength testing results

STATISTICS	Sample age		
	7 days	28 days	90 days
No. of samples	67	257	20
Arithmetic mean, MPa	0.92	1.34	1.76
Standard deviation, MPa	0.14	0.219	0.38
Coefficient of variation	0.16	0.16	0.22

Table 2. Control samples results statistics

Panel P-003	density, t/m ³	compressive strength, MPa	profile, cm	height, cm
1	2.28	1.28	5.90	12.2
2	2.25	1.32	5.90	13.1
3	2.20	1.46	5.90	15.9

Table 3. Test results for 55-day cut-off wall samples

The design requirement for individual strength was met, i.e. the individual strength values and the mean strength value is higher than the specified, with 10% results lower than specified.

At fracture, all the cylinders had typical vertical cracks along the entire height, Photo 5 and 6, which indicates that the friction between the sample and the crushing machine plates was very low. Still, the strength of the higher samples was in average 8% lower (20% for plain concrete).

The correlations of the test results for the dried-out and wet samples cannot be considered reliable on the basis of the number of results. Still, since the testing was conducted on parallel samples from the same plastic concrete batches and under the same curing conditions, the indications are clear. Even more since these data correspond with the known relations valid for the plain concrete. Thus the strength of samples which were exposed to drying in the laboratory atmosphere, i.e. dried out on the surface, 4 hours prior to testing had increased by 10 to 73%. The strength of the cube with 15 cm edge is approximately equal to the strength of 15 cm high cylinder.

Testing the plastic concrete strength after more than 28 days has indicated the strength growth common for the concrete with lower percentage of cement (50-70%).

The results of the Contractor's internal control and the external control are in good agreement.

All individual densities of the cylinders taken from the structure are higher than the mean densities of the laboratory cylinders. The cylinder heights exceed the diameter by two or more times, but the strengths are approximately equal to the strengths of the laboratory samples. Based on these data, it may be assumed that the plastic concrete density and strength within the structure increases with the depth of sampling. Due to the difficulties related to the structure sampling it was not possible to set a correlation between the laboratory samples and the structure samples.

The results of the **tensile strength testing by bending** are given in the Table 4. According to the U.M1.010 standard it is calculated from the formula

$$\beta_s = \frac{3 \cdot P \cdot l_s}{2 \cdot b \cdot h^2} \quad (9)$$

where:

β_s	bending strength (N/mm ²)
P	fracture force (N)
l_s	span (mm)
b	sample width (mm)
h	sample height (mm)



Photo 5.



Photo 6.

No.	Specimen	Fracture bending force (N)	Bend. strength β_x (N/mm ²)	Max. compress. force (N)	Compress. strength (N/mm ²)	f_{Eas} (N/mm ²)	σ_{Eas} (N/mm ²)	f_{Eas} (N/mm ²)	σ_{Eas} (N/mm ²)	f_{Eas} (N/mm ²)	σ_{Eas} (N/mm ²)
1.	P-010	2250	0.844	25250	2.273	0.374	1.308	0.335	2.009	0.305	4.115
2.	P-012	2681	1.006	34750	3.130	0.445	1.559	0.399	2.394	0.363	4.904
3.	P-009	1427	0.535	15850	1.430	0.237	0.829	0.212	1.274	0.193	2.611
4.	P-011-1	1654	0.620	18350	1.655	0.275	0.961	0.246	1.477	0.224	3.025
5.	P-011-2	1798	0.674	18950	1.710	0.298	1.045	0.267	1.605	0.244	3.289
6.	P-013	1922	0.721	22000	1.985	0.319	1.117	0.286	1.716	0.260	3.516
7.	S-009	1449	0.544	17050	1.535	0.241	0.842	0.216	1.394	0.196	2.651
8.	P-015-1	1668	0.626	20750	1.870	0.277	0.969	0.248	1.489	0.226	3.051
9.	S-012	1304	0.489	19500	1.755	0.217	0.758	0.194	1.164	0.177	2.385
10.	P-017-1	1521	0.571	16250	1.463	0.253	0.884	0.226	1.358	0.206	2.782
11.	P-019-1	1945	0.729	-	-	0.323	1.131	0.289	1.757	0.263	3.558
12.	P-021-1	2541	0.953	-	-	0.422	1.477	0.378	2.269	0.344	4.648

Table 4 10 cm x 10 cm x 40 cm prism bending test results and strength calculation

The bending strength obtained in this way is not realistic because the actual distribution of stress per the cross-section height is not linear. In materials such as concrete, the increase in load (before the cracks open) causes the neutral axis shifting towards the compression edge (stress stage Ia). Now, it is possible to assume the stress distribution along the parabola (for the compressive part) and rectangle (for the tensile part). This assumption also applies to the plastic concrete since its plasticity is more pronounced than for the plain concrete. Two cases of equilibrium in the cross-section were investigated in the moment when the maximum force is achieved. The first case assumes that the neutral axis shifted by 20% towards the compressive edge, and the other assumes the 30% shift. The results of the two cases show slight difference, but a considerable difference as compared to the strength calculated from the formula (9). For that reason, and for the assumed nonlinear stress distribution, the bending stress results for these two cases were more realistic. The possible conclusion is that the bending stress is within the range of these two results. The bending stress is calculated from the below formula:

$$f_t = \frac{M}{h \cdot (h - x) \cdot z} \quad (10)$$

where:

f_t	bending strength (N/mm ²),
$M = PL/4$ (Nmm)	max. moment,
h	cross-section height (mm),
x	compressive zone height (mm),
$z = 0.625 \cdot x + 0.5 \cdot (h - x)$	the arm between the tensile and compressive force in concrete (mm).

The stress on the compressive edge is calculated from the formula:

$$\sigma_c = \frac{3}{2} \cdot f_t \cdot \frac{h - x}{x} \quad (11)$$

where:

σ_c	stress on the cross-section compressive edge (N/mm ²)
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The stresses on the cross-section compressive edge were calculated for three positions of the neutral axis, i.e., for the neutral axis shift of 20%, 30% and 40% from the cross-section compressive edge. According to the results, the neutral axis moved in the moment of fracture for 20% to 30% towards the compressive edge when the compressive stress was close to compressive strength. For further neutral axis shifts, the compressive strength

would be exceeded on the cross-section compressive edge, which means the fracture would result because the compressive strength is reached, which is not correct. This only proves that the above assumption is correct and that the tensile strength can be calculated from the formula (10).

The summary results of the plastic concrete **impermeability** are presented in the below bar chart, Fig. 10. Out of 30 tested samples 5 had the impermeability coefficient a little above the designed $k \leq 10^{-8}$ m/s, i.e. from 1.22 to 5.52×10^{-8} m/s. The impermeability of all other samples was lower for an order of magnitude.

The impermeability of the samples for which the impermeability testing was carried out perpendicular to the concrete placement direction was lower for an order of magnitude than the impermeability of plastic concrete samples taken in the concrete placement direction. The impermeability decreases with the testing duration.

During the plastic concrete **erosivity** test (pin-hole test), no changes were noticed within the usual period of 24 hours. Therefore, the testing was extended for another day or two on a larger number of samples, and the flow rate was measured under the equal pressure. Also, no change in a flow rate was recorded. The eroded surface evaluation is visual, Photo 7. When the samples were cut, no difference was noticed either between the holes through which the water run for 1 or 2 days or between the holes through which the flow rate was 3 m/s or 6 m/s. The sediment color does not change after erosion, as would be the case with clay, and the correct monitoring of the changes in sample weight was not possible because of the sample water content and poor cohesion.

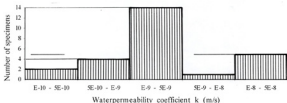
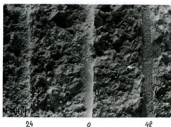
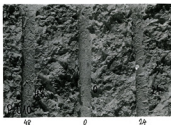


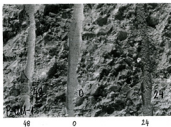
Figure 10. Impermeability coefficient results bar chart



P-004



P-010



P-011-1

Photo 7.

REFERENCES

- [1] Filling Materials for Watertight Cut off Walls, ICOLD Bulletin 51, Commission Internationale des Grands Barrages, Paris, 1985
- [2] The Peruća Dam Remediation Design, Impermeable Cut-off Wall, Elektroprojekt Zagreb, 1994.
- [3] The Peruća Dam Remediation, Investigation Works Program, Elektroprojekt Ltd., Zagreb, March 1993
- [4] Preliminary Laboratory Testing of the Component Material Quality and Determination of Optimum Bentonite-Concrete Fill Mix for the Peruća Dam Cut-off Wall - A Report, Geexpert - IGM, October 1994
- [5] 1st Preliminary Report on the Cut-off Wall Plastic Concrete, Civil Engineering College, Zagreb, December 1994
- [6] 2st Preliminary Report on the Cut-off Wall Plastic Concrete, Civil Engineering College, Zagreb, February 1995
- [7] 3st Preliminary Report on the Cut-off Wall Plastic Concrete, Civil Engineering College, Zagreb, June 1995
- [8] Nonveiller E., Soil Grouting, Workbook, Civil Engineering College, Zagreb 1969
- [9] Final Report on Bentonite-Concrete Mix Preparation, IGH, Split Division, April 1995
- [10] Final Report on Cut-off Wall Concrete Samples Testing, IGH, Split Division, June 1995

GROUTING GALLERY DAMAGE AND REMEDIATION

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SPLIT

ABSTRACT

The Peruća Dam grouting gallery remediation was the most challenging task and, with respect to safety of the crews, a high risk activity.

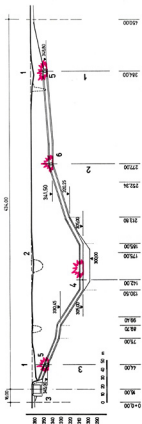
One gallery section had been completely demolished, and the rest was heavily damaged. Since the available space for excavation and loading of caved-in material was very confined, it was not possible to use the machinery and the majority of works had to be done manually.

The grouting gallery remediation works lasted for 16 months.

INTRODUCTION

The explosion which caused heavy damage to the Peruća Dam actually happened in the grouting gallery where the explosive was planted.

The original grouting gallery had a horse-shoe cross-section and was 384 m long. The gallery was constructed of plain concrete with minimum wall thickness in the vault of 80 cm. In 85% of its length, the gallery was founded on the bedrock, and in the deepest 50 m long section it was founded on 7-8 m thick concrete slab which replaced the alluvial deposits from the Cetina riverbed in the dam site. To the elevation of the vault abutment the gallery was cut in the rock, so only the vault was in the clay core.



KEY

1. CRATERS ON THE RIGHT AND LEFT DAM ENDS
2. SUBSIDENCE IN THE CENTRAL DAM SECTION
3. DESTROYED SPILLWAY STRUCTURE
4. GALLERY DRAINAGE PIPE
5. DESTROYED GALLERY ACCESS TUNNELS
6. DAMAGED GALLERY ACCESS TUNNEL

Figure 1. The Peruća Dam - longitudinal section

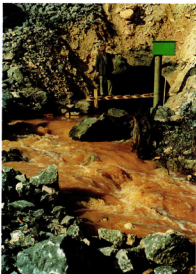


Photo 1. Water flowing through heavily damaged access tunnel No. 2

When the dam was constructed in late fifties, the joint between the gallery concrete and bedrock was contact grouted. In 1973, when the dam was in operation, the additional consolidation grouting of the surrounding bedrock was performed. There was actually no seepage into the gallery since.

The explosive blasted by the Serb rebels was planted in four points in the gallery, Fig. 1. On the basis of an analysis of seismograms obtained from the seismic stations, which registered the earthquake as a result of the explosion, the experts from the Geophysical Institute of the University of Zagreb estimated that approximately 30 t of TNT explosive charge was set in the gallery.

The explosion and dam damage were accompanied by intensive water seepage in the left gallery end. The water flowed through the drain pipe and vertical shaft under high pressure, and reached on the Peruća HPP switchyard platform, carrying along the clay core chunks to 50 cm in diameter. By this route, the water washed away about 400-500 m³ of the clay core and filter material.

When the drain pipe got clogged, the water flow continued through the access tunnel No. 2 located in the right dam end, and the flow rate became about 500 l/s. The water flow through the tunnel was not stopped until the reservoir level dropped below the elevation 345.00 m a.s.l.

The Peruća Dam remediation design specified that the comprehensive remediation solution should encompass complete reconstruction of the grouting gallery.

The present paper shall describe the gallery damage and remediation activities. For better understanding, the remediation design has also been presented, although it is detailed in the paper titled The Peruća Dam Remediation Design.

REMEDICATION DESIGN

Background

The difficulties encountered in the initial stages of work were predominantly those caused by the lack of adequate data on the dam body damage scale and nature. Precise determination of the gallery condition and damage scale was not possible because the two access tunnels were destroyed and inaccessible, and the third (central) tunnel was heavily damaged. The access through this tunnel requested special precaution because of possible mines, but it lead to the gallery, El. 341.50 m a.s.l. and enabled inspection of a comparatively small portion of about 50 m, Photo 1.

Therefore, the decision on the dam remediation solution was brought on the basis of assumptions which were founded on a visual inspection of 50 m of the gallery, dam crest settlement monitoring after the explosion and information on the explosion sites and intensity. The assumed damages were classified in three degrees for the design purposes.

- 1st degree damage

The gallery is cracked in the vault and walls, but no deformations were suffered and the vault retained its static function.

- IInd degree damage

The gallery is strongly cracked but did not cave in. The opening span is deformed but it still retained its shape. The parts of the vault are ripped off, but it retained its static function although there is a threat of its partial or complete caving-in.

- IIIrd degree damage

The gallery is completely demolished, which includes the vault and walls, while the foundation slab is damaged.

In addition to the above damages, the construction technology had to take into account the deposited high-plasticity clay deposited by the clay core flushing and flow of water through the gallery. Also, the gallery walls were completely covered with soot from the hot gases and fire caused by the explosion.

Remediation Design

The remediation design was conceived on the basis of the above assumed degrees of damage. Three gallery remediation methods were proposed.

(a) TYPE-1

For the first degree damage.

The remediation includes placement of 10 cm thick shotcrete lining reinforced with double Q-131 steel mesh, Fig. 2. No supporting was planned.

(b) TYPE-2

For the second degree damage.

The remediation for this degree of damage includes placement of 15 cm thick lining with steel stilted arches with flexible joint, and double steel mesh reinforcement, Fig 3.

KEY

1. GROUTING GALLERY
2. EXISTING CLAY CORE
3. BEDROCK
4. EXISTING CONCRETE
5. CRACKS
6. STEEL MASH
7. STEEL ARCHES
8. SHOTCRETE

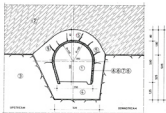


Figure 2. TYPE-1 grouting gallery remediation method

KEY

1. GROUTING GALLERY
2. EXISTING CLAY CORE
3. BEDROCK
4. EXISTING CONCRETE
5. CRACKS
6. STEEL MASH
7. STEEL ARCHES
8. SHOTCRETE
9. STEEL BARS

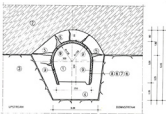


Figure 3. TYPE-2 grouting gallery remediation method

KEY

1. GROUTING GALLERY
2. EXISTING CLAY CORE
3. EXISTING CONCRETE
4. NEW CONCRETE
5. ANCHORS
6. STEEL PLATES
7. FORMWORK
8. STEEL ARCHES

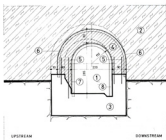


Figure 4. TYPE-3 grouting gallery remediation method

(c) TYPE-3

For the third degree damage

The remediation is performed in two stages:

- First, the excavation is carried out under the protection of steel stilted arches with flexible joint and steel plates. The excavation goes along the profile perimeter to the steel arch, after which the internal part of the profile is excavated.
- The 80 cm thick gallery concrete lining is placed in a protected trench, Fig. 4.

The design also planned that the same remediation method should be applied for the left end access tunnel and the access tunnel in the right end.

REMEDIATION PROCEDURE

After considering the available data on the gallery damage scale and nature, and the situation on the site, the decision was brought to carry out the gallery remediation in three stages.

The first remediation stage includes the gallery section between the stations 2+77.00 and 3+42.00. The gallery was accessible from the access tunnel No. 2, which is the only tunnel passable to a certain degree. The works in this stage were of tunnel type.

The second stage encompasses a gallery section between the stations 0+40.00 and 1+33.00, including the reconstruction of the access tunnel No. 3; after removal of the left dam end, the works were partly conducted in an open trench, between the stations 0+40.00 and 0+89.70, and partly in tunnel, between the stations 0+89.70 and 1+33.00.

Such site organization was affected by the below factors:

- the grouting gallery remediation could have been started much earlier than the other works since the grouting gallery design was completed by the end of July 1993, while the dam remediation design was not finished until late August 1993;
- between July 1993 and February 1994 the Peruća Dam and HPP were occasionally exposed to rebel Serb shelling, so the activities on the dam crest and ends removal could not have been initiated; the only safe access to the gallery was the access tunnel No. 2.

Also, it was not planned that the remediation from the station 1+30.50 in the deepest straight section be conducted in the upwards direction because it was assessed the rockslides were possible which could threaten human lives.

As regards the work technology, it was planned that the gallery shall be entered to assess the damage and determine the remediation procedure on the site. Where necessary, the gallery vault was secured by steel arches immediately upon entrance. On the outside, the arches were covered with reinforcement mesh. The arches were longitudinally braced with dia. 25 mm steel rods, and transversely embedded into the gallery concrete. Also, the advance upon entrance should have been preceded by checks for planted foot mines in the deposited clay on the gallery bottom. This was performed by the military personnel.

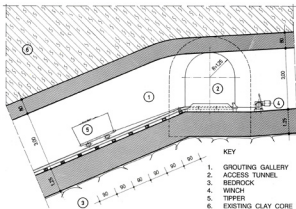


Figure 5. Removal of clay and other caved-in material from the gallery



Photo 2. Rail vans winches



Photo 3. Van rail in the gallery



Photo 4. Concrete mixing plant for the gallery remediation, operating platform in front of the access tunnel No. 2

The clay deposit was removed as the gallery was entered, along with the caved-in clay from the demolished gallery sections. It was planned that the clay shall be removed by winch-operated rail vans, Fig. 5 and Photo 2. Since the situation in the gallery was not known, it was not possible to determine the necessary cleaning and clay excavation machinery in advance. The concrete lining placement in the sections where the works were carried out in the tunnel followed after the excavated section was supported. In points where the gallery was completely demolished, the lining was placed immediately after supporting, in 4-6 m long sections. The old and new concrete were linked by the anchors drilled into the old gallery concrete.

When the lining was placed inside the gallery, the existing concrete was sandblasted to remove the soot, and all the cracks were cleaned of clay and dust. This was done to achieve better bond between the old and the new concrete. The lining was shotcreted. The shotcreting was preceded by embedment of the mesh reinforcement.

SITE ORGANIZATION

The grouting gallery remediation site organization was adapted to the remediation work requirements and specific on-site conditions.

The site was organized for the first remediation stage which was completely of the tunnel type, with the work site access through the access tunnel No. 2. For that purpose an operating platform was arranged in front of the access tunnel No. 2, where all the site offices, concrete mixing plant, fan and compressor station, water tank, shotcrete and pumped concrete equipment were located, Photo 4.

Due to permanent threat of shelling, the work, vehicle traffic and movement of staff was organized so that it was mainly performed indoors. The majority of staff was moving in the gallery or near the access tunnel No. 2, so that in case of shelling they could quickly find a shelter. Such a situation lasted from August 1, 1993, when the works started, to April 1994 when the ceasefire agreement was signed with the Serbs.

REMEDATION WORKS AND DAMAGES

Introduction

The grouting gallery remediation started on August 1, 1993, and were completed on November 24, 1994, namely within 16 months.

The works started with remediation of the access tunnel No. 2. The works and damages are presented as per the work stages.

Access Tunnel

The remediation of the access tunnel was initiated first. A new entrance portal was constructed, since the old one was completely destroyed in explosion, Photo 5. The damaged concrete lining was supported with the stilted steel arch and mesh reinforcement, Photo 8. Before the supporting started, about 50 cm thick clay deposit had been removed from the access tunnel. The material had been checked for mines by military experts.

A serious problem for the contractor arose in the joint of the access tunnel with the gallery, station 2+77.00. A part of the gallery vault had caved in, and it needed complete reconstruction. The reconstruction was performed so that the part of the gallery which had not caved in was supported by the steel arches and mesh reinforcement, and subsequently shotcreted, Photo 9.

Only now could the works be started in the gallery, to the left and right of the station 2+77.00.

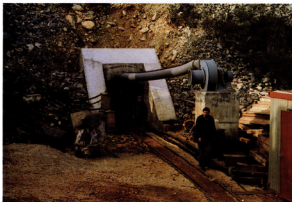


Photo 5. Access tunnel No. 2 entrance with the ventilation equipment



Photo 6. Reconstruction of the access tunnel No. 2 portal



Photo 7. Reconstructed access tunnel No. 2 portal



Photo 8. Access tunnel No. 2 after placement of steel arches

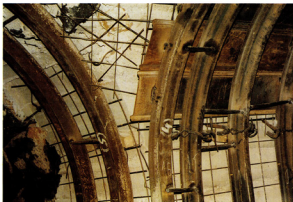


Photo 9. Supporting damaged vault in the joint between the gallery and access tunnel No. 2

**Gallery Section from Station 2+77.00 Towards the Right Dam End
to Station 3+42.00**

The works in the joint between the gallery and the access tunnel, towards the right dam end, run with no major problems. This part of the gallery is not steep, so the cleaning and removal of the clay deposits was carried out rather quickly.

In this section, the gallery was heavily cracked and quite deformed, but it still retained its shape. The complete gallery was covered in sooth as the result of high temperatures which developed in explosion. Typical for this section is that in explosion the downstream vault section, probably in the bedrock and clay core contact, shifted for 10-20 cm in the downstream direction, Photo 10. In 65 cm long stretch of the access tunnel the gallery had caved-in, Photo 11.

The complete section was supported with steel arches and shotcreted, therefore the Type-2 remediation procedure was applied.

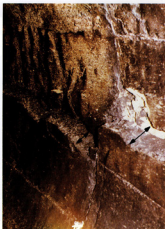


Photo 10. Gallery vault downstream shift of some 10-20 cm



Photo 11. Caved-in gallery in station 3+42.00



Photo 12. Clay deposits in gallery

Gallery Section from Station 2+77.00 Towards the Bottom and Central Dam End to Station 2+00.00

The works from the joint with the access tunnel towards the bottom were performed under more difficult condition and at much slower pace than the earlier described works. The reasons were:

- steepness of the tunnel,
- large clay deposits, about 1.00 m thick; clay water content was very high, which caused difficulties with the loading, Photo 15,
- the confined available space was an obstacle for use of machinery, but for the winch-operated rail vans which removed the clay, so the clay was manually dug and loaded,
- the rail van speed was limited to 6 m/min, so the van lowering and lifting was time-consuming.

The damage scale and nature in this part of the gallery was similar the gallery damage in the right end. The 10-20 cm shift was noticed downstream the left gallery vault, accompanied with very large predominantly longitudinal cracks, Photos 13 and 14. The complete gallery was sooted as the aftermath of the high temperatures developed in the explosion.

The lining was supported by steel arches and mesh reinforcement, and a shotcrete layer was placed, so the Type-2 remediation procedure was applied, Photo 15.

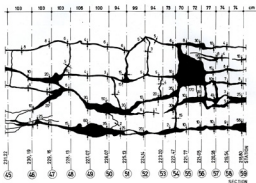
Gallery Section between the Stations 2+00.00 and 1+33.00

The gallery vault in station 2+00.00 was completely demolished because the dam core caved in, Photo 17. The concrete parts of the vault fell on the bottom, and were covered with well compacted clay from the core. The traces of fire were encountered everywhere. The same situation was to the station 1+33.00 where the first stage of remediation works was finished.

The works in this part of the gallery, including the works in the section between the stations 1+14.00 and 1+33.00 where the gallery was completely caved in and the reconstruction was of the tunnel type, was the most difficult segment of the entire Peruća Dam remediation.

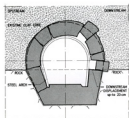
Here, the caved-in clay was excavated, along with the concrete blocks and other caved-in material, the supporting with steel arches and plates was completed and 80 cm concrete lining placed in 4-6 m segments, Photo 18. The works in this part of the gallery advanced at a very slow pace, about 40 cm a day in average, in three shifts.

FLAT VIEW



DEFORMED GALLERY LINING

SECTION 49



SECTION 57

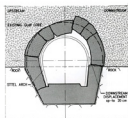


Figure 6. Gallery concrete lining damage and deformations



Photos 13 and 14 Concrete lining damage in the central grouting gallery section



Photo 15. Gallery supported with the steel arches



Photo 16. Shotcrete lining

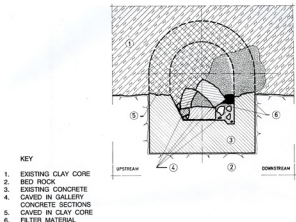


Figure 7. Caved-in gallery section

Excavation of material was particularly painstaking. Because the available space was confined, the works were performed manually, with occasional use of small vibrating hammers with special shovels for clay. Special problem were the caved-in gallery lining concrete blocks which were to be manually loaded in the rail vans, Photos 20 and 21.

The additional problems were caused by the water seepage into the gallery and its collecting in the excavation bottom. It had to be pumped out.

The water leaked into the gallery because the reservoir level was maintained at the EL 325-330 m a.s.l. to enable the Peruća HPP operation. The maximum water inflow was 130 l/s at the reservoir elevation of 330.00 m a.s.l. 50 l/min was concentrated on EL 321.80 m a.s.l. Typically, the seepage was noticed in the cracks in the joint between the lining and bedrock. The leaking water was completely clear, with no traces of washed out material, which indicates that the water passed through the fissures in the bedrock, Photo 22.



Photo 17. Supporting the excavation site with steel arches and plates - the caved-in concrete blocks are visible

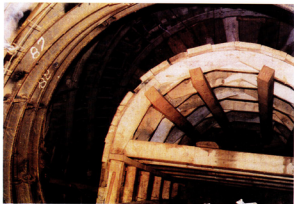


Photo 18. Lining concreting formwork in completely demolished part of the gallery



Photo 19. Reconstruction of the completely demolished gallery with 80 cm concrete lining



Photo 20. Manual excavation of the caved-in gallery part



Photo 21. Loading excavated material on rail vans

In station 1+95.00, the dia. 1.0 m cavern was encountered in the dam clay core, above the gallery apex. In order to avoid troubles with any more water filled caverns directly connected with the reservoir, the decision was brought to drill three frontal 10 m deep boreholes before excavation of each section is started. This was expected to reveal any potential caverns and protect the working site.

Specially large damage was discovered on a stretch between stations 1+52.00 and 1+62.00. In this stretch no gallery walls existed any more, the bottom was heavily damaged, and the traces of burned concrete were scattered around. This was probably the explosive planting site, Photo 23.

A 10 m long section of the drain pipe in the station 1+42.00 completely caved-in, while the remaining part was preserved and trapped in clay. The caved-in part of the drain pipe was remediated and the rest cleaned from clay.



Photo 22. Water seepage into the gallery, El. 321.80



Photo 23. Burned concrete

1. GROUTING GALLERY
2. EXCAVATION IN CAVED-IN DRAINAGE HOLE
3. DRAINAGE PIPE
4. DRAINAGE SHAFT
5. SWITCHYARD
6. DRAINAGE HOLE EXCAVATION CROSS SECTION
7. REMEDIATED CAVED-IN GALLERY PART CROSS SECTION

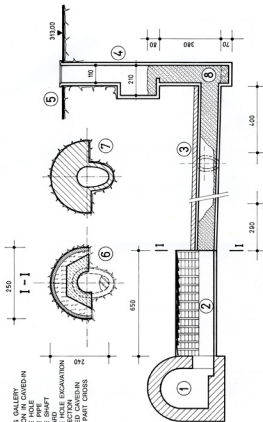


Figure 8. Remediated drainage pipe

Gallery Section between the Stations 3+42.00 and 3+84.00

This part of the gallery was remediated after the right dam end had been removed. The works started on July 15, 1995.

Here, the gallery was partly demolished in station 3+42, and the rest had cracks in the vault, Photo 24. The access tunnel No. 1 was completely demolished.

The gallery vault was reconstructed by 80 cm thick plain concrete. The gallery vault and old walls were fixed by anchors. The remaining parts of the gallery were cleaned from soot by sandblasting.

The access tunnel No. 1 was completely reconstructed, Photo 25.

Gallery Section between the Stations 0+40.00 and 1+33.00

The works in the gallery between the stations 0+44.00 and 1+33.00 started on July 1, 1995, after the left dam end had been removed. These were the remediation second stage works.

The gallery damage in the excavated section was very heavy, so during the excavation works it caved-in, and the part of the gallery and access tunnel No. 3 in the explosion site were completely demolished, Photos 26 and 27. This part of the gallery and the access tunnel No. 3 were completely reconstructed, Photos 28 and 29.

The gallery part in central dam section, stations 0+91.50 and 1+16.30, was heavily damaged and completely filled with clay and filter material. The Type-2 remediation method was applied with supporting and shotcrete lining, Photo 16.

Between the stations 1+16.30 and 1+33.00 the gallery was completely demolished, so the Type-3 remediation method was applied. The steel arches and plates, along with the 80 cm concrete lining were used as a support.

Between the stations 1+91.50 and 1+33.00, the problems the workers encountered were of the same type.



Photo 24.
Partly demolished
gallery in the right dam end



Photo 25.
Part of reconstructed
access tunnel No. 1



Photo 26. Demolished gallery in the left dam end

On June 26, 1995, the gallery remediation to the station 1+24.00 was completely finished. The excavation started on both sides of the gallery, to the station 1+24.00 to the left and station 1+33.00 to the right. The workers met in the station 1+28.00, and the left and right parts of the gallery joined on December 6, 1994, Photos 33, 35 and 36.

Contact Grouting

After the gallery remediation had been completed, the contact grouting started. It was aimed at obtaining the contact between the clay core and the gallery and fixing of the cracked gallery lining, primarily in the vault. This topic is discussed in another paper.



Photo 27. Demolished gallery in the left dam end



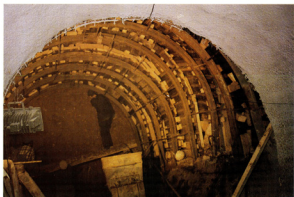
Photo 28. Grouting gallery reconstruction in the left flank



Photo 29. Grouting gallery reconstruction in the left dam end



Photo 30. Partly reconstructed gallery



Photos. 31 and 32. Gallery reconstruction by supporting and shotcreting from the left dam end



Photo 33. Tunnelling gallery



Photo 34. The deepest part of the gallery in the central part



Photos 35 and 36.

Connecting gallery back vaults

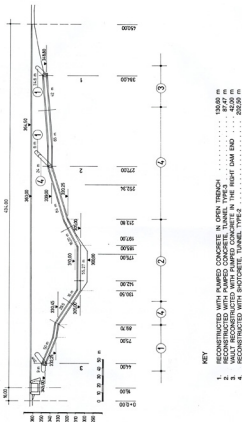


Figure 9. Grouting gallery remediation per sections

QUANTITY OF WORKS

Totally 462.57 m of gallery and access tunnel was reconstructed, wherefrom

- concreting in the open trench	130.60 m
- tunnel type concreting, Type-3	87.47 m
- concreted vault in the open trench in the right end	42.00 m
- shotcreting, tunnel type, Type-2	202.50

1,840 m³ of different material - clay, filter material, concrete blocks - was excavated and removed from the gallery.

1,560 m² of indoor gallery surface was shotcreted to remove soot.

The below quantities of material were built in

- stilted steel arch	4,700 m
- mesh reinforcement Q-131	3,265 m ²
- ribbed reinforcement 400/500	12,800 kg
- pumped concrete MB-30	2,100 m ³
- shotcrete	1,730 m ²

CONCLUSION

The Peruća Dam grouting gallery remediation was the most challenging task and, with respect to safety of the crews, a high risk activity. The working conditions were extremely difficult, and only the devotion of all participants enabled successful completion of this task with no accidents.

Since the contact grouting was finished, practically no seepage was noticed in the gallery, and there are no cracks and deformations. This indicates that the gallery remediation was successful.

DAM REMEDIATION GROUTING

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SPLIT

ABSTRACT

The grouting works are described which have been performed for remediation of possible damages in the grouting gallery under the dam, filling in the cracks and cavities in the fissured concrete gallery lining; achieving impermeability in the joint between the concrete gallery lining and the bedrock, namely the existing clay core; realization of continuity of a cut-off wall role as an impermeable dam element to the grout curtain, and achieving the best quality bond between the newly built core and the dam foundation surface.

These works have been successfully completed, because at the reservoir elevation of 353 m a.s.l., which is 85% of the maximum hydrostatic pressure, the water seepage into the grouting gallery is about 0.3 liters/s.

INTRODUCTION

Multipurpose application of grouting as a modern building method which improves mechanical characteristics of structural elements and soil has reached its full confirmation during the Peruća Dam remediation works.

The dam core grouting with cement-bentonite slurry was aimed at decrease in the slurry loss risk and increase in trench shear resistivity during construction of the cut-off wall, as described in the paper titled "Dam Core Grouting".

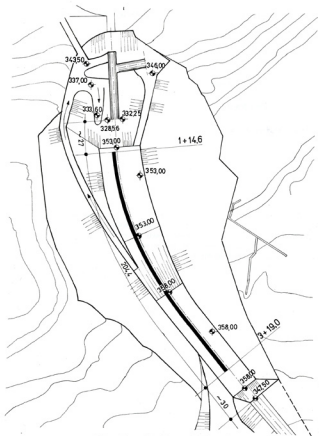


Figure 1. Grouting curtain layout

The present paper shall focus on other grouting applications, i.e.

- remediation of the existing grout curtain under the dam, and the contact surface between the grouting gallery and the foundation ground;
- grouting of the concrete grouting gallery lining damaged in explosion, and the lining contact with the bedrock, namely clay core,
- grouting of a joint between the cut-off wall and bedrock, namely grouting curtain.

REMEDICATION OF THE EXISTING GROUT CURTAIN UNDER THE DAM

Due to hydrogeological relations on the dam profile, the construction of the reservoir and realization of impoundment requested construction of a quite large grout curtain.

In the Cetina river canyon, downstream the dam, a three-row 450 m long and 200 m deep curtain was constructed. To extend the Cetina dam profile, two- and single-row curtains were constructed, 570 m long and 150 m deep in the left flank, and 580 m long and 100 m deep in the right flank, respectively, Fig. 1. Therefore, the total length of the curtain is 1,600 m, and its total area is about 260,000 m². 164,000 m of boreholes was drilled, of which 140,000 m was grouted. 51,393 t of dry matter consisting of clay, cement, bentonite, sodium and fine Dolomitic sand was consumed. The average grout mix consumption was 368 kg per meter of grouted borehole, namely 198 kg per square meter of the grout curtain.

The measured water seepage from the reservoir is about 0.7 m³/s, which is 1.7% of the mean annual flow of the Cetina at the dam profile. This speaks of successful grout curtain construction and verifies impermeability of the Peruća reservoir.

According to Deer (1976), the average grout mix consumption is high, which is characteristic for fissured and permeable rocks. Since the dam explosion was caused by blasting of explosive planted in the grouting gallery, a justified suspicion arose that the integrity of the curtain under the dam might be disturbed, although in this section the curtain was three-row, with 5.5 m interval between the external-row boreholes and 4 m distance between the rows.

In order to get the general information about the damage, the curtain impermeability under the dam was tested by Lugeon method in 9 boreholes arranged along the grouting gallery. The boreholes were 25 m deep, which was the first approximation of the depth to which the energy released in explosion could cause reopening of the fissure systems

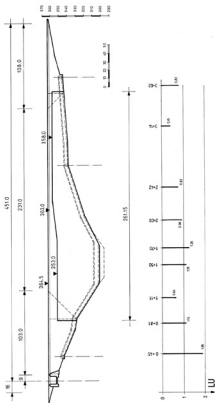


Figure 2. Results of the grout curtain permeability investigations

closed by the original curtain grouting. The Fig. 2 shows the arrangement of the boreholes along the dam and the maximum permeabilities measured in each borehole. The results indicate that the curtain permeability ranges between 0.45 and 1.96 LU, namely within the limit criteria of 2 LU set for the curtain construction.

The obtained data indicated that, generally, the curtain damage was not high, but the quantity of data was not representative of the complete 450 m long curtain section.

Therefore, a decision was brought to undertake continuous grouting along the entire dam length, at the depth of 10 m from the foundation joint between the gallery bottom, namely clay core and the bedrock, in the dam section in which the gallery was not constructed.

Grouting Procedure

The grouting was carried out in two 1.4 m apart rows, upstream and downstream. The distance between the boreholes in a row is 4 m, and between the boreholes in different rows 2 m, Fig. 3.

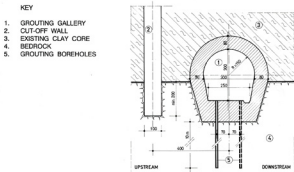


Figure 3. Boreholes arrangement

Each borehole was grouted in two 5 m deep stages. In this way, not only the top 10 m of the curtain was grouted, but also the contact between the gallery bottom concrete lining and the bedrock which, we were almost certain, had suffered a damage in the explosion.

Additionally, the foundation joint between the clay core and the bed rock was grouted in right dam end which was to be reconstructed, stations 3+83 to 4+67.

The grouting works in the left dam end, off the gallery, including the joint of the spillway structure and the clay core, were to be performed after the dam and spillway structure reconstruction has been completed.

The cement-bentonite slurry was used for grouting. The dry matter contained 96% cement and 4% bentonite, and the initial dry matter and water ratio was 1:3, which resulted in the slurry density weight of 1.11 kg/dm³.

The grouting was conducted under the pressure of 6 bar, which corresponds with the pressure of the maximum reservoir water column of 61.5 m. We suspected that higher pressure would disturb integrity of the curtain because the mechanical strength of the binding material would be exceeded.

The grouting completion criterion was that the slurry consumption under 6 bar should be below 20 l in 20 minutes. In case it is not possible, the mix ratio had to be decreased to 1:0.7.

Grouting Results Analysis

220 boreholes 2,198 m long were drilled, and 60,153 kg of cement and bentonite was consumed.

The grouting criteria were met in all boreholes on both stages. At the end of the grouting, the maximum slurry consumption at the **lower stage** was 12 l in 20 minutes, on the borehole in station 4+41 where the maximum slurry consumption of 5,944 l was recorded namely 358 kg of the dry matter per meter of the borehole.

The maximum consumption in the **upper stage** at the end of the grouting was 8 l in 20 minutes in the borehole on station 3+81, where the maximum slurry consumption of 9,095 l was recorded, namely 959 kg of the dry matter of the borehole.

The grouting results are presented by the mean dry matter consumption T (kg) per meter of borehole in Fig. 3, and the lower and upper stages are presented with plots 1 and 2.

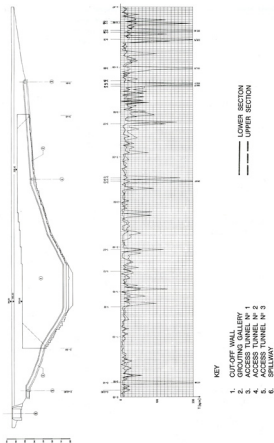


Figure 4. Grout curtain remediation under the dam

The **lower stage** boreholes length is 1,100 m, the cement and bentonite consumption 26,559 kg, thus the average dry matter consumption was 24.14 kg per meter of grouted lower stage. The plot 1 in Fig. 3 leads to the conclusion that the average consumption is comparatively low and uniform, which confirms the impermeability test results which indicated that the curtain was not damaged in explosion.

The exception are the boreholes in stations 2+73, 3+81, 3+99, 4+31, 4+41 and 4+49, in which the recorded consumption was equal or above 200 kg of dry matter per meter of grouted stage, the maximum being 357.83 kg.

These values cause high coefficient of variation for the average dry matter consumption in the lower stage of $c_v=1.92$. However, high positive value of the skewness coefficient of $\alpha_3 = 4.91$ shows that the frequency of the below-average consumption in all the lower stage boreholes remains significant. This is particularly visible in the dam part in which the gallery is constructed, with the exception of the stages in stations 2+73 and 3+81.

A substantially increased slurry consumption in station 3+81 can be explained by the fact that the explosive had been planted in this point. The energy released in explosion had damaged the bedrock and, consequently, the curtain, demolished the gallery and caused opening of 20-25 m dia. and 6-7 m deep craters in the dam. The bedrock had probably been damaged because in this point the gallery is cut into the bedrock for 2-3 m, so the explosion-induced elastic seismic waves had been transmitted to the bedrock.

Similar effects were not recorded in the dam explosion site in the left end, station - 0+40, because in this point the gallery is not cut into the bedrock. Thus, the explosion energy had been transmitted to the dam body, as the medium of significantly lower stiffness than the bedrock, and caused opening of 25-30 m dia. and over 10 m deep craters.

The same explanation the low consumptions at the highest, central dam section, stations 1+32 to 1+72, where the explosive was also blasted in the gallery. The gallery had been founded on 7-8 m thick block. The explosion completely demolished the gallery flanks and vault, and the gallery was completely filled with clay from the core. The effect of the explosion on the dam crest was its initial settlement of about 2-3 m in approximately 50 to 60 m long stretch.

The average slurry consumption in the dam section where there was no grouting gallery, stations 3+85 to 4+67, was the result of the bedrock degradation caused by the excavation for the dam core foundations.

33,594 kg of cement and bentonite was consumed for 1,098 m of boreholes in the **upper stage**. Therefore, the average dry matter consumption was 30.59 kg per grouted meter of

the upper stage, which is approximately 27% more than for the lower stage. The difference may be explained by increased slurry consumption in the contact between the bedrock and concrete gallery lining which had been disturbed in explosion.

The average slurry consumption plots in Fig. 3 lead to the conclusion that the upper stage consumption, plot 2, are less uniform than the consumption in the lower stage, plot 1, as supported by higher coefficient of variation of $c_v=2.40$. High skewness coefficient $\pi_3 = 9.97$ indicates high frequency of the below-average consumption of 30.59 kg per grouted meter of the upper stage for all the boreholes.

The average consumption is very high in stations 0+49 and 3+81, e.g. 327.86 and 958.51 kg per meter of borehole, respectively. Since these were the explosion sites, there is no doubt the consumption is high because the seriously damaged contact between the concrete gallery lining and the bedrock was grouted.

The average slurry consumption plot for the upper stage clearly shows the point of damage in the contact between the concrete gallery lining and the bedrock, which is particularly pronounced between the stations 1+35 to 1+55, namely in the central dam section explosion site and in stations 1+97, 2+77 and 3+37.

The increased slurry consumption in the dam section which does not include the grouting gallery, between the stations 3+85 and 4+67, have been caused by the bedrock degradation during the dam core foundation excavation and need to fill in the cavities in the contact between the core and the bedrock which had not been filled with clay during the core construction.

Conclusion

The grouting results have shown that the grouting gallery had not been destroyed in dam explosion, and the points in which the contact between the concrete gallery lining and the bedrock had been disturbed were successfully remediated.

However, the additional grouting is planned at greater depths and under higher pressures in order to obtain better information on the curtain condition in the points of increased consumption.

20 m deep boreholes are planned to be grouted under maximum pressure of 15 bars in station 2+75; since the hydrostatic pressure of stored water is below 14 m w.e., the 15 m deep boreholes are planned to be grouted under maximum pressure of 10 bars in stations 3+79, 3+99, 4+31, 4+39 and 4+51.

GROUTING BEHIND THE CONCRETE GALLERY LINING

The grouting gallery was reconstructed in 365 m long stretch, from the third access tunnel in the left, and to the first access tunnel in the right dam end, Fig. 8.

Generally, two types of reconstruction were applied in the gallery.

In the gallery section where the concrete lining had been completely destroyed, the new lining was placed along the entire perimeter or anchored to the remains of the old lining sides or bottom slab. About 186 m of gallery was reconstructed in this way, Figs. 5 and 6.

In the section where the concrete lining had cracked, but the gallery still retained its internal form, the reconstruction was carried out using the steel arch supports placed in 65 cm intervals, with 15 cm thick shotcrete layer and double steel mesh reinforcement. In this way about 179 m of gallery was reconstructed, Fig. 7.

Grouting Purpose

Grouting behind the concrete grouting gallery lining was intended to provide for an impermeable layer between the clay core, namely bedrock and the gallery concrete, in order to prevent water seepage and internal erosion process.

The gallery grouting had two purposes, depending on the applied reconstruction type.

In the section where the gallery was reconstructed with the new concrete lining, grouting was planned to fill in the cavities in the contact between the lining and the clay core, which had remained unfilled and thus realize the impermeable contact between the core and the gallery.

Additionally, in the section where the new lining bonds with the side walls of the old gallery, grouting was intended to achieve the best possible bond of the bedrock with the part of the gallery lining which was assumed to have suffered the explosion-induced damage.

In the section where the gallery was reconstructed using the steel arches and shotcrete, the grouting was planned to fill in the cracks and voids in the old lining and achieve the impermeable joint of concrete and clay core along the entire gallery perimeter.

- KEY**
1. GROUTING GALLERY
 2. CUT-OFF WALL
 3. NEW CLAY
 4. EXISTING CONCRETE
 5. NEW CONCRETE
 6. ANCHORS ϕ 20/50 cm
 7. GROUTING BOREHOLES
 8. BEDROCK

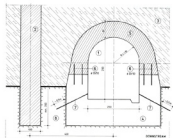


Figure 5. Grouting boreholes arrangement TYPE 1
stations: 0+49.00 - 0+91.50 and 3+42.00 - 3+85.00

The grouting was performed on the entire gallery length, but for 16 m long stretch in the left end, station 0+33 to 0+49.

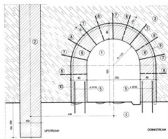
The initial section was not grouted because in this part it is not cut in the bedrock, so it was possible to place the clay and achieve a sufficiently impermeable joint between the dam core and the gallery concrete.

Grouting Procedure

The grouting behind the concrete gallery lining was carried out in 2 m apart transverse profiles.

In sections between the stations 0+49 and 0+91.5, and stations 3+42 do 3+85, where the gallery reconstruction included construction of the new lining, only the joint between the old lining and the bedrock was grouted through two boreholes drilled one on the upstream and one on the downstream side of the gallery, type 1, Fig 5.

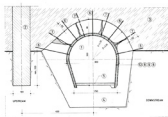
The new part of concrete lining was not grouted because the new clay core was constructed offering quality joint between the gallery concrete and the dam core.



KEY

1. GROUTING GALLERY
2. CUT-OFF WALL
3. EXISTING CLAY
4. EXISTING CONCRETE
5. ANCHORS ϕ 20/50 cm
6. NEW CONCRETE
7. GROUTING BOREHOLES
8. GROUTING BOREHOLES IN ADJACENT PROFILES
9. STEEL PLATES
10. STEEL ARCHES

Figure 6. Grouting boreholes arrangement TYPE 2
stations: 1+16.30 - 2+00.00



KEY

1. GROUTING GALLERY
2. CUT-OFF WALL
3. EXISTING CLAY CORE
4. EXISTING CONCRETE
5. STEEL BARS I 100
6. GROUTING BOREHOLES
7. GROUTING BOREHOLES IN ADJACENT PROFILES
8. STEEL MASH
9. STEEL ARCHES
10. SHOTCRETE

Figure 7. Grouting boreholes arrangement TYPE 2
stations: 2+00.00 - 3+42.00

The grouting between the stations 0+91.5 and 1+16.3, and stations 2+00 to 3+42, where the gallery was reconstructed using the steel arches and shotcrete, was performed with five, namely six boreholes arranged along the perimeter of the gallery sides and vault, type 2, Fig. 7,

The same arrangement of boreholes was prepared for grouting of the central section, between the stations 1+16.3 and 2+00, where the gallery was reconstructed by construction of the new lining, type 2, Fig. 6, because it was necessary to fill in all the voids between the new lining and the old clay core which remained unfilled in this part. Additionally, this gallery section is in the area of the maximum hydrostatic pressures, so the top quality joint was requested to prevent both seepage and internal erosion process.

The cement-bentonite slurry was used for grouting; the dry matter contained 96% cement and 4% bentonite, and the initial mix ratio was 1:2, which results in the slurry density of 1,16 kg/dm³.

The grouting pressure was 3 bars. The final grouting criterion was that the slurry consumption at 3 bars should be less than 20 l in 20 minutes. In case it is not possible, the mix ratio had to be decreased to 1:0.7.

Grouting Results Analysis

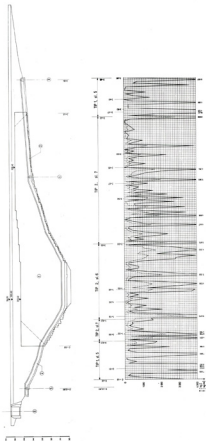
Since the purpose of grouting was different for particular sections, the results were analyzed depending on the type of lining used for gallery reconstruction and arrangement of boreholes in the grouting profiles.

The dry matter T (kg) consumption per grouting profiles is presented in Fig. 8.

In sections where the contact between the old lining and the bedrock was grouted, stations 0+49 to 0+91.5 and 3+42 to 3+85, grouting was conducted in 42 profiles. 102,652 kg of cement and bentonite was used, thus the average dry matter consumption per profile was 2,444.1 kg.

The consumption in different profiles is not uniform, and it shows a trend of higher frequency of the below-average consumption. The conclusion is derived from the coefficients of variation and the skewness coefficient which are $c_v = 1.41$ and $\alpha_3 = 2.02$, respectively. The maximum consumption in the grouting profile was 16,412 kg in station 3+47, and the minimum of 21 kg in station 3+61.

The reason for such a high consumption is seriously damaged rock, and consequently completely disturbed contact between the concrete lining and the bedrock.



KEY

1. CUT-OFF WALL
2. GROUTING GALLERY
3. ACCESS TUNNEL N° 1
4. ACCESS TUNNEL N° 2
5. ACCESS TUNNEL N° 3
6. SPILLWAY

— TOTAL DRY MATTER CONSUMPTION PER PROFILE (kg)

- - - DRY MATTER CONSUMPTION IN PROFILE PER METER OF BOREHOLE (kg/m)

Figure 8. Grouting behind the concrete gallery lining

All the boreholes met the grouting completion criteria. The maximum slurry consumption on completion of grouting was 10 l in 20 minutes with the mix ratio increase to 1:1.5, recorded in the station 0+61.

In sections where **the old cracked gallery lining and its contact with the existing clay core was grouted, stations 0+91,5 to 1+16,3 and stations 2+00 to 3+42**, the grouting was performed in 86 transverse profiles. 111,115 kg of cement and bentonite was used, so the average consumption per grouted profile was 1,292.03 kg.

Almost twice lower consumption per profile, as compared with the sections where the contact between the old lining and the bedrock was grouted, can be explained by the fact that during the gallery reconstruction the cavities in the old concrete lining had to be filled in with shotcrete.

The consumption in these sections is not uniform, and it shows a trend of higher frequency of the below-average consumption. The conclusion is derived from the coefficients of variation and the skewness coefficient which are $c_v = 1.70$ and $\alpha_3 = 2.08$, respectively. The maximum consumption in the grouting profile was 6,736 kg in station 0+99, and the minimum of 33 kg in station 2+85.

All the boreholes met the grouting completion criteria. The maximum slurry consumption on completion of grouting was 14 l in 20 minutes recorded in the station 2+67, with no decrease in the mix ratio.

In the central part of the gallery, in the contact between the new lining and the old clay core, **station 1+16.3 to 2+00**, the grouting was performed on 41 profile with the average consumption of 1,948.93 kg of dry matter per profile. Total consumption of cement and bentonite was 79,90 kg.

The reason for high average consumption per profile is the presence of cavities between the concrete and core, which had not been filled in during placement of concrete. With respect to the exceptionally difficult working conditions, such grouting results were expected.

The consumption in different profiles is not uniform, and it shows a trend of higher frequency of the below-average consumption. The conclusion is derived from the coefficients of variation and the skewness coefficient which are $c_v = 1.01$ and $\alpha_3 = 1.94$, respectively. The maximum consumption in the grouting profile was 10,270 kg in station 1+19, and the minimum of 106 kg in station 1+35.

All the boreholes met the grouting completion criteria. The maximum slurry consumption on completion of grouting was 10 l in 20 minutes recorded in the station 1+49, with no decrease in the mix ratio.

Conclusion

The grouting was performed on 169 profiles, in 787 boreholes with total length of 393.5 m. 293,673 kg of cement and bentonite was used, and the average consumption per grouted profile was 1,737.41 kg, per borehole 373.16 kg, and per meter of borehole 746.31 kg.

All the grouted sections, regardless of the type of lining used to reconstruct the gallery, are typical for large difference between the maximum and minimum consumption per profile, variation in consumption of the neighboring profiles and tendency of frequent below-average consumption.

These characteristics are actually the same for all grouted sections, as indicated by the coefficients of variation and skewness which are quite similar.

The large difference between the maximum and minimum consumption and the variation in consumption between the grouting profiles can be explained by small distances between the profiles which caused overlapping of the grouting extent in the neighboring profiles. The said could lead to the conclusion that the distance between the grouting profiles could have been larger. In that case the consumption variations would be smaller, and the scale of damage in lining and the contact between the lining and the bedrock, namely the clay core along the gallery, better pronounced.

CUT-OFF WALL AND BEDROCK JOINT GROUTING

Since the dam explosion caused heavy damage of the clay core, the remediation design planned construction of a plastic concrete cut-off wall in the dam clay core. In this way, the cut-off wall and the clay core form an impermeable dam element in this dam section.

The cut-off wall was constructed between the stations 0+87.70 and 3+48, 4 m upstream the gallery axis. The cut-off wall is 261.15 m long, 1 m wide, with surface area of 8,505 m². The maximum cut-off wall depth of 51.5 m is in the central dam section, and minimum depth of 12.9 m is in its right hand end. The minimum cut-off wall foundation depth in the bedrock is 2 m.

Grouting Purpose

In order to achieve the continuity of the cut-off wall function as an impermeable dam element, the bedrock was grouted between the gallery and the cut-off wall and under the cut-off wall foundation.

In this way, the rock was consolidated with respect to filling in of both natural and the cracks caused during the trench excavation by a hydraulic cutter, and the impermeable joint was realized between the cut-off wall and the grout curtain.

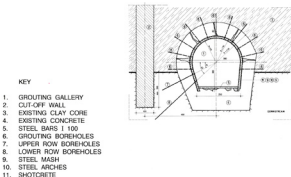


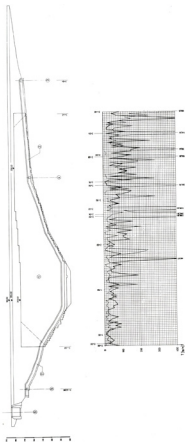
Figure 9. Boreholes arrangement

Grouting Procedure

The grouting was performed from the gallery in two rows of boreholes, **upper and lower**, set under variable angles, depending on the depth of the cut-off wall foundation, Fig. 9. The distance between the boreholes in the upper and lower row is 2 m.

The cement-bentonite slurry was used for grouting; the dry matter contained 96% cement and 4% bentonite, and the initial mix ratio was 1:2, which results in the slurry density of 1.16 kg/dm³.

The grouting pressure was 4 bars for the upper row, and 5 bars in the lower row.



KEY

1. OUT-OFF WALL
2. GROUTING GALLERY
3. ACCESS TUNNEL N° 1
4. ACCESS TUNNEL N° 2
5. ACCESS TUNNEL N° 3
6. SPILLWAY

GROUTING BOREHOLES

- UPPER ROW
 --- LOWER ROW

Figure 10. Contact grouting between cut-off wall and bedrock

The final grouting criterion was that the slurry consumption at 4 bar and 5 bar should be less than 20 l in 20 minutes. In case it is not possible, the mix ratio had to be decreased to 1:0.7.

Grouting Results Analysis

131 profiles were grouted; the length of the upper boreholes was 163.2 m, and of the lower 358.2 m. The consumption of cement and bentonite was 20,708 kg in upper and 27,294 kg in lower boreholes, thus the average dry matter consumption was 126.89 kg and 76.2 kg per meter of upper and lower boreholes, respectively.

The 1.67 times higher consumption in the upper boreholes could be attributed to the grouting sequence, since the upper boreholes were grouted first, and to higher scale of cracking in the bedrock in the area of the upper boreholes.

All the boreholes met the grouting completion criteria. The maximum slurry consumption on completion of grouting was 10 l in 20 minutes recorded in the station 2+66, with no decrease in mix ratio.

The average dry matter consumption T (kg) per meter of the upper and lower row boreholes is shown in Fig. 10.

The average consumption per meter of the **upper row** boreholes varied between 7.17 kg in station 2+70 and 5,036.4 kg in station 2+40. The exceptionally high consumption was also recorded on the profiles in stations 2+32 and 2+34, i.e. 2,269.38 and 2,842.33 kg per meter of borehole, respectively. The large consumption variations are indicated by the coefficient of variation of $c_v = 3.47$. The trend of higher frequency of the below-average consumption is characteristic, since the skewness coefficient is $\alpha_3 = 7.40$.

The average consumption per meter of the **lower row** boreholes varied between 3.08 kg in station 0+90 and 1,584.65 kg in station 2+98. The exceptionally high consumption was also recorded on the profiles in stations 2+66, 3+24 and 3+48, i.e. 1,330.70, 1,146.30 and 889.95 kg per meter of borehole, respectively. The large consumption variations are indicated by the coefficient of variation of $c_v = 2.37$. The trend of significantly high frequency of the below-average consumption characteristic, since the skewness coefficient is $\alpha_3 = 4.98$.

Conclusion

It is characteristic for both borehole rows that the average dry matter consumption considerably varies per meter of borehole, with a trend of increase from the left to the right cut-off wall end. Also, both borehole rows are characteristic for frequent consumption below the average.

It is obvious that the extremely high consumption in the upper and lower grouting row does not correspond.

CONCLUSIONS

The basic objective of the grouting works was remediation of all damaged sections of the grout curtain under the dam, filling in of cracks and cavities in the concrete gallery lining, realization of an impermeable contact between the concrete gallery lining and the bedrock, namely existing clay core, continuity of the cut-off wall as an impermeable dam element to the grout curtain, and realization of the best possible contact between the newly constructed core and the dam foundation surface.

In this way, the seepage in the contact surfaces between different materials is prevented along with the internal erosion. Therefore, it is particularly important that the realized joints were of the best possible impermeability. The best indication of the achieved result is the water seepage quantity.

The Peruća Dam grouting gallery is constructed in about 79% of the dam crest length, in the part where the hydrostatic pressures are maximum and range between 12.6 and 56.5 m. For that reason the seepage quantity in the gallery, at the maximum reservoir level, can be used as one of the best indicators of the project success.

When the reservoir level is on the El. 353 m a.s.l., total seepage into the gallery is estimated to be 0.3 l/s. At this reservoir level the hydrostatic pressure in the lowest dam section is about 85% of the maximum, and in the highest access tunnels Nos. 1 and 3 it reaches 32% and 45% of the maximum hydrostatic pressure, respectively. Therefore, there is no doubt that the performed grouting works fulfilled their purpose, particularly because the gallery seepage before the explosion was about 1 l/s.

REFERENCES

- [1] "Filling materials for watertight cut-off walls", ICOLD Bulletin 51, International Commission on Large Dams, Paris, France, 1985
- [2] Lombardi, G., "Grouting design and control using the GIN principle", *Water Power & Dam construction*, June 1993
- [3] Nonveiller, E., "Grouting - Theory and Practice", Školska knjiga Zagreb, 1989

SPILLWAY CONSTRUCTION AND HYDROMECHANICAL EQUIPMENT

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ABSTRACT

The paper describes civil works on the spillway structure. The reasons for increase in the spillway flap height are explained. The hydromechanical equipment is briefly described, as well as an approach to the flap metal structure sizing.

The quality assurance system played a special role because the high accuracy was requested in flap manufacture and erection.

SPILLWAY CONSTRUCTION

When the former Yugoslav army blasted the Peruća Dam on January 28, 1993, one of explosion sites was immediately by the right spillway wall in the left dam flank. The explosion destroyed the right spillway wall, flap and the spillway bridge. In the dam crest, by the spillway, a crater was formed 25-30 m in diameter and about 10 m deep. The explosion was so strong that the parts of the spillway side wall got scattered within the 50 m radius, and a 12 m³ piece of structure was found 30 m far from the explosion site. The spillway bridge was thrown on the left bank, and remained hanging over the side wall, Photo 1.

The discharge capacity of the spillway was 340 m³/s at the reservoir level of 361.20 m a.s.l. It was regulated with 5.0 x 10.0 m flap.

The characteristics of the Cetina river were examined for development of the dam remediation design documentation. It was determined that the spillway discharge capacity should be increased to 427.93 m³/s if the 10,000-year flood is to be discharged at the

maximum operating reservoir level. As the consequence, the flood level increased for about 1.5 m, requesting the heightening of the dam crest and the spillway walls from the elevation 363.00 m a.s.l. to the elevation 364.50 m a.s.l. This lead to increase in the spillway flap height from 5.0 to 7.0 m.



Photo 1. Spillway after explosion

The design documentation on the dam ends and crest removal planned demolition and removal of concrete parts of the spillway destroyed in explosion.

These parts were removed during the first stage of the remediation works. After the dam and spillway heightening was defined by the design, the embedded lateral shields of the spillway flap had to be removed from the right and left spillway wall. Therefore, it was decided to remove the complete spillway concrete structure and build a new 25.54 m long one, along with the new gate house on the left platform, by the spillway wall, and new reinforced concrete bridge over the spillway, Figs. 1 and 2.

The Spillway Structure Remediation design was prepared by Elektroprojekt Consulting Engineers Ltd. Zagreb in 1995.

The works on removal of the existing concrete structure were intensified in January 1995. The structure demolition and removal was carried with drilling rods, blasting, cutting the reinforcement bars and embedded steel members, and removal from the site to the spoil area. When the steel structure was removed, the excavation in bedrock started for the flap hydraulic cylinder shaft and spillway slab.

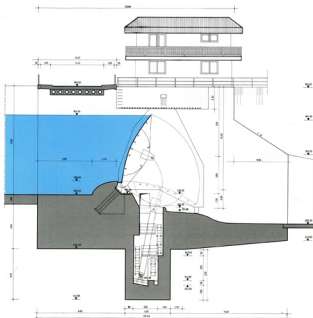


Figure 1. Spillway structure - longitudinal section

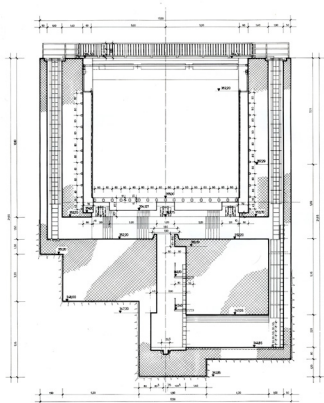


Figure 2. Spillway structure cross-section through the hydraulic cylinder shaft and air vents

In April 1995, the excavation was followed by concreting of the cylinder shaft and spillway concrete structure, Photo 2. The spillway concrete was MB-30, with ribbed reinforcement RA 400/500. The spillway concrete placement went on into two stages. In the first stage, the structure with the holes left for installation of the hydromechanical equipment was concreted. During this stage the anchors and anchor plates are embedded to which the equipment shall later be welded. The second stage includes concreting of the holes.



Photo 2. Spillway construction pit and placement of cylinder shaft reinforcement

The most sensitive works on the spillway slab included placement of anchors and anchor plates for the flap bearings and concreting of the reinforced concrete beam which is designed to accept the loads from the anchors and flap bearings. The design allowance the anchors installation was ± 5 mm. The geodetic survey during the anchors installation, immediately before and after concreting, enabled installation of anchors within the design allowances.



Photo 3. Flap bearing anchors and anchor plates

After completion of the spillway slab and spillway sill the concrete placement was continued for the spillway walls with the spillway air vents. In the right wall, in the joint with the dam clay core, a concrete wall was constructed for better bond between the concrete and clay.

As the works on the right spillway wall progressed, the material was being built in into the dam body in the immediate vicinity of the spillway, Photo 4.

Until early September 1995 the first stage of the spillway structure concrete placement was to be completed, and construction of the spillway bridge and gate house started. The hydromechanical equipment erection also started.



Photo 4. Spillway right wall and slab - Dam backfilling works in the back

HYDROMECHANICAL EQUIPMENT

Metal Structure - System Outline

The spillway flap enables controlled discharge of the water from the reservoir.

The metal flap is 10 m long and fitted with rubber seals on the sides. The seals are lead along the lateral metal shields embedded in the spillway walls.

The flap shape, depending on the flow requirements, and particularly the strength and stiffness requirements, is shown in Fig. 3.

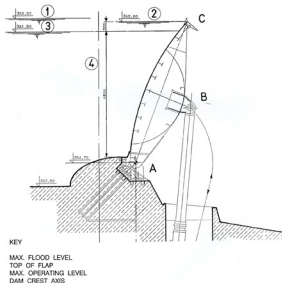


Figure 3. Flap cross-section, "closed" position

By rotation around the axis of rotation A (three bearings along the flap length) the flap top C (fitted with jet splits) is lowered or lifted and the water discharged from the reservoir.

The flap is moved by the hydraulic cylinder/piston rod acting in the force application point B during the flap "lifting", namely closing. The flap opening is controlled and exposed to water pressure and flap weight effect. Stringent functionality requirements for the sealing (water leakage in closed position ≤ 0.05 l/s per meter of sealing surface) include correct and even fitting of seals along the entire perimeter.

The typical seal details are given in fig. 4.

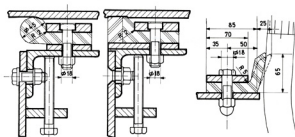


Figure 4. Seal installation - detail

The hydraulic cylinder is placed in a shaft, in the spillway bay central line, behind the spillway sill, Fig. 5.

The cylinder/piston rod data:

Effective stroke	5,265 mm
Maximum force	4,030 kN
Mass	8,500 kg
Nominal cylinder pressure	19 MPa (190 bar)

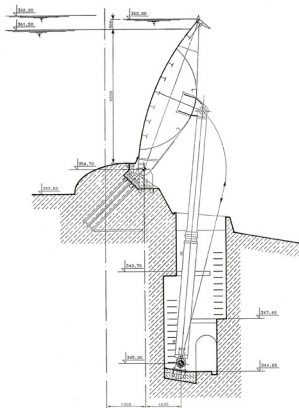


Figure 5. Flap with hydraulic cylinder

Controlled discharge, namely piston rod stroke control, is conducted from the gate house which accommodates hydraulic power units, electric motors, pumps, distribution valves, oil tank, oil filters, fittings etc.

The gate house also houses the distribution cabinet supplying the power for electric motors, HVAC system, sockets, signalization, metering instrumentation. The gate house is the central dam control room.

The spillway flap lateral shields are made of stainless steel. The sealed gate slides along them. They are fitted with the heat tracking system on the inside walls, immediately by the lateral shields, where the pipe bundles are installed. The pipes are filled with a liquid with low freezing point which shall be heated with submerged heaters. The heat transfer to the lateral shields shall prevent freezing and enable quality sealing during the winter season.

Sizing

The equipment is sized for the heavy duty operation. The DIN 19704 and DIN 19705 hydraulic metal structures requirements were applied which specify the load types and allowable stress and strain for particular elements.

The sizing was generally based on statistical calculations. For some elements, the dynamic loads and seismic forces were taken into account. Depending on the intended application, various materials were used according to DIN or European standards (St 37-2, St 37-3N, 6 Cr Ni Ti 1810).

Since the majority of equipment is welded, the welded joints calculations were done.

The assumptions for determination of loading were:

- the flap dead load is increased by the paint coats weight (wind effects and overgrowth are neglected),
- the hydrostatic and hydrodynamic loads are determined for maximum water levels; the hydrostatic loads for different flap positions are estimated from the analogy with model tests conducted previously for similar flaps;
- possible ice load was included through increase in the surface pressure,
- the seismic load was solved by including the Westergaard theory into the calculations,
- the temperature increment $\Delta T = 30$ K (the flap rotates around three bearings two of which enable axial displacement).

The calculation was done on a PC using the ANSYS 5.1 program. In sizing, namely selecting of the hydraulic cylinder/piston rod, special attention was paid to buckling and sizing of the bearing lugs force application point.

Quality Assurance and Quality Control (QA/QC)

A number of preplanned QA activities should be realized if the adequate reliability of a particular piece of equipment of system and its satisfactory operation is to be achieved.

Aware of the fact that the adequately and successfully prepared and introduced Quality Assurance Program is vital for quality and operational availability of the plant, the Croatian National Electricity engaged a specialized QA institution to do the work.

The QA/QC activities are planned to be conducted in the below stages:

- | | | | | |
|----|---|------------|-----|--|
| 1. | <div style="border: 1px solid black; padding: 2px; display: inline-block;">PREPARATORY
ACTIVITIES</div> | | 1.1 | documentation review
and preparation procedure |
| | | | 1.2 | defining QA requirements |
| | | | 1.3 | harmonization/approval of production
and inspection QC plans |
| 2. | <div style="border: 1px solid black; padding: 2px; display: inline-block;">EQUIPMENT
MANUFACTURE</div> | ABROAD | 2.1 | final testing and test witnessing |
| | | | 2.2 | equipment QA/QC documentation
verification |
| | | BRODOSPLIT | 2.3 | review of certificates (all) |
| | | | 2.4 | follow-up of machining, cutting, welding |
| | | | 2.5 | witnessing non-destructive testing |
| | | | 2.6 | witnessing final dimensional checks |
| | | | 2.7 | control of corrosion protection
procedures and parameters measurement |
| 3. | <div style="border: 1px solid black; padding: 2px; display: inline-block;">PRIOR TO
ERECTION
ON-SITE
TAKEOVER</div> | | 3.1 | erection documentation review |
| | | | 3.2 | comments on testing and takeover
procedures (from BRODOSPLIT) |
| | | | 3.3 | determining quality requirements
- QC plans approval |

- | | | |
|----|---------------------------------|---|
| 4. | ERECTION
TESTING
TAKEOVER | 4.1 supervision (during testing) of equipment installation and erection
4.2 attending final test/takeover
4.3 attending system final acceptance
4.4 review of Q/A/QC documentation and reports of erection, testing and takeover inspections |
| 5. | FINAL REPORT | |

CONCLUSION

The below conclusion can be drawn from the above overview of the spillway and hydromechanical equipment:

- The spillway damaged in the explosion requested removal of the concrete sections and metal structure and subsequent design and construction of the new structure. During construction, special attention has been paid to accurate installation of anchors for the flap bearings.
- The spillway is sized for the heavy duty operation.
- Very stringent requirements, particularly the permitted flood discharge, ask for very strict allowances, namely very precise accuracy in manufacture and erection. Consequently, special attention has been paid to the quality assurance which would provide for reliable and long flap life.

REFERENCES

- [1] Statical Calculation, MAN-GHH, Drawing No. 198-4-001907, 1995
- [2] Quality Assurance Philosophy in Construction of Conventional Power Projects, EKONERG Ltd., Zagreb, 1993

DAM MONITORING SYSTEM

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ZAGREB

ABSTRACT

The Peruća Dam is a large dam for which a comprehensive monitoring program has been prepared. The dam being a specific structure, heavily damaged and subsequently remediated, the planned scope of monitoring is somewhat more extensive than it is common for similar structures.

The monitoring system concept is based on maximum use of automatics and integration of all the measuring equipment into a centrally controlled system.

INTRODUCTION

The Peruća Dam is a large dam for which a comprehensive monitoring activities program has been prepared, and a complex system of monitoring instrumentation and equipment designed and set up.

Generally, there are several reasons for dam monitoring, but the principal reasons are safety, cost-effective maintenance and gaining of experience to be applied to improve the design engineering and construction of this type of projects.

With respect to public security, the dam monitoring should determine whether the dam behavior is within the set standards, whether any deformation or displacement occurred which could indicate disturbances in the structure, foundations or surrounding area threatening the dam stability.

The dam maintenance is one of the reasons for systematic monitoring, because the repair works on this type of projects are extremely expensive and sensitive. Timely observation and correction of any damage or failure, before the progress is full, is important because even the minor faults could often lead to major damage.

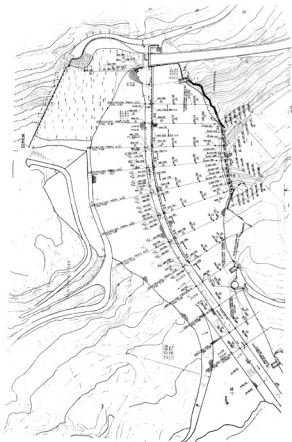
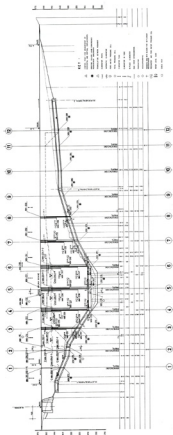


Figure 1. Monitoring posts on the Perofa Dam - layout



The systematic monitoring of this type of structures in operation and the data obtained by measurements, even if only by visual inspections, are very useful for gaining of new experience which shall contribute to improvement in design engineering, construction activities and utilization of the newly built dams of the same type.

Before the explosion, the Peruća Dam was permanently monitored. The monitoring instrumentation and equipment was state-of-the art at the dam construction time, but modest compared to the system installed now. It consisted mainly of the geodetic survey instruments and piezometers. Almost entire monitoring system was destroyed in explosion, so only the micro triangulation network and the reservoir staff gages could be included in the new system. However, it should be pointed out that the old system rendered useful data on the Peruća Dam behavior through years. These data were valuable for evaluation of the dam condition and scale of damage after the explosion.

The new system concept involved maximum use of automatics and integration of instrumentation and equipment into a centrally controlled system.

MONITORING TYPES

General

The basic criteria for determination of the monitoring scope and type and selection of the measurement method were the structural characteristics of the dam and its foundation requirements.

The care was also taken about the regulations governing the large dam monitoring. The world experience was applied, although the Peruća Dam damage and remediation are unique.

All the key dam elements important for its stability were included. Particular attention has been paid to the central dam section where the cut-off wall was constructed as a new impermeable element taking over the function of heavily damaged clay core.

Due to the specific characteristics of the dam which had been heavily damaged and then remediated, the planned monitoring scope is somewhat larger than for the similar dams.

The below monitoring activities are planned:

- visual observation,
- geodetic survey of horizontal and vertical displacement in the dam surface,
- concrete wall tilting and twisting measurements,

- clay core pore-water pressure gaging,
- total clay core pressures gaging,
- measuring horizontal displacements of points in the dam body,
- measuring vertical displacements of points in the dam body (settlement),
- measuring piezometric levels in the downstream dam toe,
- seepage measurements,
- seismological monitoring,
- reservoir water level measurements,
- grouting gallery lining deformation (convergence) measurement,
- dam crest extension and contracting measurement,
- hydrometeorological measurements.

The instrumentation adequate for the above monitoring activities was selected. The basic problem with selection of the instrumentation was the fact that the dam ends and 7-12 m of crest were to be reconstructed, and the central section repaired. Thus, installation of particular instruments could not follow standard procedures, and the new ones had to be created. This primarily regards the clay core pore-water pressure gaging, clay core total pressure gaging, and vertical displacements (settlement) measurements. The pore-water and total pressure gages in the rebuilt left end and under the dam crest were installed routinely in the open trench pits, Figs. 3 to 8. Since the central dam section was only repaired, the total pressure gages could have been installed in the dam body only from the grouting gallery, namely in and near the grouting gallery lining. The Architect/Engineer was aware that the measurement data obtained with the instrument installed in this way shall be of doubtful reliability. The installation of the pore-water pressure gage in the central section was solved in much easier way, so that the instruments were placed upstream the cut-off wall in the closed boreholes and downstream the cut-off wall in specially constructed piezometric tubes, Fig. 6. Still, the most difficult problem was measurement of the newly built central dam section settlement. The magnetic rings with blades were placed in the borehole, Fig. 8.

Visual Observations

The visual observations, as the part of the monitoring activities, are started immediately after the construction is completed, during the reservoir impoundment, and continued throughout the dam lifetime.

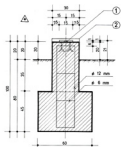
The visual observations are conducted in order to identify any events which might adversely affect the dam stability, such as deformations, cracks, water seepage through the dam body, springing of water downstream the dam, etc.

The quality and success of the visual observations depend on capability to notice a change, on qualifications of the observer, and observation frequency.

Geodetic Survey of Horizontal and Vertical Displacements

Horizontal and vertical displacements in the dam crest, downstream slope, and retaining walls are determined using the survey control points. The geodetic survey is conducted by geodetic methods and special instrumentation of adequate accuracy. The control point horizontal displacements are measured by intersections from the micro triangulation network monument, and vertical displacements are measured by indirect levelling.

The geodetic control points are arranged on the dam crest and downstream slope in 12 monitoring profiles, in the dam toe retaining wall, the retaining gate house wall and spillway walls, Fig. 1 and 3.



KEY

1. SUPPORT PLATE WITH SCREW FOR WILD T₂
2. PROTECTIVE COVER

Figure 3. Control point in the dam

Concrete Wall Tilting and Twisting Measurements

The measurements of tilting and twisting around the horizontal axis shall be conducted in the dam toe retaining wall, dam toe, retaining wall around the auxiliary gate house, and spillway walls.

The tilting and twisting measurements in the retaining wall in the dam toe shall be conducted with the installed automatic surface clinometers, Fig. 4. The clinometers are installed in each wall excavation unit. The automatic surface clinometers are checked with a portable PIZZI type clinometer, so special clinometer posts are placed on the site.

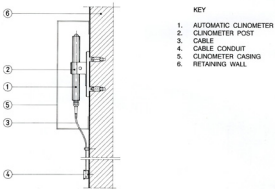


Figure 4. Automatic clinometer in the dam toe retaining wall



Photos 1 and 2
Automatic clinometer
installation in the dam toe
retaining wall



Clay Core Pore-Water Pressure Gaging

The clay core pore-water pressure measurements give an insight into the clay core impermeability. The gaging is conducted upstream and downstream the cut-off wall, in the left dam end and the upper reconstructed part of the central dam section, Fig. 5.

Upstream the cut-off wall, the pore-water pressure cell is placed in a closed borehole, Fig. 6a. Downstream the cut-off wall, the pore-water pressure cell is placed in piezometer tubes, so it may be taken out and checked. The gaging is planned in 10 m distanced levels in each monitoring profile, Fig. 6b.

In the left end, where the dam is completely rebuilt, and in the upper central section of the dam, the pore-water pressure cells are placed in an open trench along with the total pressure gages, Photos 3, 4 and 5.

Clay Core Total Pressure Gaging

These measurements determine the total pressure on various horizons in the clay core. The total pressure cells are placed in an open trench, in the parts of the dam which are completely rebuilt, left end and the upper central section, Photos 6, 7 and 8.

To get an insight into the total pressures development in the central dam section which is not newly built, the cells are also placed by and into the gallery lining, since that was the only accessible way.

Dam Body Horizontal Displacement Measurements

The horizontal displacements in the dam body are measured with clinometer.

The clinometer tubes are placed in boreholes drilled after the dam remediation is completed. The horizontal displacements shall be continuously monitored in the monitoring profiles 3-7, using permanently installed clinometers. The clinometers are arranged within the clinometer tube so that three clinometers from the top are arranged in the first 30 m, 10 m apart, and one is placed on the tube bottom.

The dam body horizontal displacement measurements in the monitoring profiles 1, 2 and 8 is conducted using the portable clinometer, Fig. 8.

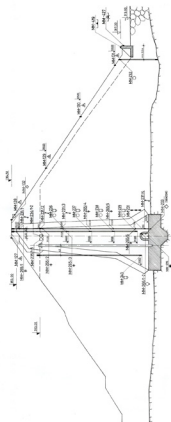
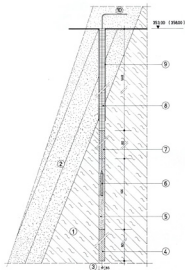


Figure 5. Arrangement of monitoring points - monitoring profile 5

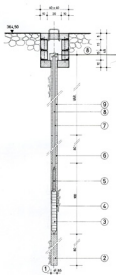
a) Upstream the cut-off wall



KEY

1. CLAY CORE
2. FILTER LAYERS
3. BOREHOLE
4. BENTONITE PLAG
5. SAND FILTER
6. PORE-WATER PRESSURE CELL
7. BENTONITE PLAG
8. GROUTING MIX
9. BOREHOLE CASING (THROUGH FILTER LAYERS)
10. CABLE

b) Downstream the cut-off wall



KEY

1. BOREHOLE
2. BENTONITE PLAG
3. POROUS FILTER
4. SAND FILTER
5. PORE-WATER PRESSURE CELL
6. BENTONITE PLAG
7. PIEZOMETER TUBE
8. CABLE
9. GROUTING MIX

Figure 6. Clay core pore-water pressure gages placed in a borehole

Dam Body Vertical Displacements (Settlement) Measurement

In addition to the clinometer tube, the magnetic rings with blades are also placed 10 m apart, heightwise. The displacements are occasionally measured with special instruments. This type of instrumentation was the only option for installation in the remediated central dam section, Fig. 8, where the possible settlement monitoring is particularly important.

Piezometric Level Measurements in the Downstream Dam Toe

The piezometric level of water shall be measured in the downstream dam toe. The open piezometers are set for that purpose. The piezometers are set in the monitoring profiles 1-8, and the piezometric level is measured using the portable meters.



Photo 3. Placement of pore-water pressure cells and total pressure cells in the left dam end



Photo 4. Pore-water pressure cell preparation before placement



Photo 5. Pore-water pressure cell placement into the dam clay core

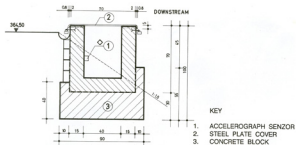


Figure 7. Accelerograph sensors placement on the dam crest



Photo 6. Total pressure cell



Photo 7. Pits dug in a clay core for placement of cells in different directions



Photo 8. Total pressure cell placement



KEY

1. BOREHOLE
2. GROUTING MIX
3. MAGNETIC RING WITH BLADES (10 m INTERVAL)
4. CLINOMETER TUBE
5. BIAxIAL CLINOMETER
6. CABLE

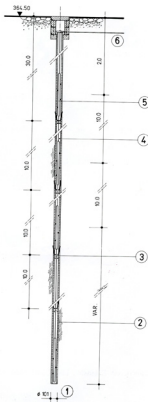


Figure 8. Placement of clinometer, settlement monitoring magnetic ring

Seismological Monitoring

Since the Perúa Dam is a large dam, the seismological monitoring is requested.

The seismological monitoring is generally divided into the induced seismicity monitoring and the dam dynamic behavior monitoring during strong earthquakes.

Only the instrumentation for monitoring the dynamic behavior has been planned for the Perúa Dam since, with respect with the reservoir age, any induced seismicity is expected to be very low and shall not significantly affect the structure.

The measurements are aimed at obtaining the oscillation time histories which are than used for determination of the structure dynamic behavior parameters.

Five measurement points have been planned for this task, two in the grouting gallery, two in the dam crest, Fig. 7 and one in the bedrock approximately 300 m downstream the dam.

Only sensors are set in the gallery, dam crest and bedrock, while the accelerographs are housed in a special room in the gate house, in the left dam end, and cable connected with the sensors.

The number of accelerographs and arrangement of the measurement points has been determined according to the Large Dams Monitoring Code and the recommendations of ICOLD.

Dam Gallery Seepage Measurements

The dam gallery seepage measurements are conducted from the grouting gallery, at the entrance into the drainage pipe, where a Thompson spillway with the automatic level recorders is constructed for this purpose.

Dam Crest Extension and Contracting Measurement

These measurements are planned to be conducted in two points at the El. 358.0 m a.s.l.

Ten extensometers with 6 m base each are installed for this purpose. The extensometers are grouped in two 30 m extensometer chains with five extensometers each, Photo 9.



Photo 9. Installation of extensometers into the dam crest

Reservoir Water Level Measurements

The reservoir water level is measured in two ways:

- using the staff gages,
- using the water level recorders.

The staff gages had been set by the headrace tunnel gate aisle, and they were used for the reservoir level gaging prior to the remediation.

The reservoir level recorder is set by the headrace tunnel intake, and is cable connected with the gate house Monitoring Center.

Grouting Gallery Lining Deformation Convergence Measurement

The convergence shall be measured in specified measurement profiles 1-12, and the lining convergence shall be measured vertically, horizontally and diagonally, along with the absolute vault displacement.

The convergence shall be measured using the vibrating string extensometer. The benchmarks have been placed in the concrete lining for this purpose, Photo 10.

The absolute vault displacement is measured by the geodetic surveying methods using the standard surveying instruments.



Photo 10. Grouting gallery convergence measurement

MONITORING INSTRUMENTATION AND POINTS NETWORK ORGANIZATION

The monitoring network includes permanently installed instruments for continuous metering, and measurement points and equipment for periodic measurements.

Each instrument or point is a measurement point designated in the drawings with MM, followed by an adequate number and a symbol of an instrument or point type.

The measurement sites in the dam are grouped in 12 monitoring profiles typical for various reasons, and they are used for different types of measurements, Fig. 1 and 2. This facilitated determination of interrelations of various events at the dam in each profile.

Another set of measurement sites is organized in the dam toe retaining wall, in the retaining wall around the auxiliary gate house, in the dam by the spillway structure, in the spillway structure, left access platform, weather station and by the grout curtain.

All measurement instruments are connected to the Monitoring Center from which the measurements are controlled and the measurement data collected.

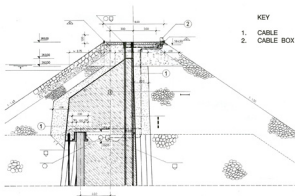
To avoid congesting number of cables leading to the Center, the cable boxes are installed in all profiles, the dam toe retaining wall, and the gallery. The cables are routed vertically from the dam body to the crest. The cables are grouped, protected by the conduit, and routed through the downstream dam filter to the crest, Fig. 9. Special attention has been paid to protection of cables against damage, Photo 11.

The cables from the measurement instrumentation in the monitoring profiles, namely those placed in the retaining wall and gallery are led to the cable boxes. The total of 10 cable boxes is installed. The cable boxes are connected by a special common cable connecting them with the Monitoring Center, Fig. 10.

The measurements in other measuring sites which are not connected to the Monitoring Center are conducted with the surveying instruments and other portable equipment, and they are useful additional means of control of the above measurements. This includes measurement of horizontal displacements in the monitoring profiles 1, 2 and 8, where the in-place clinometers are not installed, for settlement measurements, piezometric water level gaging, inclination measurements with the portable mechanical clinometer, the grouting gallery lining convergence and all geodetic surveys. All the instruments are of SISGEO make.

LIST OF THE PERUĆA DAM MONITORING INSTRUMENTATION

INSTRUMENT TYPE	DESIGNATION	INSTALLED	
		PCS	m
CONTROL POINT FOR MEASUREMENT OF HORIZONTAL AND VERTICAL DISPLACEMENTS		68	
CLINOMETER TUBE			367
IN-PLACE CLINOMETER		20	
MAGNETIC RING WITH BLADES FOR SETTLEMENT MEASUREMENTS		38	
PIEZOMETER		8	102
PORE-WATER PRESSURE CELL		40	
TOTAL PRESSURE CELL		41	
GROUTING GALLERY LINING CONVERGENCE MEASUREMENT BENCHMARKS		48	
DAM CREST EXTENSOMETERS		10	60
AUTOMATIC CLINOMETER		8	
CLINOMETER POSTS		16	
WATER LEVEL GAGE		1	
ACCELEROGRAPHS		5	
WEATHER STATION		1	



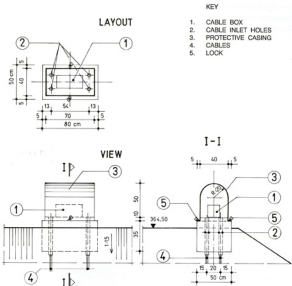


Figure 10. Cable box

MONITORING CENTER

The Monitoring Center is located in a separate room within the gate house in the left dam end. It houses the computer-controlled Data Acquisition System which uses specialized software. The software enables setting of measurement intervals and running of necessary analyses.

The Data Acquisition System is the brain of the monitoring system, and it includes two MICRO-10 Dataloggers connected with four 16/32 Canal Multiplexer units and the power supply unit, Photo 12.

The MICRO-10 Datalogger, Campbell Scientific, consists of a micro computer, clock, multimeter, calibrator, scanner, and the frequency counter and controller. Each MICRO-10 Datalogger can simultaneously scan 12 canals with equal sensors or 6 canals with different sensors, which means that simultaneous scanning of 24 or 12 canals is possible. The scanning interval for each canal is programmable.

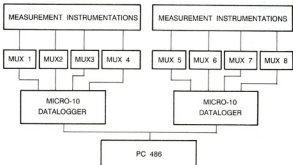
Two 16/32 canal Multiplexer model 8032 units are connected to each MICRO-10 Datalogger to which all the measurement instruments are connected with cables.

The number of canals used by a particular instrument depends on a type of the electric signal, and the complete Peruća Dam monitoring system uses 154 canals.

The Center also houses all five accelerographs, Photo 13, cable-connected with the sensors in the gallery, dam crest and downstream in the bedrock. The seismological measurements system is independent of the system controlling other measurements. The accelerographs are of SIG make, SMACH-2 - STRONG MOTION type, and they include the digital recorder with memory card and a separate sensor (triaxial accelerometer). The accelerograph triggering is setup to 0.1 g.



Photo 12. Data Acquisition System



REMARK

- MUX 16/32 CHANNEL MULTIPLEXER
- THIS SCHEME DOES NOT INCLUDE THE SEIZMOLOGICAL MONITORING WICH IS INDEPENDENT

PERUĆA DAM MONITORING SYSTEM - SCHEMATIC DIAGRAM

MONITORING SCHEDULE

The design documentation specifies the monitoring and measurement intervals.

It has been planned that the monitoring and measurements should be more frequent during the first five-year period after the remediation. Would the dam behavior prove to be within the expected framework, the number of activities can be reduced.

Monitoring and measurements intervals during the first 5-year period after the dam trial loading

- visual observation	once a day
- geodetic survey of horizontal and vertical displacements	4 times a year
- concrete wall tilting and twisting measurements, automatic clinometer	continuous
- concrete wall tilting and twisting measurements, portable clinometer	4 times a year
- clay core pore-water pressure gaging	continuous
- clay core total pressures gaging	continuous
- measuring the dam body horizontal displacements, monitoring profiles 3-7	continuous
- measuring the dam body horizontal displacements, monitoring profiles 1, 2, 8	4 times a year
- measuring dam body vertical displacements, settlement	4 times a year
- measuring piezometric levels in the downstream dam toe	once a week
- gallery seepage measurements	once a week
- seismological monitoring	continuous
- reservoir water level gaging	continuous
- grouting gallery lining deformation, convergence and absolute vault displacement measurement	4 times a year
- dam crest extension and contracting measurement	continuous
- hydrometeorological measurements	continuous

The continuous measurements are automatic, computer controlled from the Data Acquisition System. The frequency and intervals of the automatic measurements shall be determined by the Architect/Engineer.

Monitoring and measurements during the dam operation

If the first 5-year period of the remediated dam operation prove that the monitoring schedule need not remain the same, the below intervals shall apply:

- visual observation	once a day
- geodetic survey of horizontal and vertical displacements	twice a year
- concrete wall tilting and twisting measurements, automatic clinometer	continuous
- concrete wall tilting and twisting measurements, portable clinometer	twice a year
- clay core pore-water pressure gaging	continuous
- clay core total pressures gaging	continuous

- measuring the dam body horizontal displacements,
monitoring profiles 3-7 continuous
- measuring the dam body horizontal displacements,
monitoring profiles 1, 2, 8 twice a year
- measuring dam body vertical displacements, settlement twice a year
- measuring piezometric levels in the downstream dam toe once a week
- gallery seepage measurements once a week
- seismological monitoring continuous
- reservoir water level gaging continuous
- grouting gallery lining deformation, convergence, and absolute
vault displacement measurement twice a year
- dam crest extension and contracting measurement continuous
- hydrometeorological measurements continuous



Photo 13. SMACH 2-SIG accelerograph

CONCLUSION

After remediation, the Peruća Dam shall be put into operation and therefore exposed to all the extreme design loads.

The remediated Peruća Dam system is conceived so as to encompass all the key structural elements and enable the dam behavior monitoring during its operation. Regular dam monitoring is the only way to control its stability and safety in operation, and check the soundness of the design solutions applied in its remediation. On the other hand, it enables timely response to any emergency.

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- material deposit investigations
- control of material placement in dam body

GeoKon - Zagreb



Miroslav IVOS, Geotechnical Engineer

Supervisor

- control of material placement in dam body
- cut-off wall plastic concrete control

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Professor Dr.Sc. Velimir UKRAINCZYK

- cut-off wall plastic concrete control testing

CIVIL ENGINEERING COLLEGE,
UNIVERSITY OF ZAGREB



Nedjeljko AKRAP, B.Sc.C.E.

- permanent concrete testing
- permanent cut-off wall plastic concrete testing

Civil Engineering Institute of Croatia - Split Division



Ivo BARBALIĆ, B.Sc.C.E.

- permanent testing of material placed in dam body

Civil Engineering Institute of Croatia - Split Division



Vjekoslav JURKO, Building Material Technician

- permanent testing of material placed in dam body

Civil Engineering Institute of Croatia - Split Division



Ivo OZRETIĆ, B.Sc.C.E.

Site Engineer

- grouting gallery
- dam toe and bottom outlet auxiliary gate
- house retaining walls
- dam body removal and reconstruction
- spillway

Konstruktor - Inženjering Ltd. - Split



Božo BORZIĆ, Civil Technician

Site Engineer

- grouting gallery
- dam toe and bottom outlet auxiliary gate
- house retaining walls
- spillway

Konstruktor - Inženjering Ltd. - Split



Milan RADELJAK, B.Sc.C.E.

Site Engineer

- dam toe retaining walls
- dam body removal and reconstruction

Konstruktor - Inženjering Ltd. - Split



Marko ZORIĆ, B.Sc.C.E.

Site Engineer

- The Cetina riverbed training
- dam downstream slope development

Cetina Public Water Company - Sinj



Ante BARAĆ, Geotechnical Engineer

Site Engineer

- investigations
- grouting

Geotecnika Ltd. - Zagreb



Zvonimir ŠEPAC, B.Sc.C.E.

Site Engineer

- plastic concrete cut-off wall

Geotecnika Ltd. - Zagreb



Rudolf HENNING, Engineer

Site Engineer

- plastic concrete cut-off wall

Bauer - Schrobenshausen



Branko BOSANČIĆ, B.Sc.C.E.

Site Engineer

- plastic concrete cut-off wall

Geotecnika Ltd. - Zagreb

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Part of the Konstruktor - Inženjering crew engaged on the grouting gallery reconstruction





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Franciscan monastery in Split



Split beltway overpass

WORKS ABROAD

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Spinat residential area, Split



Maslenski most bridge -
mock-up



Učka tunnel



Dale HPP on the Cetina



Kafae Gorge HPP, Zambia
(underground power house)



International Croatian
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