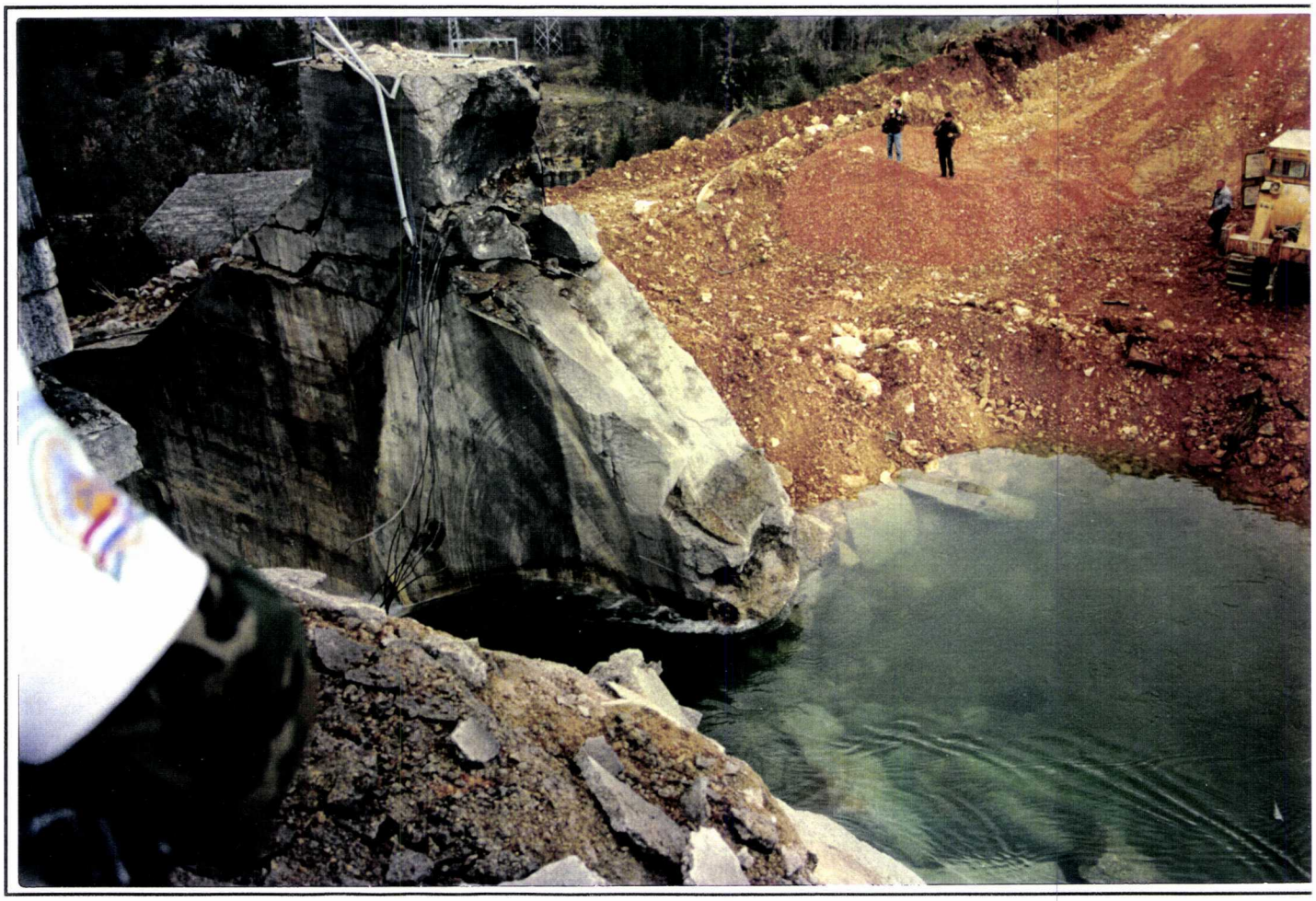
REPORT NO 01-93

NVE
NORWEGIAN
WATER RESOURCES AND
ENERGY ADMINISTRATION

REPORT FROM INSPECTION AND MEETINGS REGARDING PERUCA DAM IN CROATIA

By

E. Torblaa, R. Dahl, B. Kjærnsli



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TITTEL/TITLE REPORT FROM INSPECTION AND MEETINGS REGARDING PERUCA DAM IN CROATIA	RAPPORT NR/REPORT NO 01-93
SAKSBEHANDLER(E)/EXECUTIVE OFFICER(S) Eivind Torblaa Rune Dahl Safety and emergency planning department	DATO/DATE March 1993 Revised April 1993
OPPDRAGSGIVER/COMMISSIONING AGENT ROYAL NORWEGIAN MINISTRY OF FOREIGN AFFAIRS	OPPLAG/NUMBER PRINTED 45

SAMMENDRAG/ABSTRACT

The Peruca dam, a 450 meter long and 65 meter high embankmentment dam was extensively damaged by strong explosions the 28 January 1993. The dam is situated close to the border between Croatia and Bosnia-Hercegovina in the Cetina river in the Dalmatia region.

The dam was completed in 1958 and the owner is Hrvatska Elektroprivreda (HEP) or the Croatian Electric Power Authority.

This report contains mainly the material which was prepared for HEP by the Norwegian delegation who visited the dam two weeks after the damage. The intention of all the appendices were to answer question which were raised during their stay in Croatia.

Additionaly, as an introduction to this collection of reports and papers related to the matter, an article from Water Power (April 93) is included in Appendix 1. This was written after the Norwegian delegation visited Croatia. It is a well arranged and very detailed description of the situation at the dam so far.

EMNEORD/SUBJECT TERMS

Sabotage
Embankment dam

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INTRODUCTION

This report is prepared for Hrvatska Elektroprivreda in accordance with enclosed Terms of Reference (Appendix 1).

The trip to Croatia was initiated by the Royal Norwegian Ministry of Foreign Affairs, and hence carried out by Mr B. Kjærnsli from The Norwegian Geotechnical Institute (NGI) and Mr R. Dahl and Mr E. Torblaa from the Norwegian Water and Energy Administration (NVE).

In the morning of 9th Febr. 1993 we were introduced to Mr B. Jaksic who is the advisor of the President on matters concerning the Peruca Dam at the Presidential Palace in Zagreb. The following report describes our involvement with the problems related to the dam during the delegations stay in Croatia.

The undersigned wish to stress the high degree of friendliness, hospitality and professional cooperation under which the work was carried out during week 6, 1993.

Bjørn Kjærnsli Rune Dahl Eivind J. Torblaa

Executive summary.

The Peruca dam, a 450 meter long and 65 meter high embankment dam was extensively damaged by strong explosions the 28 January 1993. The dam is situated close to the border between Croatia and Bosnia-Hercegovina in the Cetina river in the Dalmatia region.

The dam is a rockfill dam with a central core of clay. The ground is built up of crushed limestone with irregular transitions into weathered bauxite. The dam encloses a reservoir of 540 mill. m^3.

and the Norwegian Geotechnical Institute(NGI) travelled to Croatia after initiative had been taken the Royal Norwegian Ministry of Foreign Affairs to support Croatia in this emergency situation.

The main objective of the assignment was to give advice on:

1. The urgent short term measure to prevent flooding and thereby catastrophic consequences for people and properties.

2. Permanent long term measures to bring the dam back in its function.

Engineers from the former Yugoslavia have considerable experience in the field of dam engineering. Hence, it was assumed that the visit by the Norwegian delegation also represented an important moral support.

During the first day in Croatia the delegation was informed about the sabotage and the immediate actions which had been taken by engineers in Zagreb from Hrvatska Elektroprivreda(HEP) and Elektroprojekt. Hrvatska Elektroprivreda is the dam owner and Elektroprojekt their consultant on the matter. The main events were as follows:

- Explosives equivalent to between 20 and 30 tons of TNT were set off during the morning 28 Jan.93 by Serbian forces.

- The explosives were placed at five locations in a gallery in the bottom of core of the dam.

- The immediate consequences of the explosions were severe, but the dam did not fail.

- The immediate measures taken by the dam owner consisted basically of the following: 1) to fill the craters caused by the explosives 2) to evacuate 3000 people who would be at risk if the dam had failed 3) to lower the reservoir water level by opening the bottom outlet gate 4) to start monitoring the behavior of the dam.

During the third day in Croatia, the delegation visited the dam site. Due to the danger of

mines, the embankment could only be inspected at limited areas. At this day, 13 days after the sabotage, there seemed to be no danger of collapse. Since the explosions took place at Peruca the reservoir water level had been drawn down by about 0.8 meters every day by discharging 220 m^3/s through the bottom outlet.
At the 10 Feb. the water level was about 16 meters below normal water level.

The explosives made two big craters at each side of the embankment and a depression at the center of the dam. The two craters were filled up with suitable material immediately after the Croats entered the dam. The depression at the center of the dam had continued to settle with a decreasing speed since the explosions.

The water outflow which followed the explosions discharged into the switchyard just downstream the dam at a rate of 540 l/s. The water was brownish-red transporting large fragments of the clay core up to 50 cm in diameter. More than 2000 m^3 of fill material was flushed out during the first 9 days after the sabotage.

Luckily, the spillway gate was in an open position at the time of the explosion and the water level was about 4.2 meters below normal water level. If this had not been the case, the dam would very likely have been overtopped with a dam breach as a result.

In the meeting after the site visit the immediate actions taken by HEP and possible long term measures were discussed.

The Croatian team of experts and the workforce at the site undoubtedly acted very quickly to prevent a much larger scale disaster, that is, complete failure of the dam. Their work was well coordinated, and temporary remedial measures saved the main body of the dam, although it sustained severe damage.

Some possible long term measures were considered. Before any firm conclusions can be drawn, further investigations has to be carried out. Nevertheless, the general idea was that a total reconstruction of the dam is probably not necessary. If parts of the reservoir can be maintained throughout a rehabilitation period, the negative impacts will be drastically reduced.

The HEP engineers were very interested in information about a solution with a central asphaltic-concrete core. This is a method of construction which we have good experiences with in Norway, and it can be a good alternative for the rehabilitation of the dam.

Another field of interest was Norwegian dam safety philosophy. In Norway, we have about 170 larger embankment dams. A general rule is that these dams shall not require any operation in emergency situations. They are often situated in remote areas far away from people. HEP was interested in ideas on the matter.

Minutes from meeting in Zagreb.

Date: 9th. Febr. 1993.

Participants:	M. Pazic	Hrvatska Elektroprivreda(HEP)
	Z. Sever	Elektroprojekt
	J. Rupcic	Elektroprojekt
	F. Arthur	Norwegian Charge d'Affaires, Zagreb
	B. Kjærnsli	NGI
	R. Dahl	NVE
	E. Torblaa	NVE

The meeting was arranged in order to introduce the Norwegian delegation to the situation concerning the Peruca dam. See appendix 2 for a short brief of the dam design.

Mr Z. Sever from Elektroprojekt went through the main events based on his report dated 3rd. Febr. 93. Further remedial measures were discussed briefly.

The main content of the report is the following:

- Between 20 and 30 ton of explosives were set off during the morning of 28th. Jan. 93 by Chetnik (Serbian) forces.

- The explosives were placed at five locations (see appendix 4).

- The Chetnik (Serbian) forces controled the area after driving the UN-forces (UNPROFOR) soldiers away.

- The Croatian army forced the Chetniks forces away immediately after. The HEP engineers then started their observations and urgent measures.

- The immediate consequenses of the explosions were severe but the dam did not fail.

- The immediate measures consisted basically of the following 1) to fill the craters caused by the explosives 2) to lower the reservoir water level by opening the bottom outlet gate 3) to start monitoring the behaviour of the dam.

- The report concludes that, luckily, the resevoir water level was about 4.2 meters below normal water level. If this had not been the case the dam would most likely have failed.

- The engineers from Elektroprojekt feel that the situation is under control at the moment.

P.S. See pictures in appendix 3 for illustration of the situation one day after the brake.

Minutes from short meeting with HEP in Sinj.

Date: 10th. Febr. 1993

Participants:	I. Covic	HEP
	R. Miskov	HEP
	M. Vilovic	HEP
	J. Macan	HEP
	B. Kjærnsli	NGI
	R. Dahl	NVE
	E. Torblaa	NVE

The meeting was arranged in order to prepare the Norwegian delegation for a visit to the dam site and to give the view of HEP's Sinj office which is responsible for the operation of the Peruca dam and power station.

Different aspects were discussed but the main point are listed below:

- The explosives were placed in the dam during the period Sept. 91 to July 92 when Serbian forces controled the dam. The explosives were not removed during the period up till 27th. Jan. 93 when UN-forces controled the dam. The 27th. Jan. 93 the dam was reoccupied by Croatian forces.

- The HEP engineers were not permitted to enter the dam during the period when UNPROFOR controled the area. After several requests from HEP the flood gate was eventually opened in order to reduce the reservoir water level. Luckily this gate was left in open position at the time of the explosions.

- The information given by the engineers from Elektroprojekt was confirmed, exept for some small adjustments of the damage caused by the explosives and the quantity of explosives.

- Geophysical seismic shows that the bauxite in the foundation is degraded 30 meters below the position where the explosives were placed at the left hand side of the dam.

- The HEP engineers are anxious to find out to what extent the clay core and the grout curtain is affected by the action, but at the present they are limited to certain investigory operations due to the war situation. Further, they want to investigate the whole area for mines.

- When the Croatian army and the HEP personnel entered the dam 28th. Jan., they could observe a large amount of water, carrying silt and clay, coming up in the switch yard area just below the deepest section of the dam in the downstream area (See appendix 3).

- One day later this stopped, and the seepage moved to the access gallery entrance at the right hand side. To begin with this seepage was roughly estimated to be 600 l/s.

- The bottom outlet was opened and the total discharge was about 220 m3/s and the water level in the reservoir decreased by 0.8 meters/day.

- The settlements at the deepest section of the dam were about 0.15 meter/day the first days after the initial settlement of about 1.8 meters.

- Great efforts were taken to carry out the sufficient monitoring.

Site visit, Peruca dam.

Date: 10th. Febr. 1993

Participants:	I. Covic	HEP
	R. Miskov	HEP
	M. Vilovic	HEP
	J. Macan	HEP
	B. Kjærnsli	NGI
	R. Dahl	NVE
	E. Torblaa	NVE

EMBANKMENT

The embankment could only be inspected from a limited area due to the danger of mines. The area had not yet been demined. At the time of the inspection, the leakage through the left access gallery had decreased to below 50 l/s and the water level is at about 345 m.a.s.l. The leakage through the gallery is reduced from more than 600 l/s 29th. Jan. when the water level was at about 356.2 m.a.s.l. The leakage water is now clear.

The settlements at the deepest section were monitored when we were there. This part suffered severe settlements of about 1.8 meters caused by the explosions. This settlement has now decreased to 0.03 meters/day. The total settlement since the explosions is about 2.5 meters.

SPILLWAY

The spillway is badly damaged by the explosives.See pictures in appendix 3.

BOTTOM OUTLET

The discharge through the bottom outlet is about 220m3/s. The reservoir water level decreases by about 0.8 m/day.

The site visit gave the impression that the dam was very well maintained in the time before the war. The steelwork and the concrete did not show particular signs of deterioration.

Minutes from meeting with HEP in Sinj.

Date: 11th. Febr. 1993.

Participants:	R. Miskov	HEP
	M. Vilovic	HEP
	J. Macan	HEP
	J. Raos	HEP
	M. Zec	HEP
	Z. Zanchi	interpreter, Croatian Government
	B. Kjærnsli	NGI
	R. Dahl	NVE
	E. Torblaa	NVE

The meeting was arranged in order to discuss the situation and for the Norwegian delegation to give their opinion on the matter. The main points are written in short below:

- The immediate emergency situation is under control. The HEP personnel has done an excellent job in preventing the damage of the dam to lead to a catastrophic failure.

- The monitoring programmes carried out by HEP is after B. Kjærnsli's opinion satisfactory at the present stage. Further, HEP's plans for additional monitoring when the situation allows it, are adequate.

- Due to the fact that water is a very precious resource in the area, it is desired to keep as much water as possible in the reservoir. Mr. B. Kjærnsli agrees that the suggested maximum reservoir water level at 340 m.a.s.l. sounds adequate to keep the situation under control. Although, it must be kept in mind that the safety of the dam is dependant on the behaviour of the gates in the bottom outlet tunnel. The HEP engineers are perfectly aware of this.

- In order to keep the water level at about 340 m a.s.l. it might be necessary to regulate the bottom outlet gate. This gate is not designed for this purpose and vibrations induced by the water can represent a problem. NVE will try to provide HEP with information about the matter. See appendix 7.

- The hydrology should be checked and the increase in reservoir water level must be calculated for different flood situations.

Concerning the long term measures the following was discussed:

- The value of the stored water must be established.

- Different reconstruction alternatives can then be evaluated. Intuitively, Mr. B. Kjærnsli thinks a reconstruction of the central part of the dam while keeping as much as possible of the existing upstream and downstream fill undisturbed is most economical. The upstream fill can be used as a coffer dam during construction period. The assumption is basically based on the value of water and the additional cost of a new grout curtain if an upstream sealing is considered.

- Different types of core material must be considered. An asphaltic core was briefly discussed. NGI/NVE will provide HEP with information. See appendix 6.

- The existing spillway was briefly discussed and Mr. E. Torblaa described the Norwegian philosophy on this matter. NVE will provide HEP with information about open, ungated spillways. See appendix 8.

- The throughflow and stability were briefly discussed. See appendix 9.

APPENDIX 1

Article from Water Power.

Investigation work begins at Peruća dam, Croatia

The condition of the 65 m-high Peruća dam on the Cetina river in Croatia is now considered to be stabilizing, following the severe damage it sustained on 28 January when five explosive devices were detonated (see *WP&DC*, January and March 1993).

Temporary remedial measures implemented at the site by Croatian engineers on 29/30 January, and in the subsequent few days, while the reservoir was being drawn down at a rate of about 85-90 cm/day through the bottom outlet, were successful in preventing a complete breach of the dam. This could have threatened the homes of the population of 20 000 downstream, who had been prepared for evacuation. These temporary measures (described in more detail on p16) included the rapid backfilling of craters formed by the explosions, and of sinkholes which began to develop on 30 January as a result of internal erosion, as large quantities of core and fill materials were washed away.

The remedial measures taken so far, as well as preliminary investigations on the force of the explosions and condition of the dam body, have had to be conducted only from the crest of the dam and limited areas at the abutments and around the powerplant, as there are still fears that other parts of the dam could be mined.

As we went to press (8 March) the reservoir level had been drawn down to el. 320 m, and further major structural deterioration to the dam was thought to be unlikely, although crest settlement was continuing and deformations were apparent on the upstream face. Both transverse and several longitudinal cracks could be seen on the 8 m-wide, 450 m-long dam crest.

It has now been established that the first explosion occurred at 1048 h and 55 s on 28 January; this was registered at three seismic monitoring stations (at Trilj, 21.1 km downstream from the dam, on the island of Hvar 67.9 km away, at a place called Puntijarka near Zagreb, 242.3 km away).

Fig. 1 shows the positions of the five explosive devices, and a sketch of the craters which were formed as a result.

Background

As reported in our January and March issues, the Peruća dam and hydro plant had been taken by Serbian forces on 17 September 1991, and at that time the management and operating personnel of the Croatian Electric Power Authority were expelled from the plant. J. Macan, Manager of the power station, told *Water Power & Dam Construction* that at 1000 h on that day, the two generating units had been closed down by the departing Croatian engineers, and the flap gate on the spillway and the bottom outlet had been opened. Five days later, the Yugoslav Federal Army had closed the gate and outlet, and no water was released until one incident when the flap gate was opened in early 1992 (flooding an area downstream).

On 25 December 1991, during a meeting with an EC monitoring mission, Macan had been able to ascertain that the reservoir level was at el. 360.8 (0.8 m above the maximum operating level), and by January 1992 the level had reached el. 361.2. (The dam crest is at el. 363.) During this period Macan had formally requested water to be released through the spillway and bottom outlet. Meanwhile Croatian engineers calculated the water level continuously using long-range photography from a distance of 4 km.

During July 1992 it was ascertained from photographs taken in the various dam galleries by a UNPROFOR representative, that at least three boxes of explosives were inside the dam body, confirming the threats which had first been reported in October 1991.

27-30 January 1993

J. Macan and M. Vilović of the Croatian Electric Power Authority, Split, described the sequence of events at the end of January this year as follows.

On the afternoon of 27 January, there was artillery fire in the region of Peruća dam, and on the morning of 28 January, as usual there was a general alert. At 1048 h, strong explosions were heard at the dam site. The first damage to be seen was at the spillway. Croatian soldiers returning from the area at 1430 h reported more extensive damage, and in particular that muddy water was flowing from a shaft

From left: J. Macan (Manager of the Peruća powerplant), and J. Raos and M. Vilović (both of the Croatian Electric Power Authority, Split), on the crest of Peruća dam on 13 February. J. Macan and M. Vilović organized the temporary remedial measures on 30 January.

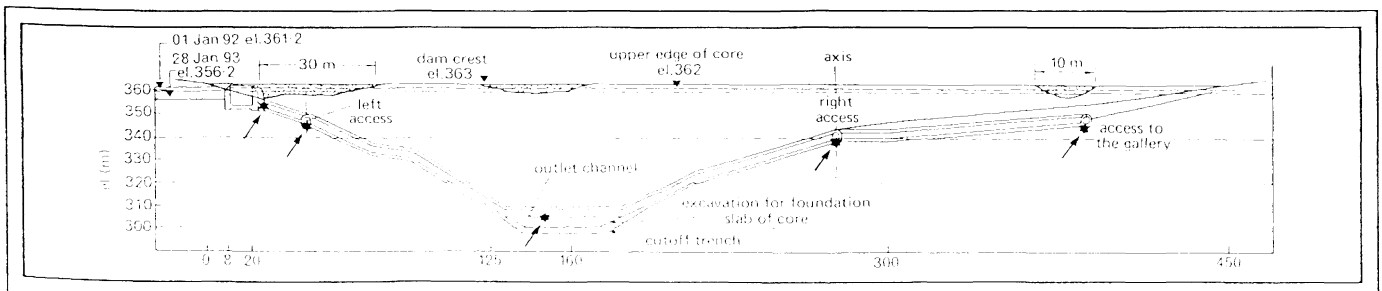


Fig. 1. Longitudinal section of Peruća dam, showing the locations of the five mines (indicated by stars) which exploded on 28 January. The shaded areas show the shapes of the three craters formed (not to scale), the largest crater being on the left side adjacent to the spillway headworks.

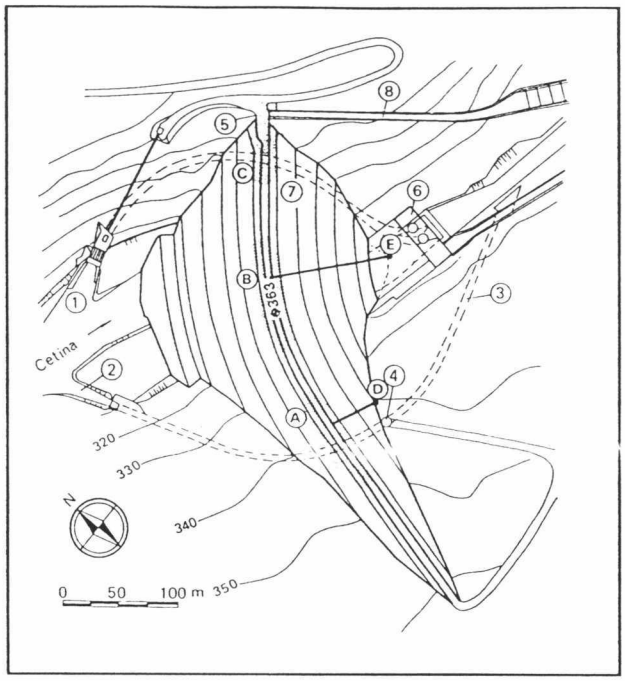


Fig. 2. Peruća dam, where: 1 = intake structure; 2 = intake for bottom outlet; 3 = bottom outlet tunnel; 4 = gatehouse for bottom outlet; 5 = spillway gates; 6 = powerplant; 7 = headrace tunnel; 8 = spillway; A = damaged crest at right bank side; B = damaged crest in centre of dam; C = damaged crest at left bank side; D = dark brown water emanating from the access gallery to the longitudinal gallery; E = culvert exit to drain, which caused a fountain of water and clay to spout into the switchyard (it has now stopped flowing).

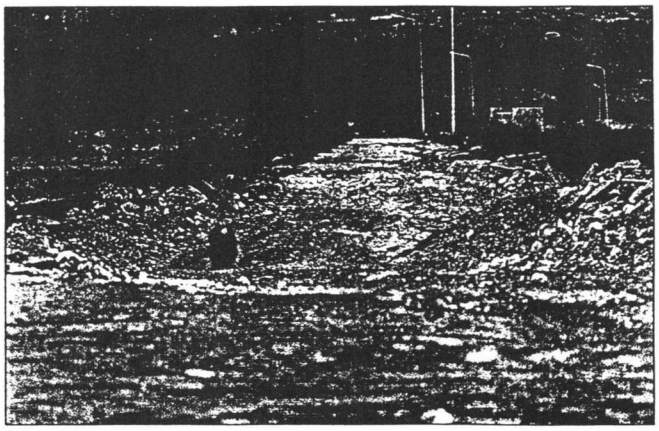
into the centre of the switchyard. In the early afternoon, personnel from the Croatian Electric Power Authority arrived at the site and reported heavy damage. First inspections by them began at 2000 h on 29 January, from the right abutment of the dam. Approximately 20 m from this abutment, a crater had been formed which was about 10 m in diameter and 6 m deep. Fissures extended 40 m along the crest of the dam, and there were also transverse cracks on the crest. There was severe subsidence in the middle of the crest, and movement could even be felt under foot.

On the left abutment the situation was the most dangerous. A section of the dam 30 m long and almost 5 m deep had been displaced in the direction of the downstream slope. It was extremely fortunate that the water level was at el. 356.2 at this time, so that reservoir water did not overtop this damaged section. However, in one place water was able to pass through the dam body into the damaged gallery, making the situation critical.

Many forms of action were organized during the night. It was decided not to be necessary to issue a general alarm to all of the downstream population of 20 000; during the night this could have caused unnecessary panic and people could have died as a result. It was decided to evacuate 3000 people who would have been at risk if the dam had failed, and this was organized quietly with the assistance of the Croatian Army and their representatives.

Meanwhile the water level was observed continuously both in the reservoir, and at the Han bridge on the Cetina river, 4 km downstream. Water sampling also began, so that initial assessments could be made of the quantity of material which had been washed out from the dam body.

All available construction plant and equipment in the area was mobilized (one Caterpillar D8 bulldozer, one loader and about 30 dump trucks); this equipment was available at the site by 0400 h on 30 January. It was not



Damage to the dam crest on 30 January.

The Peruća powerplant on 13 February; the two Francis units had an installed capacity of 41.6 MW.

immediately possible to begin remedial work, as there was artillery fire on the dam from midnight on 29/30 January, and Croatian engineers who began to inspect the dam by torchlight were fired at.

Backfilling work using the construction machinery began at 0800 h on 30 January. By 1110 h, outflow from the shaft above the 110 kV switchyard had stopped.

A total of 1000 m^3 of material was required for the backfilling of craters and areas of subsidence.

Draining of the reservoir (470×10^6 m^3 at that time) began through the bottom outlet. Labourers from the Croatian Electric Power Authority operated the bottom outlet without any problems, despite reports that the gate operation equipment had also been mined or blocked. Because of the need to draw down the reservoir rapidly, a contingency plan had been made to open the outlet with a controlled explosion in the event of it being blocked.

Apparent structural behaviour of the dam following the explosions

The original principal designer of Peruća dam, Prof. Dr. E. Nonveiller, commented, during an interview with *Water Power & Dam Construction*, that the curved axis of the structure would have been helpful in decreasing the effect

of the explosions on the dam core, as the normal load of the reservoir water on this type of dam would have had the effect of compressing the central clay core. (Peruća had been the first dam of this type to be built in the former Yugoslavia; all the subsequent rockfill structures had been designed with curved axes.)

Discussing the sequence of events on 28 January and their impact on the dam, Prof. Nonveiller said that in the first seconds after the explosions, very high air pressure would have built up in the galleries (hot gases emanating from their outlets had burnt the trees in the surrounding area); the first effect on the dam body would have been elastic deformation, followed by plastic deformation, and then, as the gas pressure dropped, deformation by gravity.

As a result of the left bank explosions, water had poured at a rate of about 20 m^3/s through the drainage gallery and out over the powerplant. The right bank explosion had caused water to emerge at a rate of about 0.6 m^3/s from an access gallery to the longitudinal gallery (see also *WP&DC* March), but this gallery had then been blocked with material, and the leakage had been reduced to 0.005 m^3/s.

The most serious problem, Prof. Nonveiller continued, was regressive erosion of the core. He considered that, while it would not be necessary to drain the reservoir completely to ensure the stability of the dam, the level would have to be drawn down to at least el. 315-320. The increasing load on the dam caused by the spring high water period (March and April) could be dealt with by discharging water through the powerplant, if necessary, he added.

Settlement was now taking place (see pp20 and 21). Deformations were far more pronounced on the upstream side of the dam, Prof. Nonveiller continued, the upstream wedge having less resistance; long fissures could now be seen on the upstream face, while on the downstream side there was no significant deformation.

Asked about his views on long-term remedial measures, Prof. Nonveiller felt that the most appropriate and economical solution would be the provision of a diaphragm on a concrete base, to prevent further erosion of the clay material. In his opinion, the provision of an upstream membrane would not be suitable, because of the highly karstic nature of the site. A grout curtain 1.6 km-long and 100-200 m deep had originally had to be constructed to ensure impermeability. Surface fissures following the explosions on 28 January indicated that it might be necessary to regrout to a depth of 20-30 m, Prof. Nonveiller said.

Site investigations and preliminary results

The following sections have been contributed by the various specialist Croatian institutes which are at present analysing the results of the preliminary investigations.

Seismological investigations

The explosions of 28 January 1993 at the Peruća dam were recorded on three seismographs operated by the Seismological Survey of the Republic of Croatia; these are the ones in Hvar (HVA), Trilj (TLJ) and Puntijarka (PTJ) (see Fig. 3). The distances to the seismograph stations of HVA, TLJ and PTJ are 21.1 km, 67.9 km and 242.3 km, respectively. The seismograms are of good quality, and enabled identification of the main phases (designated Pg and Sg at HVA and TLJ and Pn and Sn on PTJ) at all three stations. By using their onset times and assuming appropriate velocity models it was possible to determine the time of explosion as T_o = 10 h 48 min 55.5 ± 0.5 s (local time).

The seismogram of the TLJ station is very complex, probably as a result of the interference effect of waves originating from more than one explosion within a short time interval. As only one analogue recording is available and the time interval between the explosions is quite short, we must restrict ourselves only to some general conclusions. It seems that all of the explosions took place within 10 s, and that two of them (possibly the strongest ones) were separated by a time interval of some 3.5 s.

Determination of the energy released in the explosion (or the yield, usually expressed in kilotons of TNT), is of special interest. It is, however, a very difficult task because of the many uncertainties involved. One of the ways to approach the problem is to correlate seismic magnitude (M_L, M_S or m_b), as determined from the seismograms, with the yield when it is known. Such empirical relationships exist, but practically all data are related to nuclear explosions, and extrapolation is needed for small yields. To use such relationships one should know the seismic magnitude, characteristics of the rock in which the explosion took place, the depth, and the type of explosive used. Of all of these, only magnitude may be confidently determined for the Peruća dam explosion.

Using standard procedure for determining the Wood-Anderson M_L magnitude, the values M_L (HVA) = 2.3 and M_L (PTJ) = 2.4 were obtained. The seismogram from the TLJ station was not used because the recording was clipped.

Fig. 4 presents data on the M_L versus yield, Y, in (kt) (modified after Bolt[1], 1976, by using the m_b versus M_L relationship of Chung and Bernreuter[2], 1981). The numbers denote the 'hardness', H, of the rock; the granite rocks, dolomites, andesites were arbitrarily assigned a hardness value of 4, while partly saturated alluvium was assigned the value of 8. The figure clearly shows the decoupling effect of the alluvium. The straight lines represent regressions of the form

$$M_L = A \log Y + B \log H + C \qquad \ldots (1)$$

for values of H = 2, 4, 6, 8, 12.

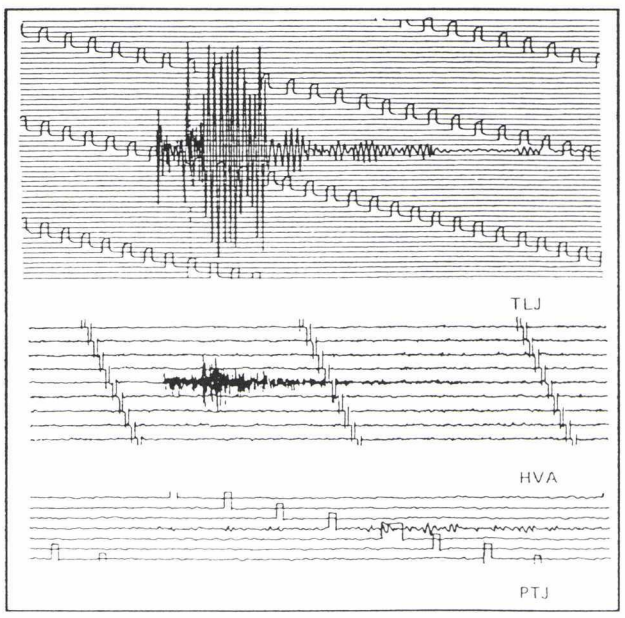


Fig. 3. Seismograms of the Peruća dam explosion(s) as recorded on three short-period vertical-component seismographs at the stations TLJ, HVA and PTJ in Croatia.

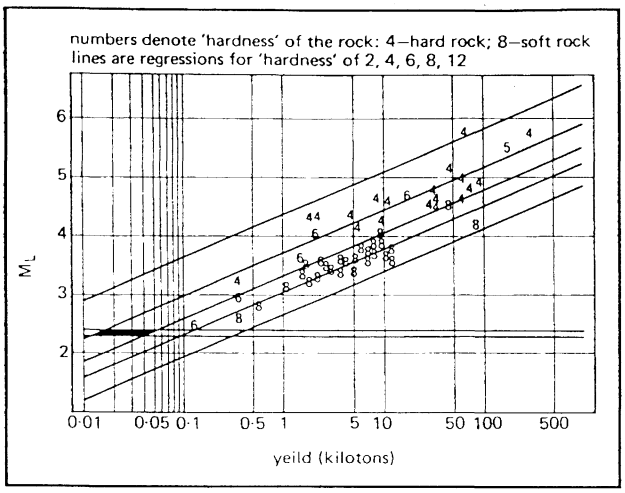


Fig. 4. Empirical data on seismic magnitude versus radiochemical yield (modified after Bolt 1, 1976). The two horizontal lines bound the magnitudes as determined for the Peruća dam. The shaded region corresponds to rock "hardness" between 6 and 4 (granites, andesites, dolomites, saturated tuff).

In Fig. 3, the region between $M_L = 2.3$ and 2.4, as determined for the Peruća dam explosion is bound by two horizontal lines. If a rock characterized by $4 < H < 6$ is assumed, one can estimate the yield to be 30 ± 20 t of TNT.

Another approach is to estimate the seismic energy, E_s, released in the explosion, using the energy-magnitude relationship. A common relationship is:

$$\log E_s = 4.8 + 1.5\, M_L$$

and $E_s = 2.5 \times 10^8$ J is obtained in this case for $M_L = 2.4$. This would correspond to the energy released by an explosion of some $Y_s = 55$ kg of TNT. If we define the seismic efficiency factor η as the ratio of the part of energy converted to seismic waves and the total radiated energy, that is,

$$\eta = E_s / E = Y_s / Y$$

we find that the value of η lies between 0.0011 and 0.0055. These values compare well with the ones reported by Bolt ($\eta \sim 0.001$ for chemical quarry blasts and $\eta \sim 0.01$ for an underground nuclear explosion in granite).

Considering several explosions occurred at the dam, and that the seismograms were the result of constructive and destructive interference of seismic waves generated by all of them, it is probably not correct to assume that the yield as estimated above corresponds to the *total* energy released. We may state, however, that the yield and seismic energy released by the strongest explosion that took place at the dam did not exceed the above estimates.

The uncertainties of the results are quite significant. It is hoped that accumulation of relevant data on explosions and increasing the number of such events recorded by seismographs will contribute to improving our understanding of these processes and the physics involved. — *Prof. Dr. M. Herak, D. Herak and V. Kuk, Seismological Survey of Republic of Croatia, A. Mohorovičić Geophysical Institute, University of Zagreb, Croatia.*

References

1. BOLT, B. A.: "Nuclear Explosions and Earthquakes: The Parted Veil", W.H. Freeman and Co., San Francisco; 1976.
2. CHUNG, D. H. AND BERNREUTER D. L. "Regional Relationship Among Earthquake Magnitude Scales", *Reviews of Geophysics and Space Physics,* Vol, 19, No. 4, 1981.

Geophysical investigations

Geophysical measurements at Peruća dam were carried out on 2 and 3 February. Their scope has been to establish the locations and extent of the damage on the dam body, particularly damage which has not been yet observed either on the dam crest or in the upstream and downstream sections of the dam body. At the beginning of the measurement work, on 2 February, the water level in the reservoir was at el. 352.0 m.

The following measurement techniques have been used:

- seismic refraction (see Fig. 5);
- self-potential measurement; and
- resistivity profiling (mapping) (see Fig. 6).

When the geophysical measurements began, the danger of mines exploding had not been entirely eliminated, and only access to the dam crest was possible; this greatly influenced the extent and the methods of the measurements.

- *Seismic refraction measurement* was carried out on two profiles, each being 230 m long, with 24 geophones and three energy source points. Because of a 60 m length of the profiles overlapping, the total investigation length was 400 m.

On the basis of the seismic refraction measurement results, it has been possible to evaluate the most significant damage which occurred at the left abutment and in the central part of the dam. Very small velocities of the primary waves (approximately 800 m/s) have been registered along the dam crest, and velocities of 1000-1250 m/s in the zones inside the dam body. Small velocities, ranging from 1000 to 1100 m/s, were also registered in a narrow zone and at the edge of the right abutment of the dam.

In the remaining zones which sustained only minor damage, the velocities of the longitudinal elastic waves ranged from 1540 to 1670 m/s. It has also been possible to establish damage to the bedrock beneath the gallery where the explosion took place.

- *The measurement of (spontaneous) self-potential* was done at 56 measuring points, 28 at each of two profiles, on the upstream and the downstream side of the road on the

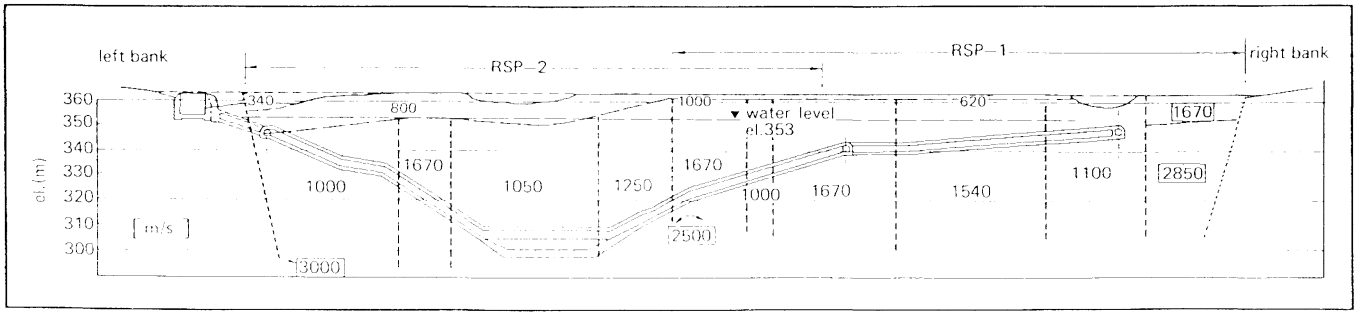


Fig. 5. Geophysical investigations: seismic refraction test results.

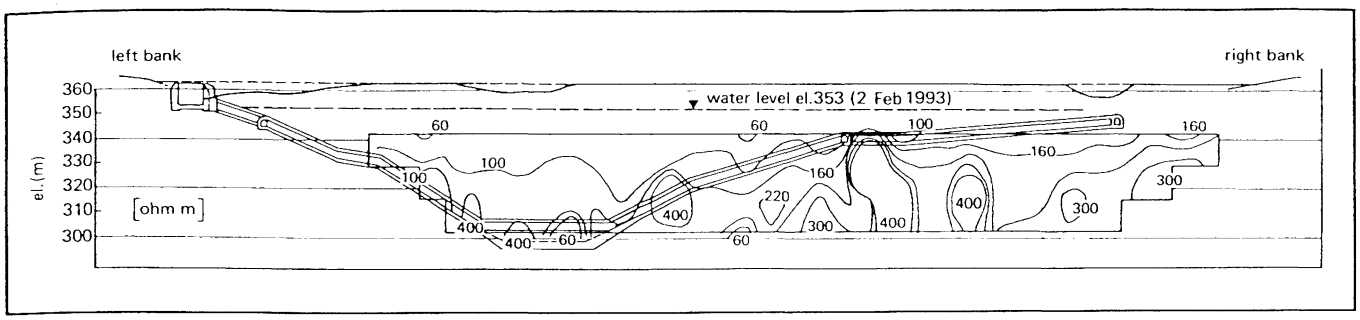


Fig. 6. Geophysical investigations: resistivity profiling (mapping) results.

dam crest.

The self-potential map shows that along most of the dam crest, the equipotential lines have mostly been parallel with the dam axis. This means that the directions of filtration are perpendicular to the dam.

Towards the right abutment of the dam, a deflection of the direction of filtration from the vertical can be seen, which indicates a filtration component in the direction of the dam axis.

• *Resistivity profiling (mapping)* was carried out at 35 measuring points, spaced at 10 m intervals along the dam crest. The results indicate five relatively wide zones with low values of apparent specific resistivity. This may be explained by the degradation of the material inside the dam body and in the bedrock additionally affected by water.

Within these zones some more narrow zones can be distinguished having very low resistivity values, of less than 100 ohm m, sporadically even less than 60 ohm m in the dam, and less than 200 ohm m in the bedrock.

In conclusion it can be said that there is good agreement between the results of the various methods used.

In view of the results obtained so far, the next stage of the geophysical investigations will cover the area of the entire dam and the bedrock. Thus, a large number of data will be obtained for a geotechnical model of the whole area under investigation, and this will be used as a basis for the design of the reconstruction of Peruća dam. — *A. Salković, Moho Co, Zagreb, Croatia.*

Water temperature flow direction and flow velocity

One of the investigations carried out to detect, as rapidly as possible, the extent and location of damage to the dam following the explosion was the simultaneous measurement of flow direction and temperature at several points in the reservoir. The measuring points are shown in Fig. 7. Dye (diluted Rodamine B) was thrown into the water so that its dispersion could be observed using underwater cameras and divers, as well as by simple visual methods.

These measurements were carried out between 1 and 5 February, during which time the water level in the reservoir was decreasing from approximately el. 354 to el. 351. Measurement of the current's horizontal component as well as the water temperature was done by an automatic recording current meter designated RCM7, manufactured by Aanderaa Instruments of Norway, with a pre-set measuring interval of 30 s. The accuracy of this instrument's sensors has been:

• for the direction of currents, ± 3 degrees (the results represented in Fig. 8)

• for the flow velocities 2.5 per cent (the results represented in Fig. 9)

• the water temperature ± 0.15°C (the results represented in Fig. 10).

The flow measurements were done by dropping a current meter from the boat at particular measuring points at a precisely determined time. The data measured were directly

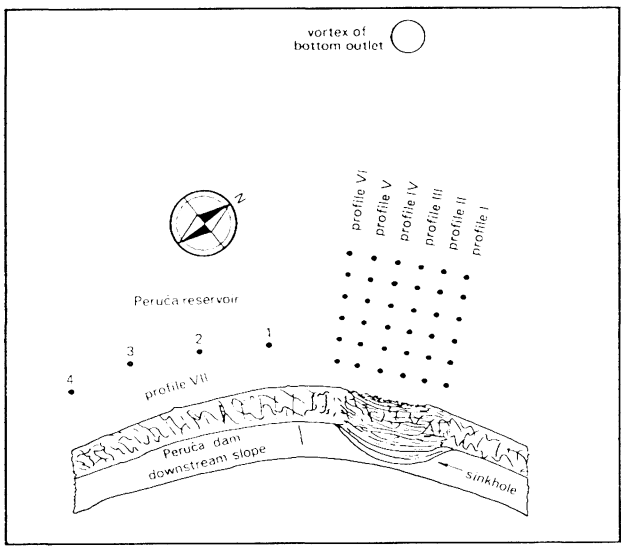


Fig. 7. The layout of the measurement profiles, with measurement points, in the Peruća reservoir near the part of the dam body which was most seriously damaged.

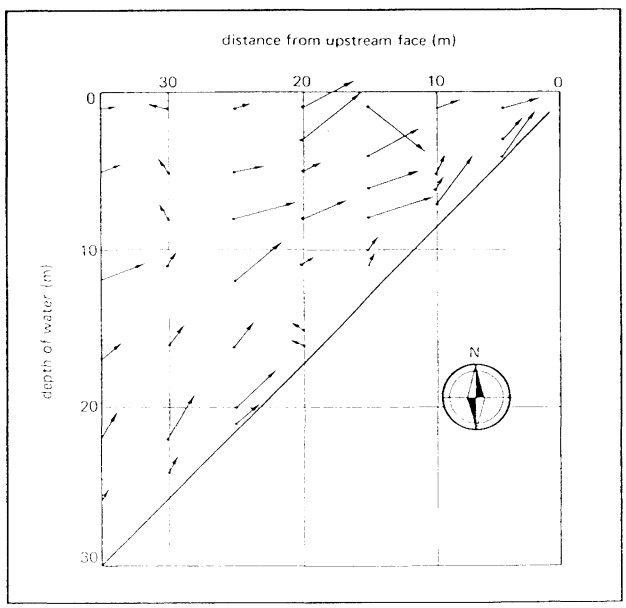


Fig. 8. Water flow measurements: distribution of the direction of water flow) near the slope of the dam on profile V (see Fig. 7).

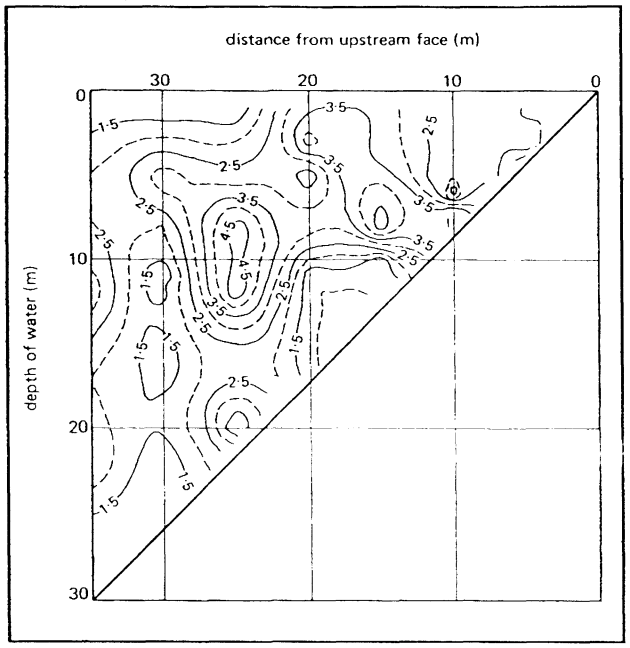


Fig. 9. Water flow measurements: distribution of water velocity (m/s) on profile V.

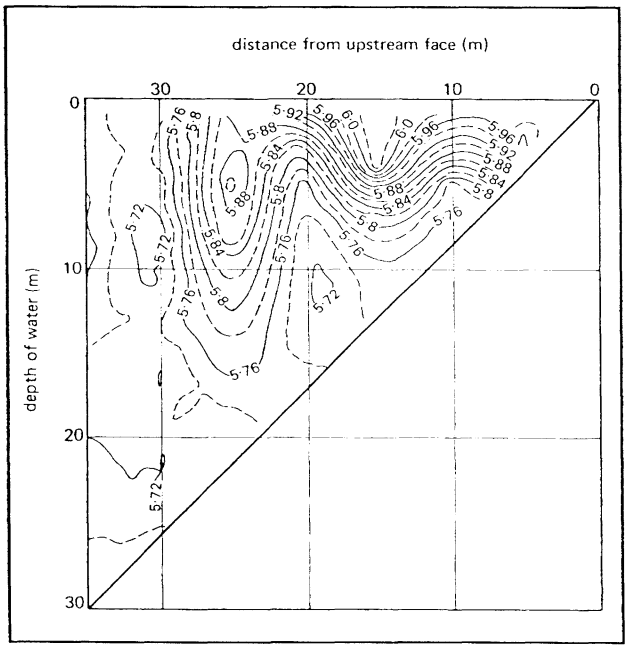


Fig. 10. Water temperature measurements: distribution of water temperature (°C) on profile V.

read by an ultrasonic receiver and a registering device on the boat.

Observations of flow direction (horizontal and vertical) were done using Rodamine B dye, which was thrown from the boat at each profile adjacent to the dam and at a distance of 10 m from the dam. The horizontal and vertical dispersion of Rodamine B was visually observed from the boat.

Among the areas damaged by the mining of the dam, a 35 m-long area of subsidence on the dam has been of particular interest. In this area most measurements were made in the reservoir. Subsidence on the dam crest was established as being 120 m from the spillway.

The measurement results have shown that the flow direction at all the measuring points was from the water surface to the bottom and along the dam from its southwest towards its northeast part (from 40° to 80° in relation to magnetic

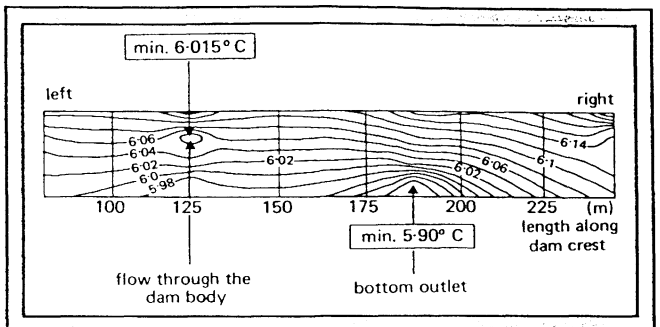


Fig. 11. Distribution of temperature on the dam's upstream face on 1 February 1993. The temperature at the gallery outlet was 6-12°C.

north).

The flow velocities were relatively small, as expected, in view of the position of the measuring points, the meteorological conditions, the dimensions of the reservoir, and the water outflow through the bottom outlet.

Certain changes were noticed in the direction of the flow in the lower water levels adjacent to the dam slope at the measuring points on profiles II and III, that is, at depths ranging from 10 to 15 m. The flow direction at these levels was about 120°, which coincided with the perpendicular on the dam axis. This indicated that in this part of the dam, some quantities of water were flowing towards the dam, and since the same flow direction was observed at two profiles, it could be concluded that the water was flowing through cracks in the dam core.

This conclusion was also confirmed by visual observations of the dispersion of the Rodamine B. At all the observation points, the direction of flow shown by the Rodamine B was parallel with the dam axis, except at profile III, where the currents coloured by the Rodamine B were directed towards the dam.

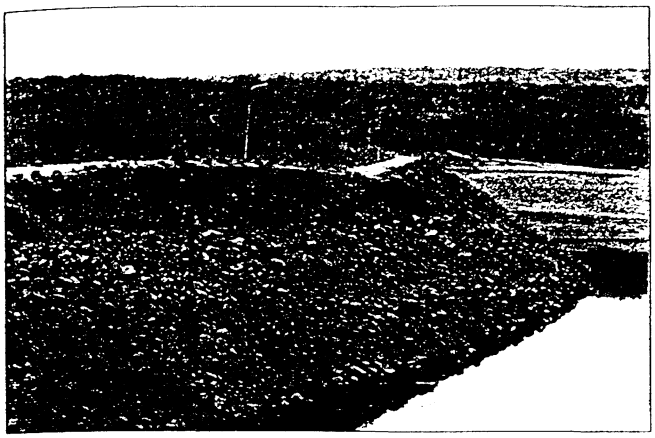
The water temperature at all the measuring points was quite uniform, ranging from 5.68°C to 6.5°C, and in general it was higher in the surface layers than in the bottom layers. Some exceptions were established at several measuring points, where the temperature was higher in the lower layers than nearer the surface. This was probably the result of the lowering of the warmer surface water, but it could not be correlated exactly with the flow of water towards the dam.

Underwater inspections of the upstream face of the dam and the penstock inlets were carried out by British and Croatian divers. The divers observed all the signs of the flow direction being towards the dam body, and at the same time they searched for any possible remaining mines. During this inspection it was found that substantial quantities of rip-rap had been displaced. — *V. Dadić and B. Grbić, Institute of Oceanography, Split, Croatia.*

Settlement of the dam body in the first 20 days after the explosions

The crest length of Peruća dam is 450 m. After completion of the dam in 1961, 19 benchmarks were installed to monitor settlement, and these were used for regular surveying of settlement for 32 years, until the occupation of the dam in September 1991. Since the day after the explosion on 28 January, surveying of 18 of the benchmarks has resumed (the other one, near to the spillway, was destroyed in the explosion).

Measurements have been taken every day at noon. The results of these measurements show the extent of settlement of the dam body which has resulted from the explosions



The upstream face of Peruća dam on 13 February (when the reservoir level had been drawn down to el. 345 m). Note areas which have been backfilled, and the depression in the centre of the crest.

and the rapid drawdown of the reservoir. The results presented here are in a simplified form.

The spillway is located on the dam crest in a section extending from 8 to 20 m. The most extensively damaged part of the dam crest has been registered as the section extending from 125 to 160 m. Thus, the dam body can be divided into three sections. Settlement results, in mm, are represented in the Table below for the time period 30 January until 17 February (for every fifth day of measurement).

One of the main explosions was below the section from 125 to 160 m, and there the most severe subsidence occurred. At this point one of the benchmarks was destroyed during the remedial works. The other benchmarks in this section show major settlement at the point where the dam was damaged. The subsidence was as deep as 5 m, although as part of the temporary remedial work this crater was backfilled with random material.

The data obtained for the section of the dam between the destroyed spillway and the central area of subsidence indicates greater settlement than those data obtained for the remaining part of the dam body in the section from 160 to 410 m.

Settlement is continuing, and this surveying work will carry on. — *M. Zec, Croatian Electric Power Authority, Split, Croatia, and M. Tomić, Institute for Surveying, Split, Croatia.*

Settlement (in mm) for various sections of Peruća dam

Date of mesaurement	Section 20 - 125 m	Section 125 - 160 m	Section 160 - 450 m
30 January	10 - 29	35 - 84	2 - 17
3 February	31 - 82	160 - 359	7 - 82
8 February	63 - 145	327 - 599	9 - 111
13 February	107 - 220	425 - 893	13 - 209
17 February	141 - 311	619 - 1168	15 - 276

Water outflow and flushing out of core material

When the explosions at Peruća took place on 28 January the water level in the reservoir was at el. 356.22. As one of the explosions demolished the spillway headworks, water was able to flow over the concrete sections of the demolished spillway (see cover picture). The gallery at the bottom of the dam core and the outlets of the gallery were demolished by another simultaneous explosion.

From the damaged area close to the spillway (near the left abutment), water started to flow into the gallery and to discharge through the other outlet on the right abutment of the dam, and the bottom outlet. From the lowest part of the main gallery, water also flowed through the drainage collector and ran out from the shaft into the switchyard.

To enable the reservoir to be drawn down as quickly as possible, and to reduce the hydrostatic and hydrodynamic pressure on the dam, the bottom outlet was opened on 29 January at 1440 h.

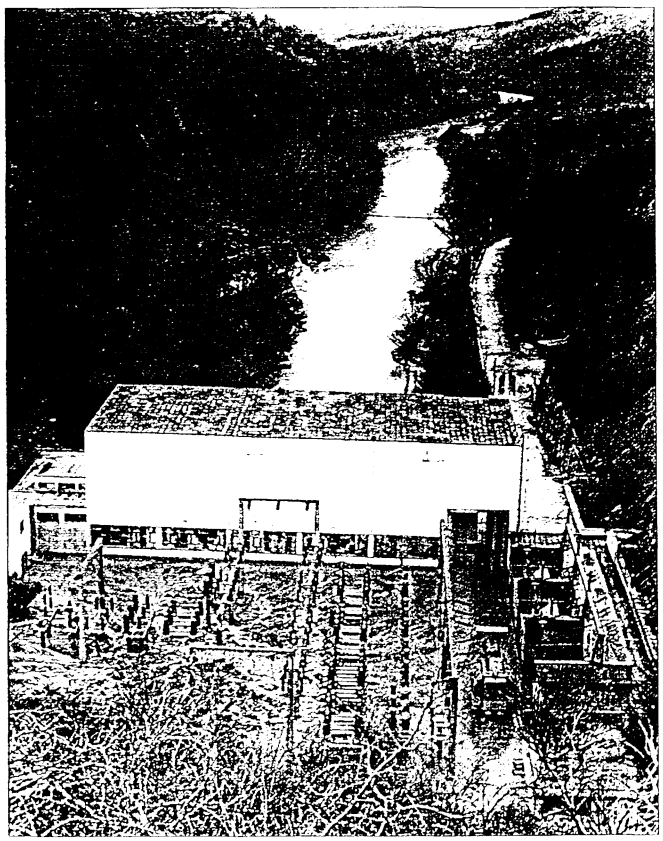
The flow from the gallery passage on the right abutment at el. 342.1 was 600 l/s. This flow gradually decreased and stopped completely on 12 February. At that time the water level in the reservoir was at el. 344.41.

From the outlet of the drainage gallery, muddy brownish-red water discharged into the switchyard at a rate of 540 l/s. In addition to the flushed core material, the water also transported large fragments of the clay core broken by explosion. These fragments were up to 50 cm in diameter. All the flushed material and core fragments were deposited on the switchyard plateau. The finer flushed material, together with muddy water, inundated the powerhouse. In the machine hall there was sediment and mud to a depth of 10-20 cm (clay material originally from the core).

Besides measurements of the material in suspension at the outlets of the gallery and the drainage gallery, the suspended material in the Cetina river downstream of the dam was also monitored.

Preliminary evaluations of the material flushed and shifted from the dam body have indicated that a quantity of more than 2000 m^3 was flushed out during the first 400 h of investigation after the explosion.

This large volume of flushed material, as well as the destroyed galleries which were either partly or completely filled with clay core material, have caused a great deal of damage, for example craters, fissures and cracks. On 16 February a new crater appeared on the downstream slope of the dam. The diameter of this crater was about 8 × 10 m, and it was about 6 m deep. — *R. Petrov, Croatian Institute of Civil Engineering, Split, Croatia.*



View of the inundated 110 kV switchyard: note the deposits of clay core material.

Soil investigations

Soil mechanics investigations are to be carried out shortly on the central clay core of the dam, when the area in the vicinity of the dam is safer. Plans for these investigations have been prepared by GeoKon Co and Elektroprojekt, both of Zagreb, Croatia.

The programme of investigations will involve drilling boreholes, especially in the areas of the dam body where the explosions took place. The boreholes will extend down to the part of the bedrock below the dam which has not been damaged by the explosion (that is, to a depth of about 100 m). Particular attention will be paid to the clay core. Sampling will be done for each 1 m of the borehole depth through the core. Detailed laboratory investigations will then be done on these samples.

During site investigations, some in-situ tests will also be done, especially permeability tests in the zone where the grout curtain has been destroyed.

The results of these investigations will be critical in determining the final decision on the design of the permanent remedial works for Peruća dam.

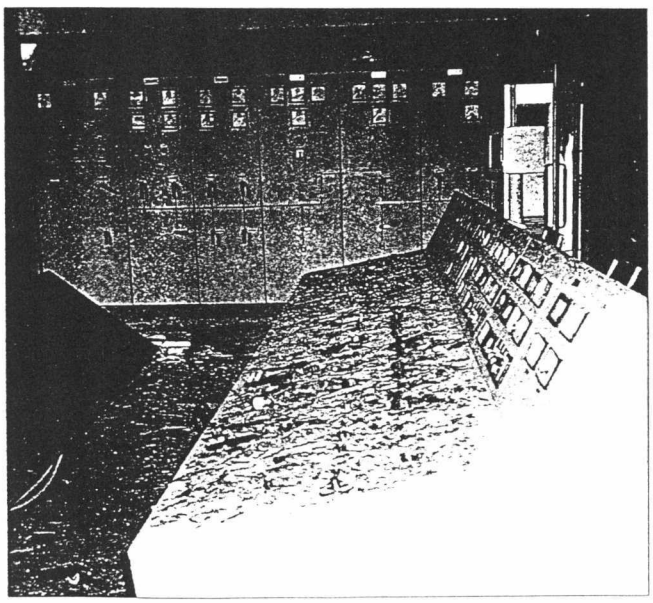
Geotehnika Co and the Croatian Institute of Civil Engineering of Zagreb specialize in this kind of soil investigation, and are prepared to undertake this work in due course. — *T. Megla, Agency for Reconstruction, Government of the Republic of Croatia, Zagreb, Croatia.*

Conclusions

Invited to summarize the current situation at Peruća, T. Megla (who described the background to the recent events at the dam, and predicted what could happen if the sabotage threats were fulfilled, in our January 1993 issue) said that nothing needed to be added to the Comment (*WP&DC* March 1993, p3) that the explosion was an act of terrorism.

He offered the following general conclusions:

- The Croatian team of experts, led by Prof. P. Stojic and J. Macan, and the workforce at the site, undoubtedly acted very quickly to prevent a much larger scale disaster, that is, complete failure of the dam. Their work was well coordinated, and these temporary remedial measures saved the main body of the dam, although it has sustained severe damage. Experts from the UK and Norway, who visited the site shortly after the explosion, agreed with all the steps which had been taken.
- The various kinds of investigations and monitoring which have been described above have enabled the state of the dam to be assessed, and the most appropriate immediate remedial measures to be selected. But all investigations carried out and data collected so far only represent an introduction to the more detailed and complex investigations to be done in the near future.
- Some possible long-term measures have already been considered, but at the moment insufficient data are available to elaborate a final design. The general idea so far is that remedial work will be carried out on the existing (875×10^3 m^3) dam body, rather than a reconstruction of the dam, as has been suggested by some experts. In designing the refurbishment project, it will be necessary to plan the execution of work in stages, so as to ensure partial utilization of the reservoir. In this way, power production at the three hydro plants downstream of Peruća dam would be increased, and the shortage of electric power in the province of Dalmatia would be eased. At the same time, the damaged section of the grout curtain has to be repaired.
- Although the threat to demolish the dam had been known for some time (see the leading article in *WP&DC* January 1993), it did not generally seem to be accepted that this could happen. In spite of the threat to the civilian population caused by the explosions on 28 January, the Croatian experts controlled the situation within a very short time, and averted a large scale disaster. The next step has therefore been to inform the world's dam experts about the incident, the immediate measures taken at the dam site, and the preliminary results of the investigation works.



The control room of the Peruća powerplant, where equipment had been systematically vandalized during the occupation of the powerplant in 1991/92 (this was not damaged further in the recent explosions).

In conclusion, it has to be said that, although Peruća dam was not completely demolished by the explosions, those who attacked the dam have, to a great extent, fulfilled their aim, because the Peruća dam and powerplant, now out of commission, were vital for the whole region (the province of Dalmatia), which will now suffer the effects of an insufficient supply of electric power. The cost of remedial work will be substantial, at a time when Croatia's infrastructure has already been severely damaged as a result of the war.

A reduction in electricity supply has already begun in Dalmatia, as of 8 March the population in this region will be without electricity for up to 10 h each day (from 0700 h to 1700 h). This situation is unlikely to improve until the remedial work on the dam has been completed. According to the most optimistic estimates, this will not take less than four years. — *T. Megla, Agency for Reconstruction, Government of the Republic of Croatia, Zagreb, Croatia.* □

Acknowledgments

Water Power & Dam Construction would like to express sincere thanks to the Ministry of Foreign Affairs of the Republic of Croatia, and the Agency for Reconstruction (in particular T. Megla), for arranging a site visit to Peruća in early February.

We also gratefully acknowledge contributions, data and assistance with coverage of the events at Peruća from: J. Macan, Manager of the Peruća powerplant; M. Vilović, Croatian Electric Power Authority; T. Megla, Agency for Reconstruction; Prof. Dr. E. Nonveiller, Designer of Peruća dam; Dr. J. Rupčić, President, Croatian Committee on Large Dams, and all the Croatian experts mentioned in the above report who have contributed data on the investigation work. Special thanks also to J. Megla for technical translations of the investigation results.

APPENDIX 2

Terms of Reference and contract.

TERMS OF REFERENCE

FOR

NVE

regarding inspection and advice of the Peruca dam in Croatia

1. GENERAL

The Peruca dam in Croatia has been damaged in a war sabotage attac. The dam is an earth- and rockfill structure with a central clay core, 65 m high, with a crest length of 450 m. The Peruca powerplant is important as it supplies el. power to a major industrial region. Moreover, the Peruca reservoir regulates the supply of water to three more powerplants downstream on the Cerina river. The reservoir is also a source of drinking water and provides irrigation supplies for agriculture purposes.

It is said that the control equipment for automatic operation of the spillway is damaged, and that the spillway gate can only be operated manually at present. In addition it seems as the equipment for operation of the bottom outlet is serious endagered, and that the generating units are out of order as a result of damage to the control building and transformers.

The question wether the dam in its present situation represent an impending danger of partly beeing washed away, with serious flooding concequenses for the downstream populated area, is unknown.

2. WORK TO BE UNDERTAKEN

The NVE expert team shall in cooperation with Croatian authorities inspect the dam and map the damages. Based on this inspection an assessment shall be made on the imminent danger for further collaps of the dam crest, and thereby flooding of the downstream area. The conclusion on this matter shall be the basis for advice to be given on:

- urgent short term measure to prevent flooding and thereby catastrophic concequences for pepople and properties.

- permanent long term measures to bring the dam back in its function.

The situation requires that the expert team in discussion with the authorities during the visit to Croatia as far as possible present proposals for immediate required measures in order to stabilize the situation. In addition to this, proposal for solution on both short and long time measures shall be presented in a report to the Croatian authorities.

Oslo 5. Februar 1993

ROYAL MINISTRY
OF FOREIGN AFFAIRS

Inquiries to
Førstekonsulent Undheim

Our date: 9 February 1993
Our reference: SU/BIB
Your date:
Your reference:

Norges vassdrags- og energiverk
O s l o

CONTRACT WITH THE MINISTRY OF FOREIGN AFFAIRS REGARDING ASSESSMENT OF DAMAGES TO THE PERUCA DAM IN CROATIA.

Reference is made to previous contact with the Ministry of Foreign Affairs concerning the above mentioned assignment.

Norges Vassdrags- og Energiverk (NVE) is hereby engaged to carry out the assignment in accordance with the enclosed Terms of Reference dated 5. January 1993.

The following personnel will be placed at ~~NORAD~~ UD's disposal:

- Torblaa
- Kjærnsli
- Rune Dahl

The assignment includes preparatory work, field work and writing a report. The journey shall take place within the period 8 - 12 February 1993.

The work to be carried out in Croatia and Norway shall be completed within a total of 180 hours.

A report, as mentioned in the enclosed Terms of Reference, shall be submitted to the Norwegian Ministry of Foreign Affairs and concerned Croatian authorities not later than 23. February 1993. The report shall be the property of the Croatian authorities and may freely be used by them without payment of any form of royalty.

The assignment will be remunerated according to time used with a ceiling of 180 hours as indicated above.

Hourly rate for work carried out both in Norway and Croatia is NOK 450,-. The calculation of the fee for field work, including international and local travel, is to be done on a weekly basis of a 6 days working week, 42 hours and at the above rate. There will be no payment for overtime.

Postal adress: P.O.Box 8114 DEP. N-0032 Oslo 1
Office address: 7. juni plassen/ Victoria Terrasse
Telephone: +472-34 36 00
Telex: 71004 NOREG N
Telefax: +472-34 95 80 34 95 81
Cable: NOREG, Oslo

The total cost ceiling (consultancy work and travel) is NOK 140.000,-.

The journey is to be undertaken according to the conditions in "the Regulations for travelling abroad at the Government's expense". The costs will be covered by ~~NORAD~~ UD in accordance with the Regulations. The Regulations constitute an integral part of this contract and it is presumed that you are familiar with the provisions.

An invoice for the work done shall be forwarded to the Ministry of Foreign Affairs, 2. Political Division.

Remuneration will be paid on the basis of the claim submitted, which must contain a detailed description of the work performed.

Travel expenses will be covered on the basis of a submitted travel claim with description of the route, used ticket counterfoils in original and other original vouchers/receipts enclosed. Reimbursement cannot be counted on if original documents are not provided. In the event of a travel allowance being paid in advance, it is requested that the travel claim be submitted to The Ministry of Foreign Affairs not later than 14 days after the return.

The above provisions are assumed fully to include alle nature of costs in connection with the accomplishment of this assignment, also necessary insurances.

We propose that this letter, together with your written confirmation that the assignment will be carried out on the terms stated herein, shall constitute a contract between you and the Ministry of Foreign Affairs. Please give such confirmation by signing the enclosed copy of this letter and return it to the Ministry of Foreign Affairs.

Sincerely,

Knut Mørkved
Special Adviser on Humanitarian Assistance and Refugee Affairs

Stein Undheim

The above conditions are hereby accepted.

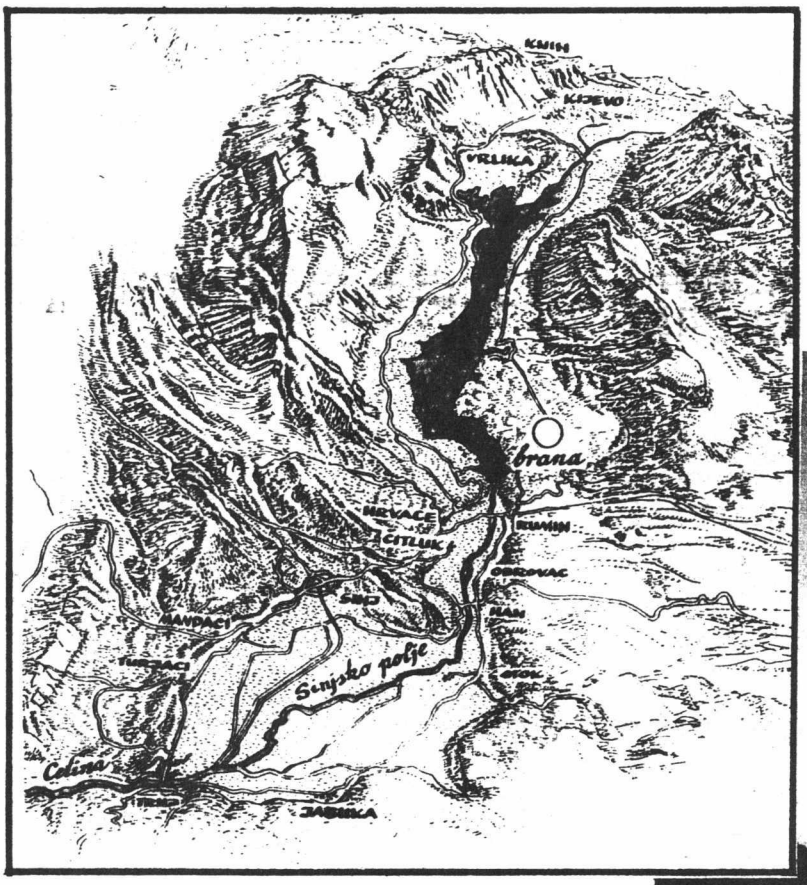
Oslo, the 22.2.1993

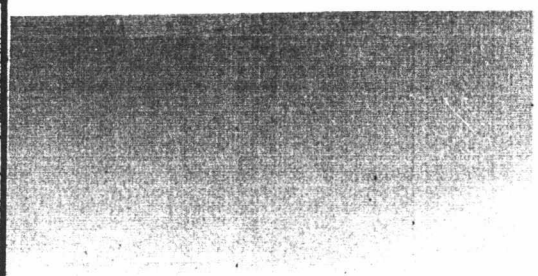
Egil Skogheland

APPENDIX 3

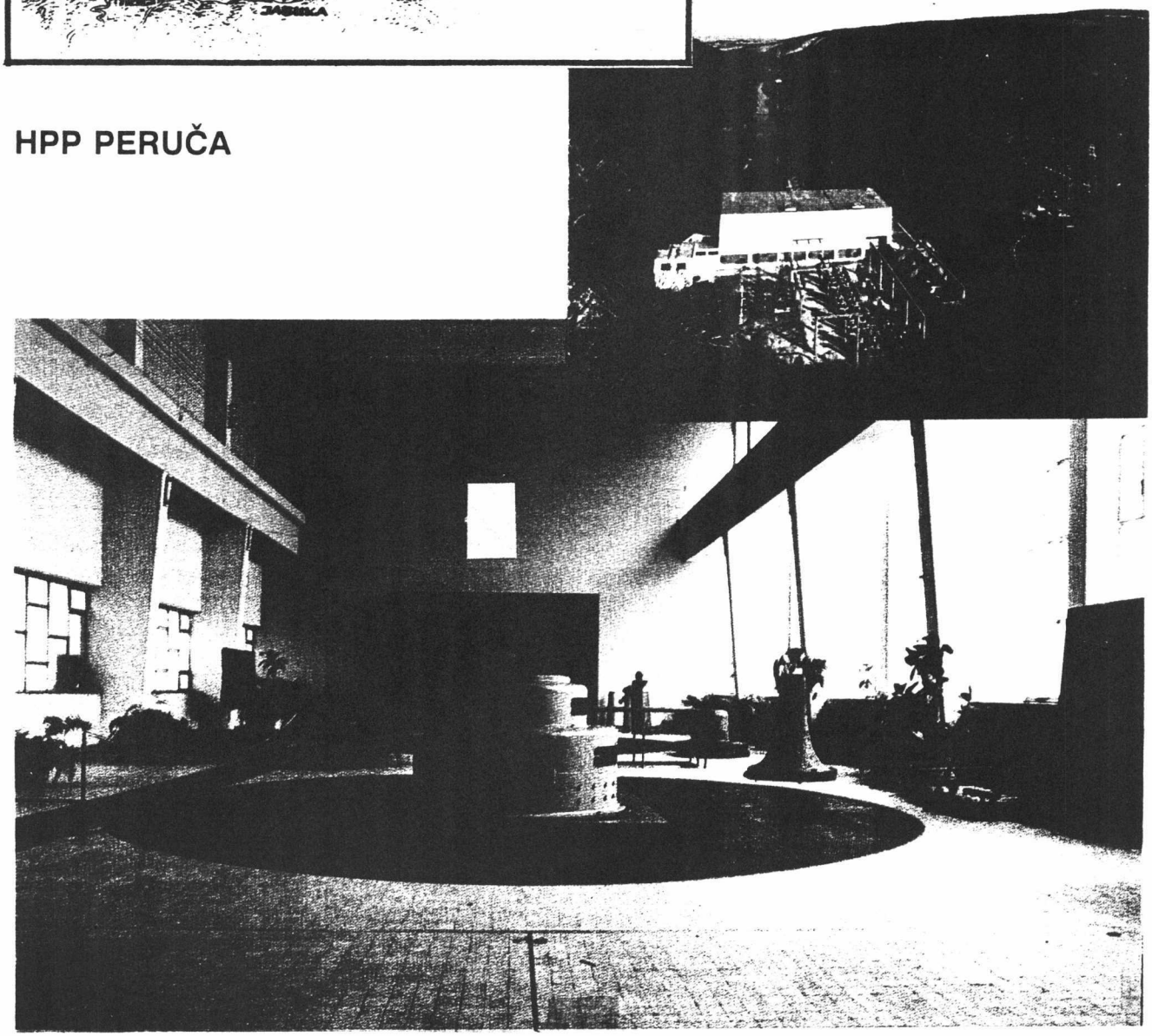
Dam characteristics.

elektroprojekt





HPP PERUČA



PERUĆA DAM

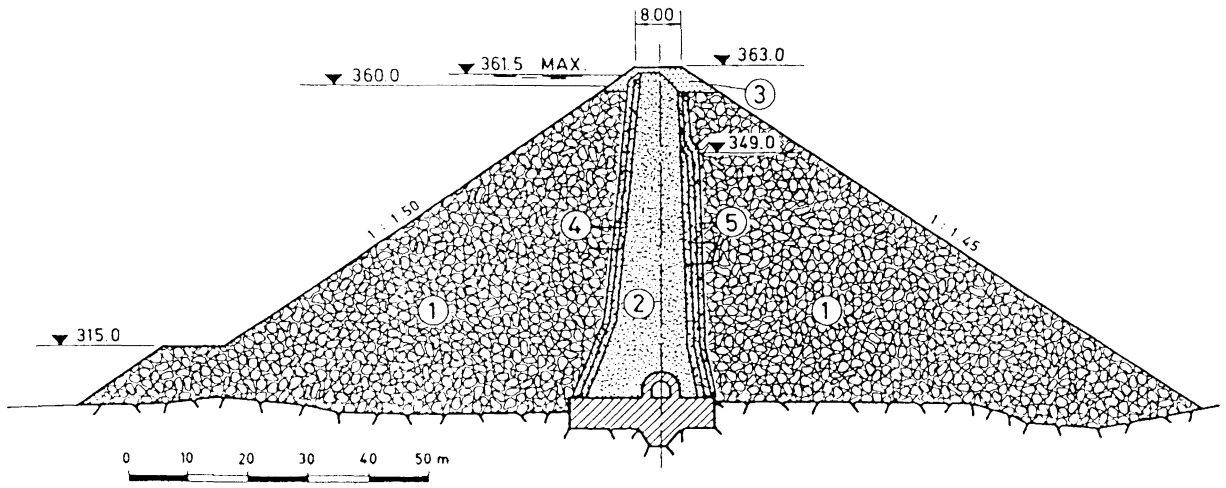
Peruća dam is a rockfill structure, with a thin clay core, of a volume of 900,000 m³, 65 m high and 450 m long at crest. It encloses a reservoir of 540,000,000 m³, designed for annual regulation of the river Cetina flow as well as for flood protection of the Hrvatačko Polje and Sinjsko Polje which lie downstream; it also stores the water necessary for operation of the remote large Split hydroelectric power station (432 MW), and of a smaller station (42 MW) located by the dam.
Both the Peruća storage and the whole drainage basin of the Cetina lie within very difficult karst area.
The decision concerning the feasibility of this project and the technical solution for impermeabilization of the reservoir were based on extensive engineering-geology investigations and studies. This was a pioneer venture in view of highly complex and difficult natural conditions under which the project was to be carried out, its successful completion having an importance that reached beyond the range of national borders.
The dam site was tightened by a grout curtain comprising 240,000 m², for which 170,000 linear meters of grout holes had to be drilled. Grouting was performed by thixotropic mixes with 30% of cement and 70% of clay with an addition of bentonite.
The spillway and chute, located at the left abutment, maximum capacity of 560 m³/s, represent a very economical solution.
The headrace tunnel, 6.7 m in diameter, has been successfully carried out through the left abutment of the dam, in geologically highly problematic ground built up of crushed limestone with irregular transitions into a weathered bauxite mass being partly exposed to an inflow of ground water.
The bottom outlet for controlled discharge of water from the storage has also been executed in karst ground, on the right side of the dam.

LE BARRAGE DE PERUĆA

La construction du barrage en enrochement d'un volume de 900.000 m³, de 65 m de hauteur, de 450 m de longueur en crête, avec mince noyau d'argile a permis la création du réservoir de Peruća de 540.000.000 m³. Il sert à la régulation annuelle du cours d'eau de la Cetina et à l'alimentation de l'usine hydro-électrique Split (432 MW), de l'usine hydro-électrique an pied du barrage (42 MW) de même qu'à la protection contre les effets des inondations des plaines: Hrvatačko et Sinjsko Polje.
Le réservoir de Peruća et l'ensemble du bassin de la Cetina se trouvent dans un terrain karstique.
La décision de construire ce réservoir et la solution technique de l'imperméabilisation du bassin dans le sol karstique a été prise en vertu des vastes travaux et d'études de reconnaissance géologiques et géotechniques. Ce travail de pionniers, compte tenu du terrain karstifié très complexe sur le plan géologique ainsi que par l'importance de la réussite, dépasse le cadre national.
Pour l'imperméabilisation du profil un voile d'étanchéité d'une surface de 240.000 m² a été réalisé; 170.000 forages d'injection on été exécutés et injectés avec des masses ticsotropes avec 30% de ciment et 70% d'argile, puis un supplément de bentonite.
Le déversoir est placé sur la rive gauche et dispose d'une capacité de déversement de 560 m³/s.
La galerie d'amenée d'un diamètre de 6,7 m de l'aménagement hydro-électrique de Peruća a été percée dans le flanc gauche du réservoir, dans un sol karstique géologiquement complexe, en calcaire broyé, avec des zones de transition en bauxite délabrée et infiltration très importante d'eau souterraine.
La galerie de vidange de fond contrôlant l'évacuation des eaux du réservoir est réalisée dans le sol karstique du flanc droit.

CROSS SECTION OF THE DAM

SECTION DU BARRAGE

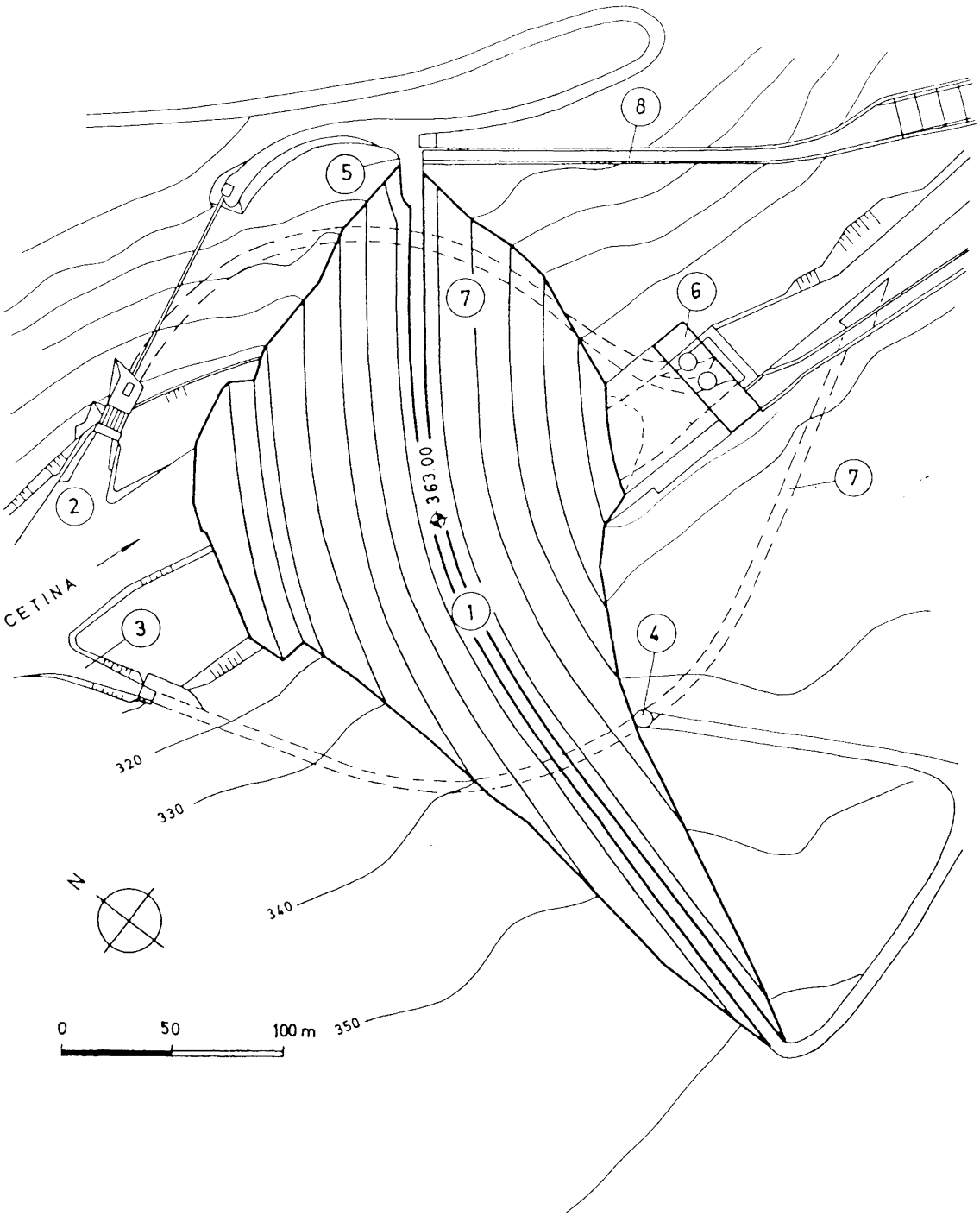


1. Rockfill
2. Clay core
3. Crushed rocks and clay
4. Filter-two layers
5. Filter-three layers

1. Enrochement
2. Noyau d'argile
3. Roche broyée avec argile
4. Deux couches de filtre
5. Trois couches de filtre

GROUND PLAN OF THE DAM

SITUATION DU BARRAGE



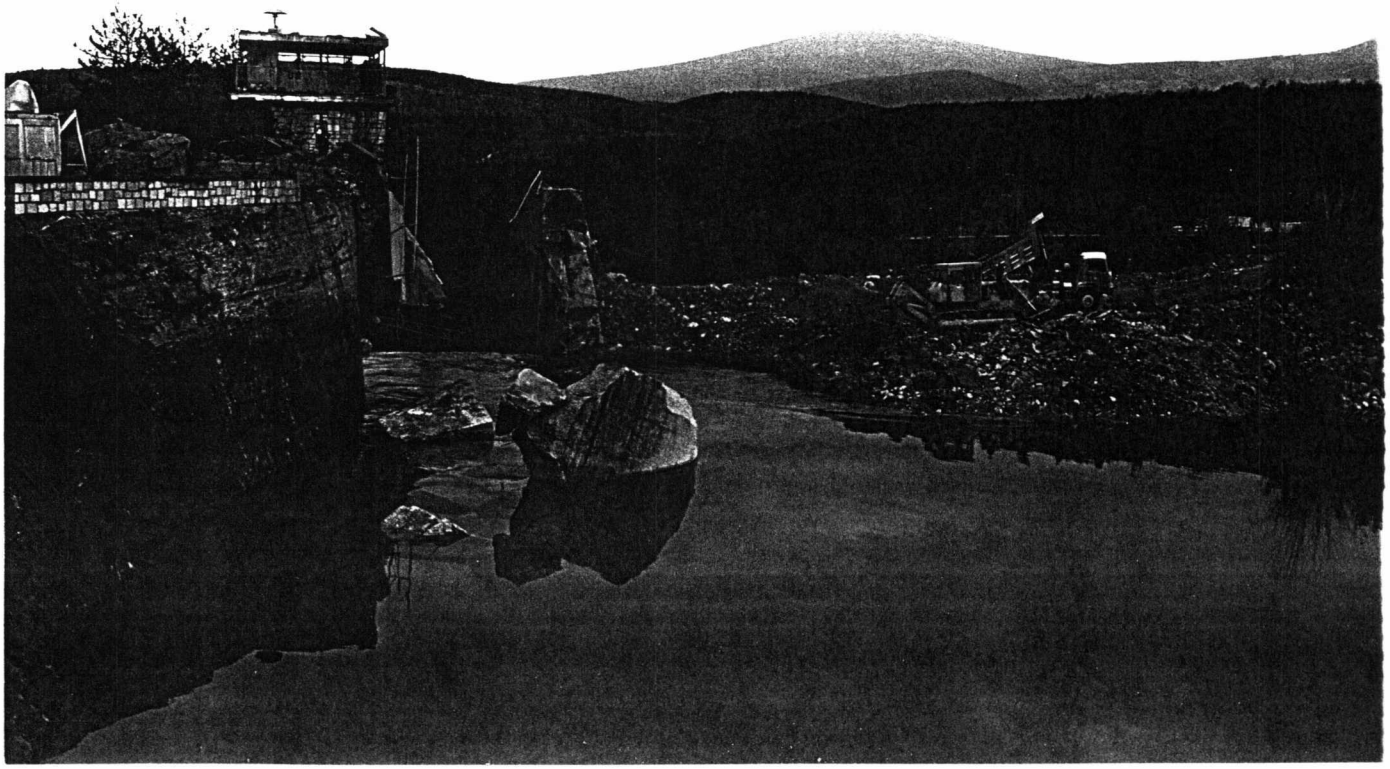
1. Dam
2. Intake structure
3. Bottom outlet
4. Gatehouse of the bottom outlet
5. Spillway
6. Machine house
7. Headrace tunnel
8. Tailrace tunnel

1. Barrage
2. Prise d'eau
3. Vidange de fond
4. Chambre des vannes de vidange de fond
5. Déversoir
6. Usine
7. Galerie d'amenée
8. Galerie de fuite

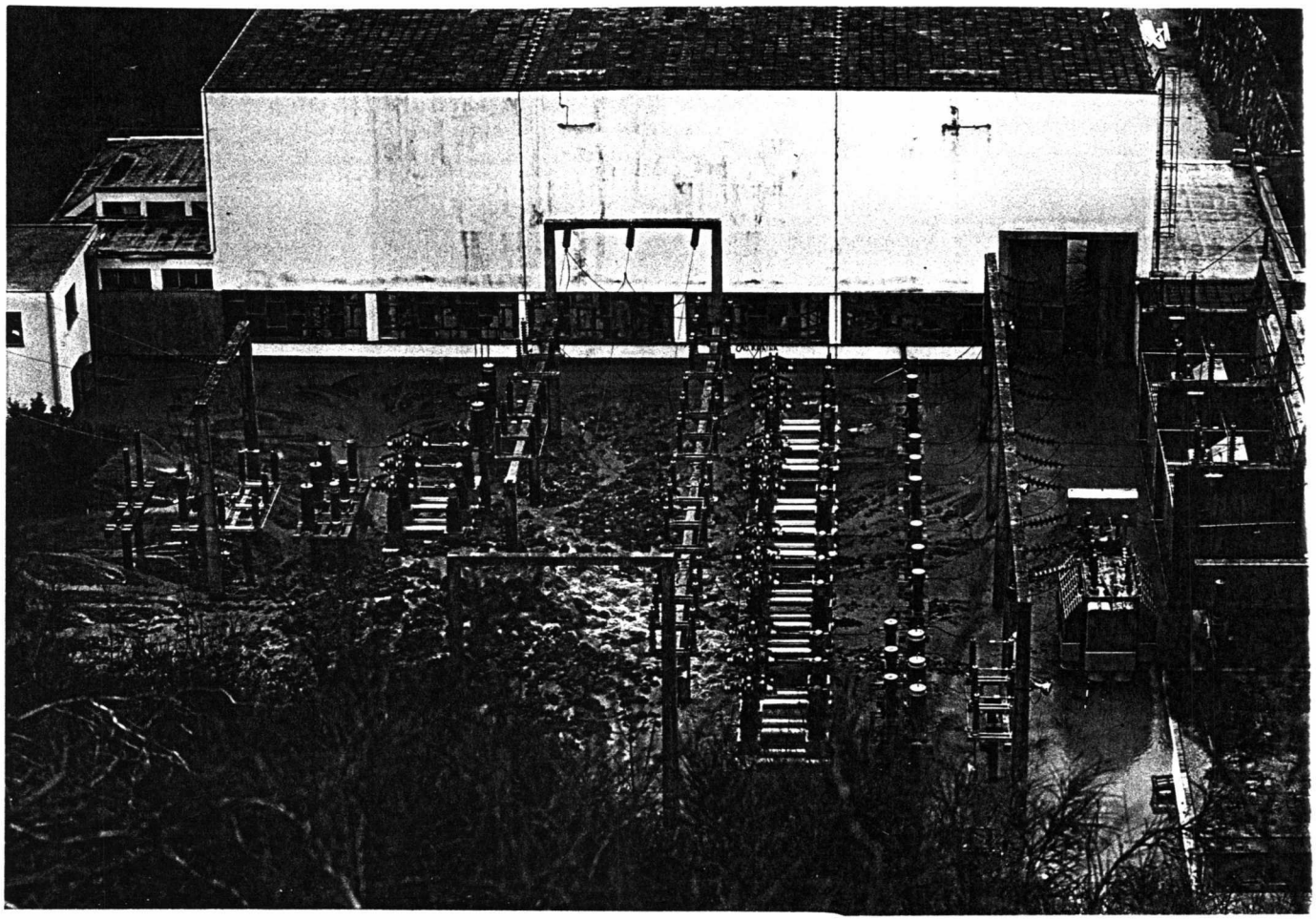
APPENDIX 4

Pictures

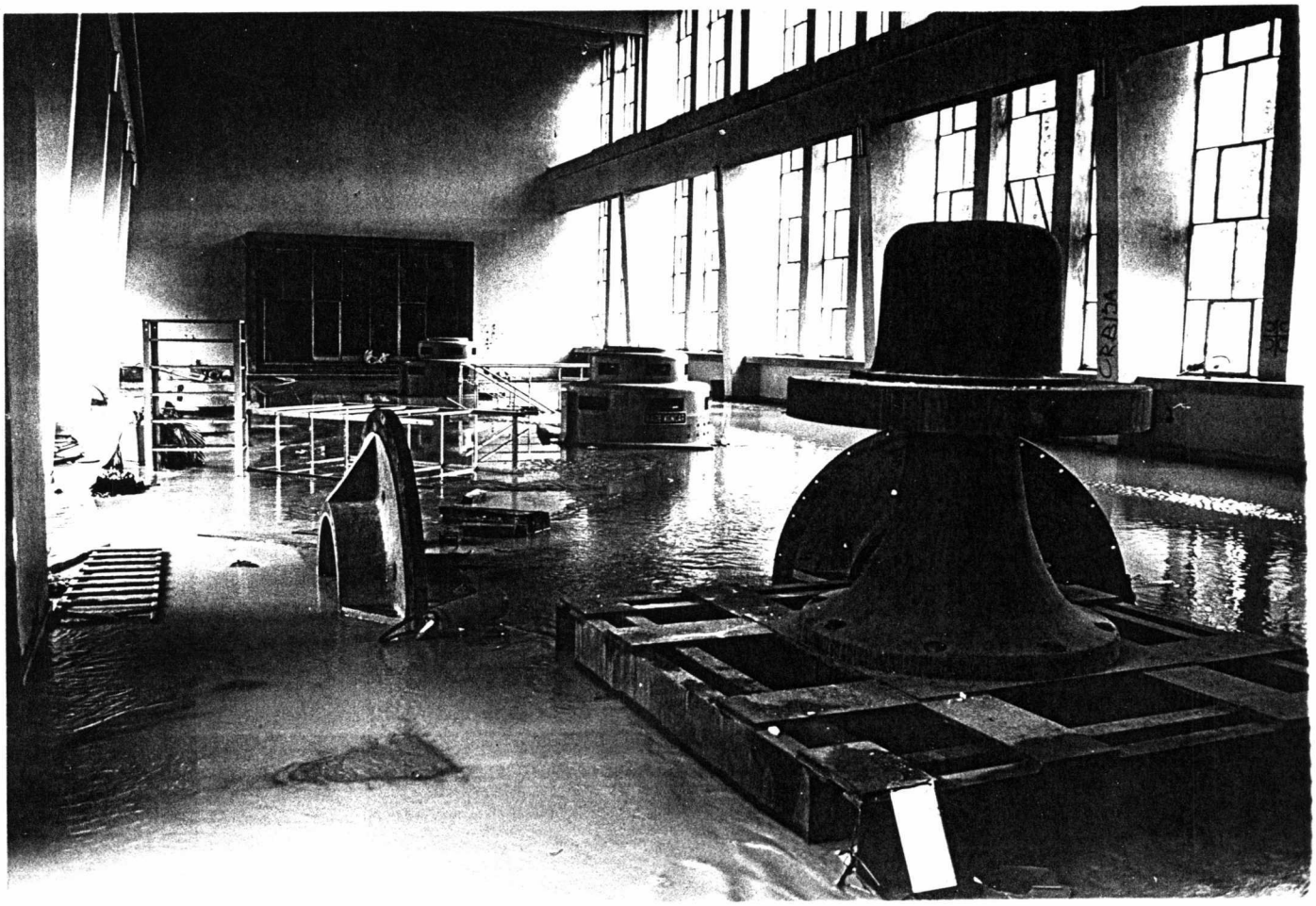
Peruca Dam, 29th. Jan. 1993.



Pictures showing the consequenses of the explosions at the left hand side of the dam, and the immediate work carried out by HEP.



The base of the switch yard was covered by three meters of fill material from the dam.

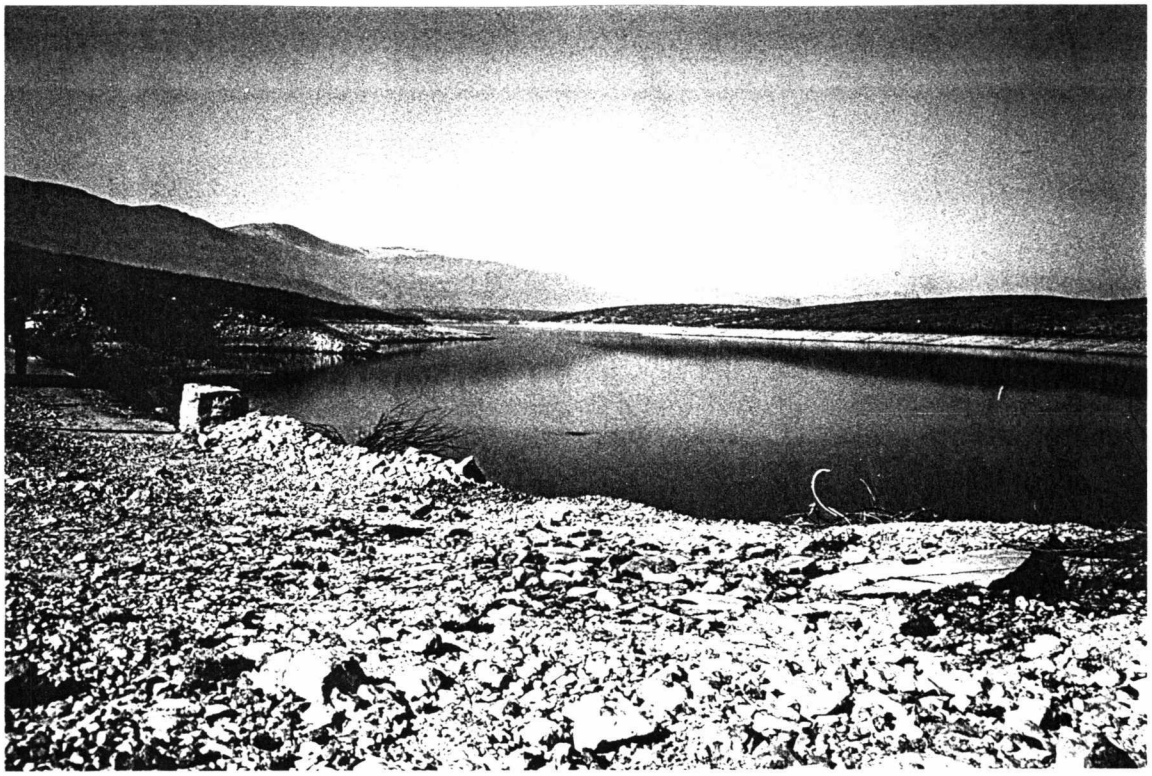


Inside power station.

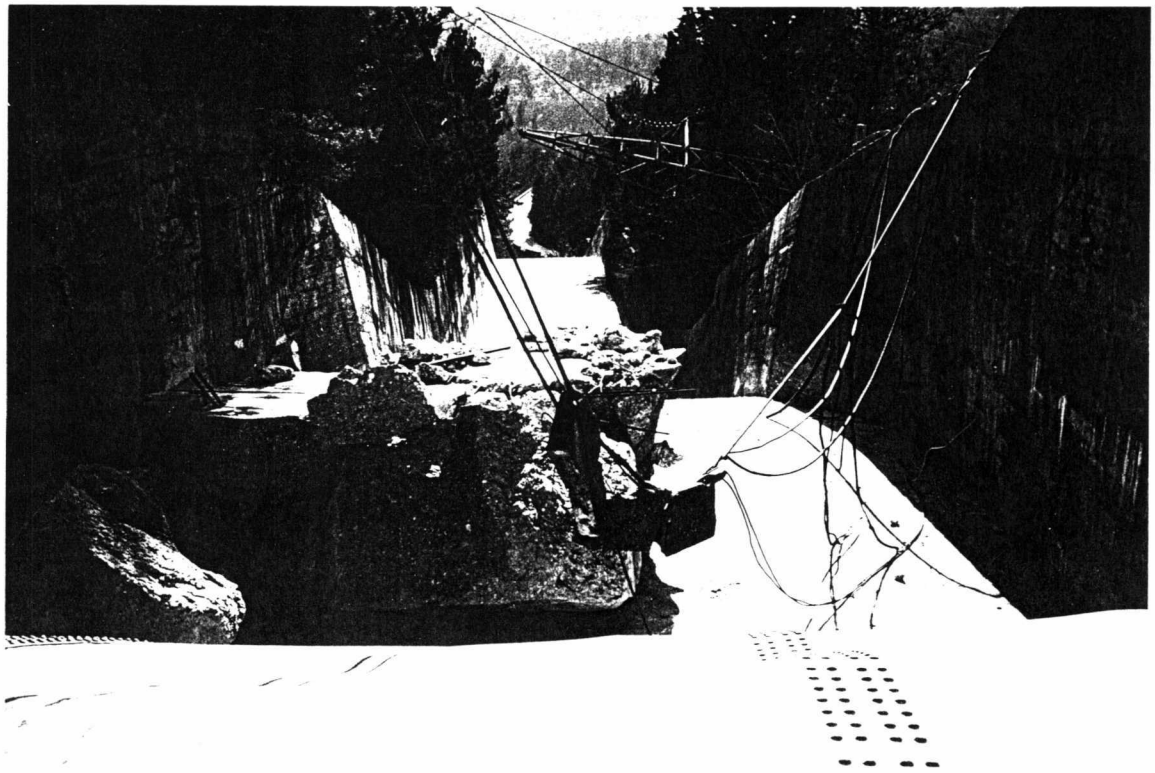
Peruca Dam, 10th. Febr. 1993.

Left hand side of the dam.

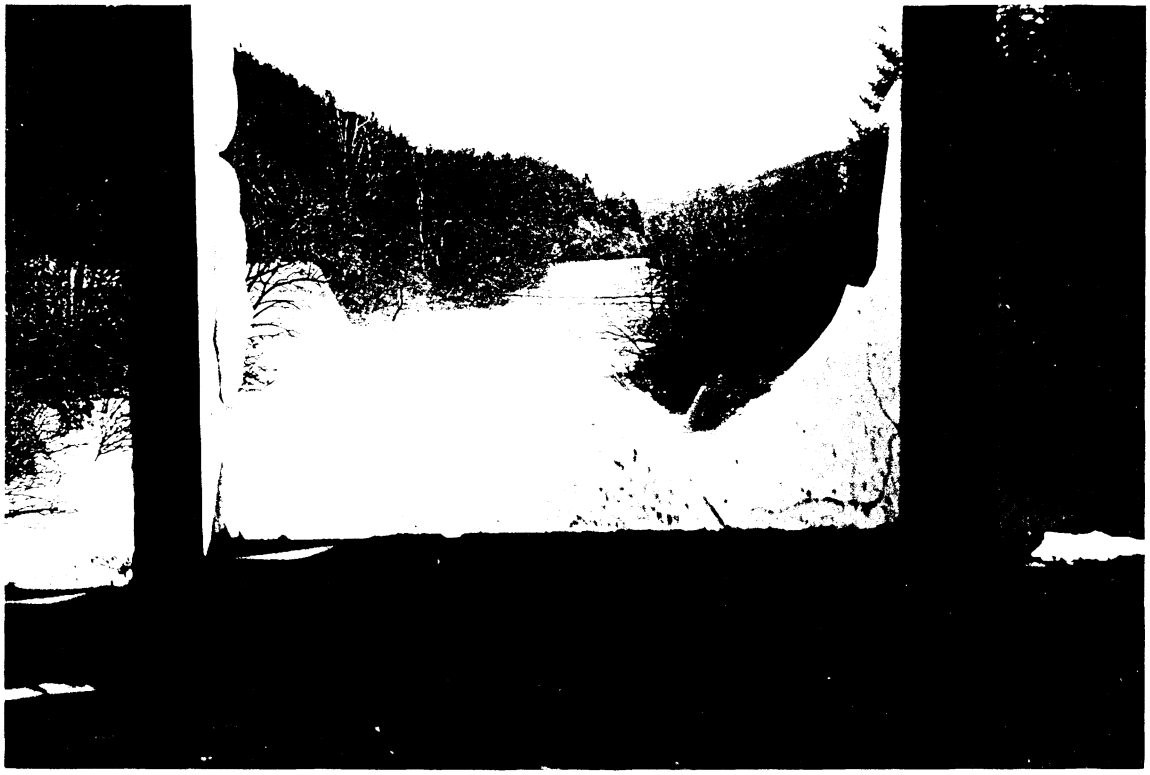
Settlement at the deepest section of the dam. About 2.5 meters.



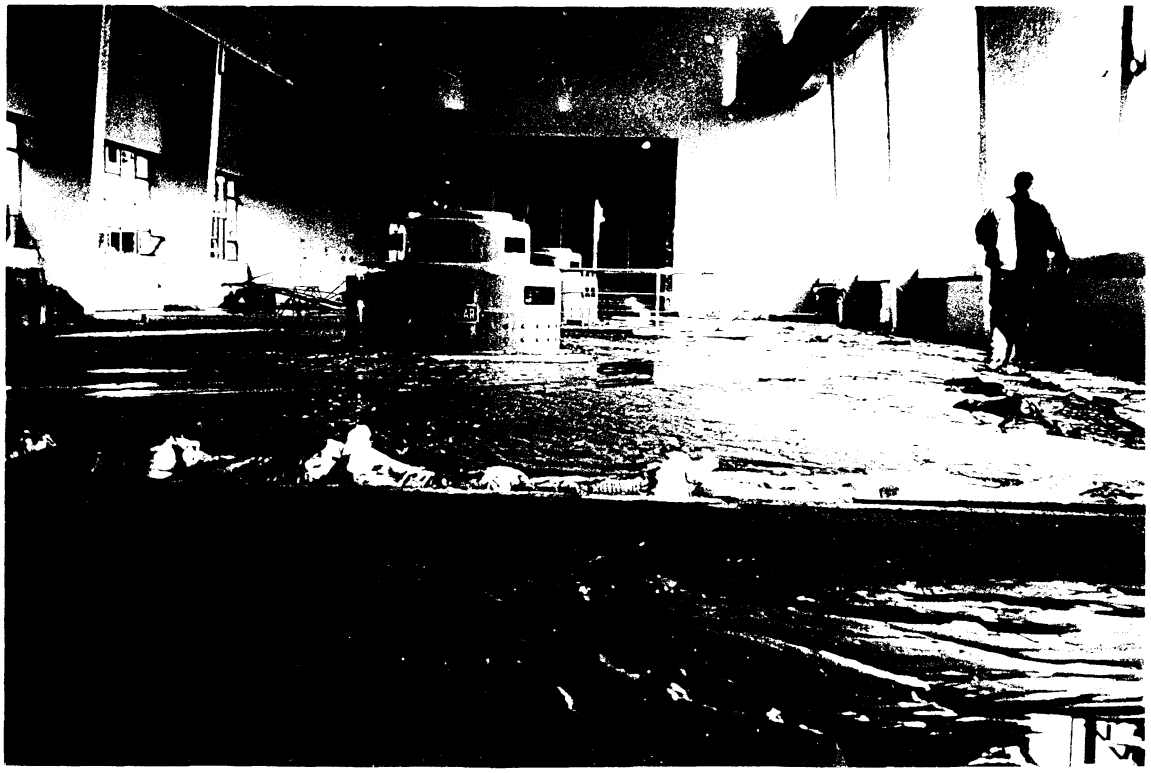
Reservoir area. Note the wake from the bottom outlet discharge.



Spillway after explosion.



Discharge through bottom outlet, about 220 m3/s.
Picture taken from inside power station.

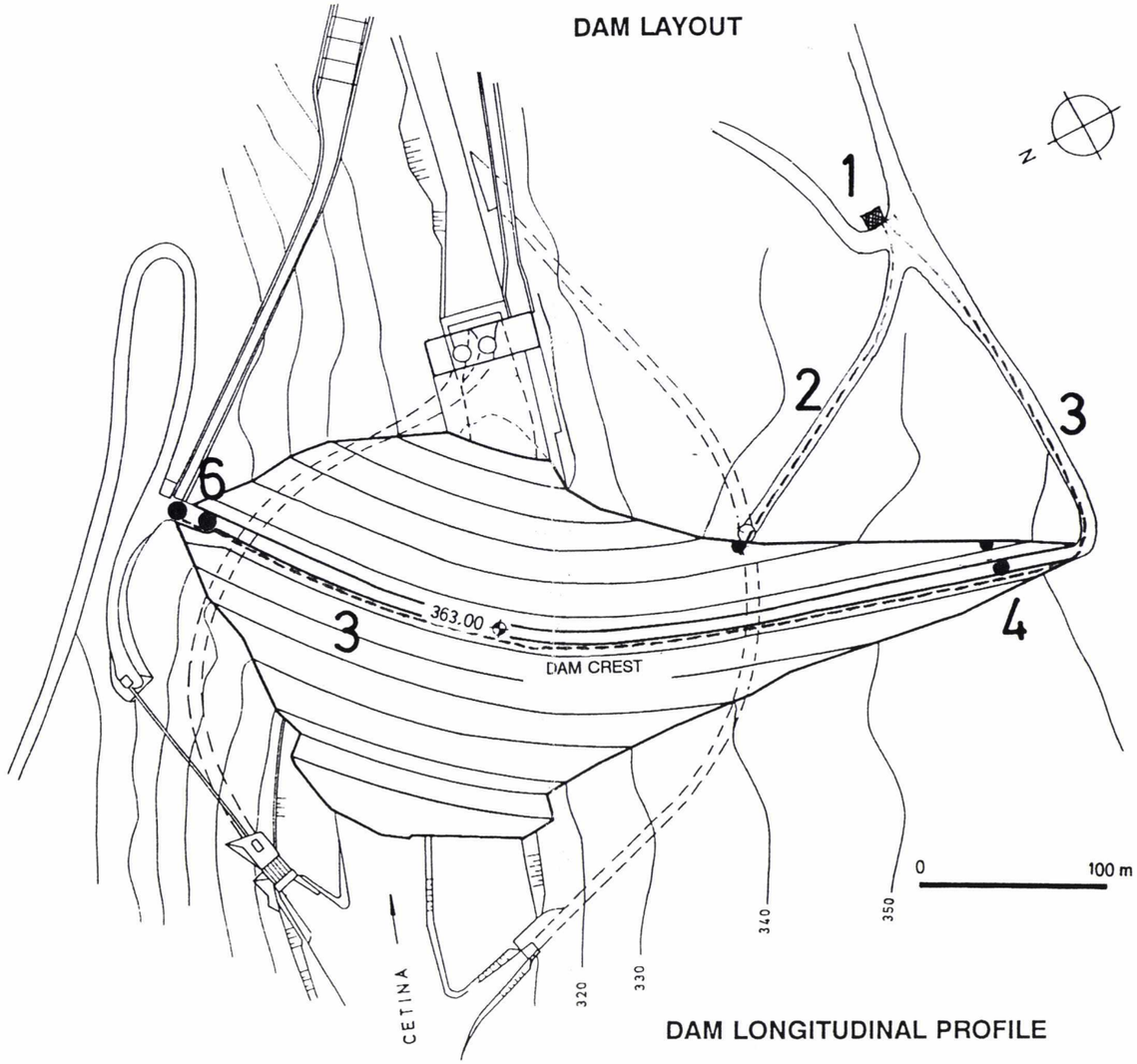


Inside power station after explosion.

APPENDIX 5

Location of explosives.

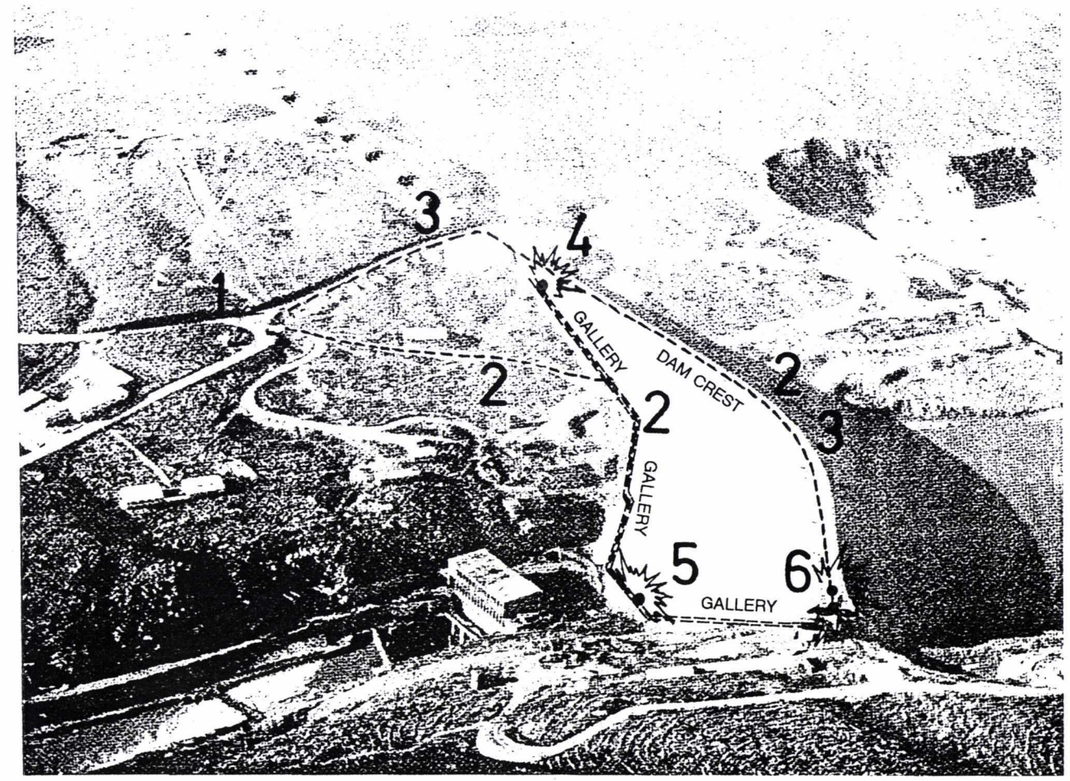
DAM LAYOUT



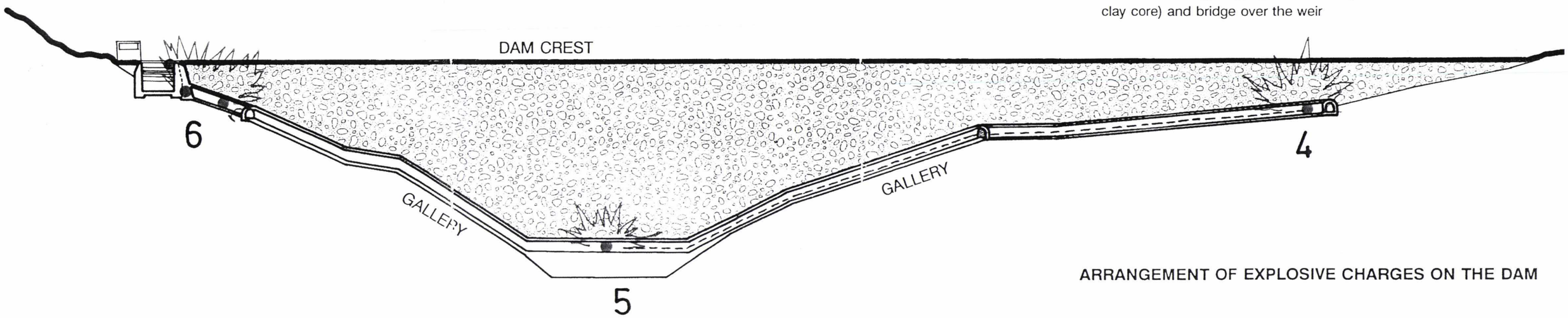
DAM LONGITUDINAL PROFILE

AERIAL VIEW OF THE DAM

elektroprojekt



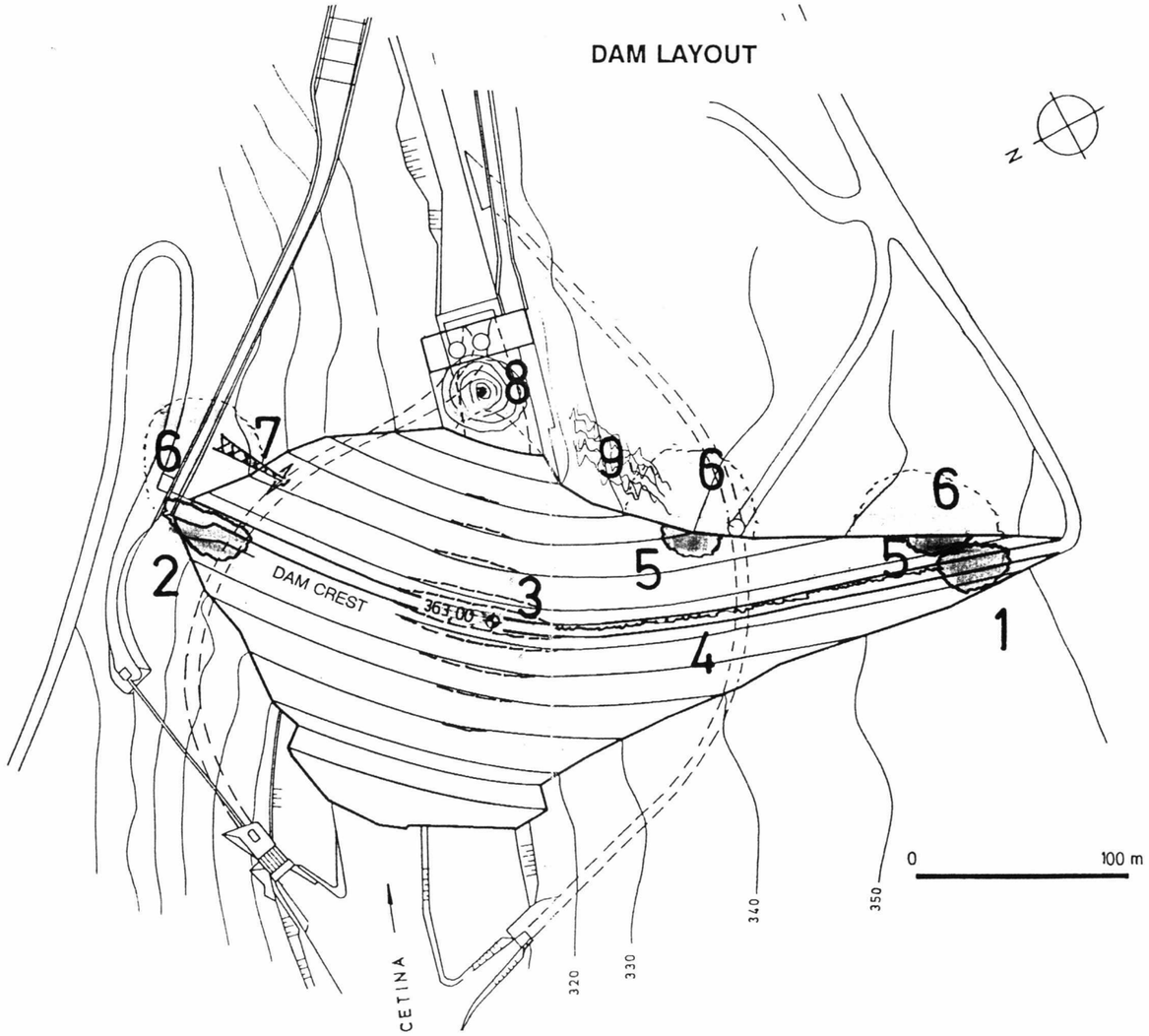
1. Location where the explosive was activated from (abandoned UN base)
2.and 3. Cables for activating the explosive
4. to 6. Locations of explosive charges in the dam
   - 4. Right end of the gallery
   - 5. Bottom of the gallery (lowest point)
   - 6. Left end of the gallery, right weir wall (connection with the clay core) and bridge over the weir



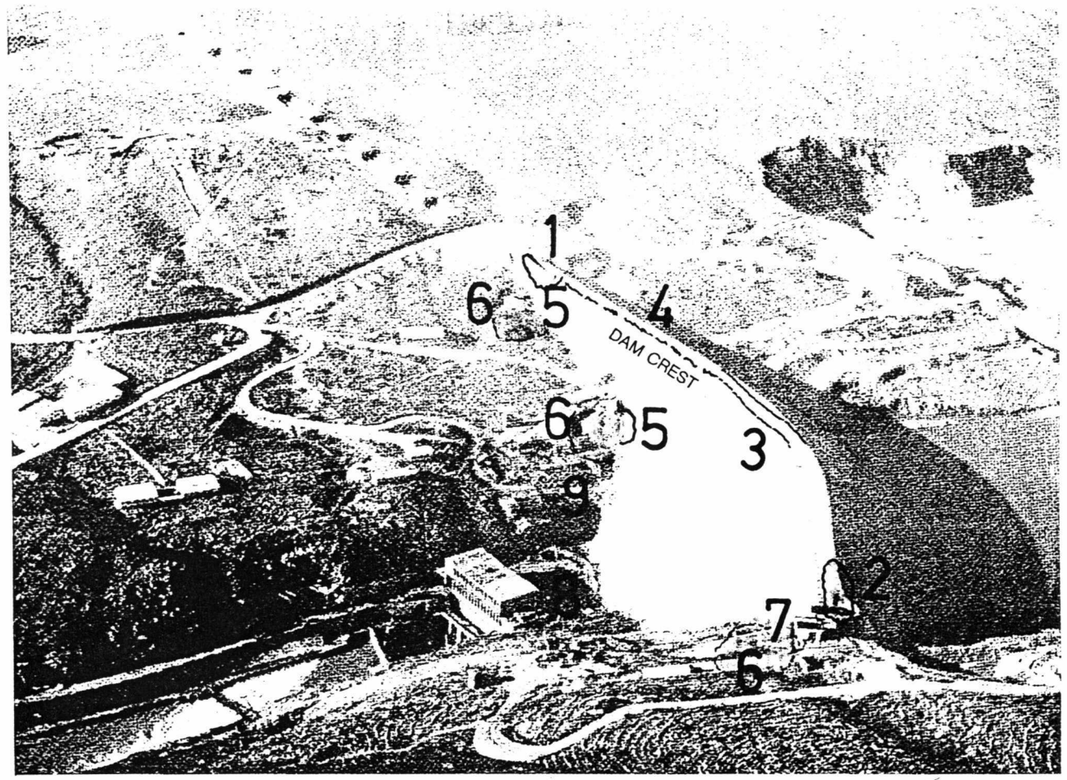
ARRANGEMENT OF EXPLOSIVE CHARGES ON THE DAM

elektroprojekt

DAM LAYOUT

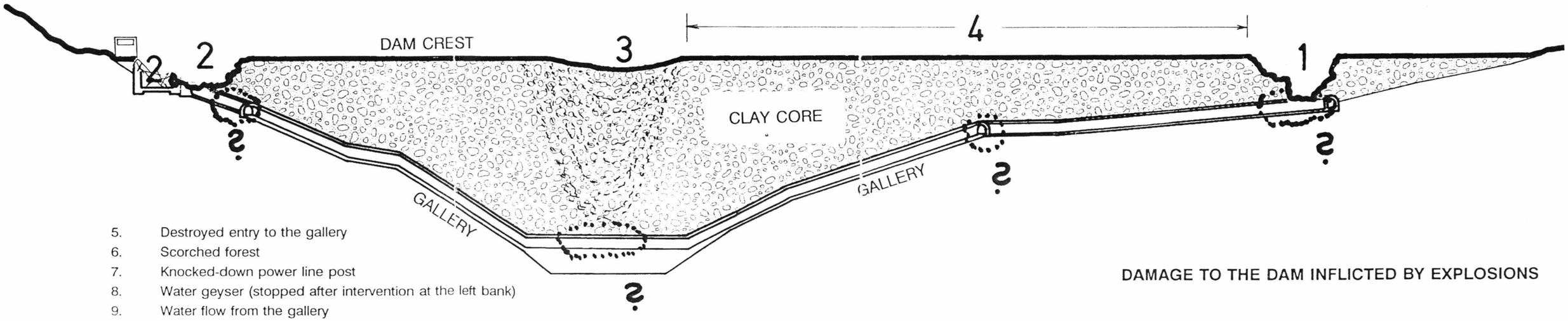


AERIAL VIEW OF THE DAM



1. Damage to the crest at the right bank (crater 25 m in diameter, 10 m deep)
2. Damage to the crest at the left bank (crater 30 m in diameter, 10 m deep), destroyed bridge over the weir and the right weir wall
3. Depression in the crest, 50 m in length, appx. 2 m deep
4. Cracks in the asphalt on the crest

DAM LONGITUDINAL PROFILE



5. Destroyed entry to the gallery
6. Scorched forest
7. Knocked-down power line post
8. Water geyser (stopped after intervention at the left bank)
9. Water flow from the gallery

DAMAGE TO THE DAM INFLICTED BY EXPLOSIONS

APPENDIX 6

Letter of intent from meeting with HEP in Sinj.

Joint statement

made in connection with the severe damage to the Dam in the Hydro Electric Power Station caused by explosives on 28th January 1993

Those present were:

1/ On the Norwegian side:
Senior Engineer and Special Adviser Mr BJORN KJAERNSLI
Senior Executive Officer of the Safety and Emergency Planning Department Mr RUNE DAHL
Senior Engineer of the Safety and Emergecy Planning Department Mr EIVIND JAKOB TORBLAA

2/ On the Croatian side:

Manager of the Hydro Electric Power Station "Peruča"	Mr JOSIP MACAN,
Senior Engineers of the Croatian Electricity Authority	Mr MARIN VILOVIĆ Mr ANĐELKO RELJA Mr RADOVAN MIŠKOV Mr MIJO ZEC Mr JOSIP RAOS
Interpreter of the Croatian Government Office	Mrs ŽELJKA ZANCHI.

The above mentioned individuals, signatories of this statement, made a visit to the Dam, powerhouse and other installations of the project on February 10th 1993. Mr Frederik Arthur, the Norwegian charge d"affairs to Croatia, was also present. Based on their joint investigation on site and a study of the available data, and following thorough technical discussion, the following conclusions were reached:

1/The damage to the Dam, which was caused by actuating the explosive in the gallery, is severe.

2/ The measures already taken at the Dam to prevent its collapse have proved to be successful.

3/ The present situation is under control and no changes that might lead to a catastrophic failure are likely to occur.

4/ At the moment the reservoir is being evacuated through the bottom outlet and investigatory works are being carried out in parallel.

5/ The Norwegian experts promised to inform their authorities on everything they had seen and established on site, after which a future cooperation and technical assistance may be agreed upon.

6/ The Norwegian experts further think that, when it comes to deciding upon a solution for the rehabilitation of the Dam, the variant allowing for a partial exploitation of the reservoir for the production of electricity in the downstream

power stations should be taken into consideration.

7/ The fact that the seepage through the gallery has decreased actually corroborates to the conclusions mentioned above /article 6/.

8/ The Norwegian experts will provide the Croatian Electricity Authority, at their request, with booklets, brochures and other materials dealing with technologies used in the world today, particularly those based on the Norwegian experience, and could be applied to the solution of problems at the "Peruča" Dam.

9/ The Croatian experts will continue cooperation with their Norwegian counterparts, and the management of the Croatian Electricity Authority will be kept duly informed.

The undersigned wish to stress the high degree of friendly and professional cooperation on the extremely serious problems and in the very difficult circumstances. The Croatian experts take this opportunity to extend their appreciation and gratitude.

Sinj, February 11th 1993

1. Bjorn Kjaernsli
2. Rune Dahl
3. Eivind Jakob Torblaa
4. Josip Macan
5. Marin Vilović
6. Andelko Relja
7. Radovan Miškov
8. Mijo Zec
9. Josip Raos

APPENDIX 7

Different reports on asphaltic concrete cores.

* An evaluation of asphaltic concrete cores for embankment dams.
  By Kaare Høeg, NGI.

* The Storevatn Dam. A rockfill dam with a central core of asphaltic clay.
  By Asbjørn Arnevik, Bjørn Kjærnsli, and Sven Walbø.

* Behaviour of Storevatn Dam, Norway - A case of prediction versus Performance.
  By G. S. N. Adikari, T. Valstad, B. Kjærnsli, and K. Høeg.

An evaluation of asphaltic concrete cores for embankment dams

By K. Høeg, Professor*

REPRINTED FROM **WATER POWER AND DAM CONSTRUCTION,** JULY 1992

An evaluation of asphaltic concrete cores for embankment dams

By K. Høeg, Professor*

Asphaltic concrete often represents a very attractive alternative to earth cores or other designs for the impervious element of embankment dams. The method has increasingly come into use internationally. In the past decade, seven large rockfill dams of this type have been built in Norway, and two more are currently under design and construction, including one 120 m high. A research and development programme is now under way in Norway to develop the technology further and to extend the applicability of the method to other geological, climatic and site conditions.

Merits of the method

Norway has a long tradition of hydropower development and dam construction. In 70 per cent of the 172 large Norwegian embankment dams, a core of morainic material has been chosen. In places where suitable moraines do not exist within an economic hauling distance, other materials have been used. For rockfill dams, a core wall of asphaltic concrete is a very attractive option, and in the past decade this method has come increasingly into use with excellent results.

Since 1978, when the method was first introduced in Norway, the equipment, placing and compaction techniques for the asphaltic core and adjacent filter and transition zones have been greatly improved. The unit costs have steadily been decreasing, which now makes this a competitive alternative even when morainic material is available locally. Furthermore, potential scars in the landscape from borrow pits are avoided.

Compared with an earth core, the placement and compaction of asphaltic concrete is almost independent of weather conditions. This enables the contractor to extend the working season, and to conduct an almost continuous operation, keeping the construction on schedule.

Asphaltic concrete is flexible, resistant to erosion and ageing, workable and compactable, and offers jointless core construction. Its viscoelastic-plastic and ductile properties provide a "self-healing" ability, should cracks develop in the core wall. Asphaltic concrete or the stone-bitumen placement procedure may also be used in cut-off walls to seal pervious alluvial deposits beneath a dam.

Table I summarizes some important data for Norwegian dams recently constructed with asphaltic concrete cores, for one which is now beginning construction, and one which is under design.

Fig. 1 shows the cross section and an overview from the construction of the largest dam of this type so far, the Storvatn dam, which has been well instrumented, monitored, documented and evaluated[1,2,3,4]. The top photograph on p34 shows the Berdalsvatn dam under construction. All the completed dams listed in Table I have performed well.

*Norwegian Geotechnical Institute (NGI), PO Box 40, Tasen, N-0801, Norway; also, Chairman of Norwegian National Committee on Large Dams (NNCOLD).

Table I — Norwegian rockfill dams with an asphaltic concrete core

Name	Height above lowest core foundation (m)	Core thickness (m)	Core inclination (v:h)	Vertical projection area of core (m^2)	Volume of asphaltic concrete (m^3)	Construction period	Main contractor/ Asphalt contractor
Vestredalsvatn	32	0.5	1:0	6000	3100	1978-80	Furuholmen/ Strabag
Katlavtan	35	0.5	1:0	4600	2300	1979-81	Furuholmen/ Strabag
Langavatn	22	0.5	1:0	3800	1900	1979-81	Furuholmen/ Strabag
Storvatn	90	0.5 0.8	1:0.2	76 000	49 000	1981-87	Statkraft/ Veidekke
Riskalvatn	45	0.5	1:0	14 600	8000	1983-86	Veidekke/ Korsbrekke and Lorck
Berdalsvatn	62	0.5	1:0	13 000	6800	1986-88	Veidekke Korsbrekke and Lorck/
Styggevatn	52	0.5	1:0	30 400	15 300	1986-90	Statkraft Korsbrekke and Lorck/
Storglomvatn (under construction)	120	0.5 1.3	1:0	~44 000	~24 000	1992-	Statkraft/ not decided
Holmvatn (under design, to commence construction soon)	56	0.5	1:0	~12 000	~6200	1995-	Statkraft/ not decided

Construction procedure

The use of the technique requires high quality construction methods and control.The asphaltic concrete is compacted at a temperature of 160-180°C and is given immediate lateral support from the adjacent zones. Placement of the core wall and the filter and transition zones proceed simultaneously, with equal layer thickness, usually limited to 0.2 m. The width of the wall is about 1 per cent of the water head, but at least 0.5 m. The core may be inclined or vertical.

The special laying machines (crawler paver) handle the core and adjacent material on either side simultaneously, discharging the asphaltic concrete immediately ahead of the gravel (see lower phtograph on p34). In the front, the machine is equipped with a heavy duty

vacuum cleaner which soaks up any dirt and water on the existing surface. Behind the vacuum cleaner, an infra-red heater is mounted, which causes any remaining surface moisture to evaporate and heats the previous layer to ensure firm, seamless adhesion to the next layer. The asphalt and gravel are placed in even and horizontal layers, automatically controlled by laser, and are simultaneously given an initial compaction. The paver follows a data-controlled centreline. Final compaction is achieved by vibratory rollers which follow the placing unit. The rollers operate in a co-ordinated manner, side by side, so as to avoid lateral displacement of the hot, asphaltic concrete (see cover photo, bottom right).

For the production of the asphaltic concrete, a controlled mixing plant is needed, together with silos, heating drums, screening and weighing equipment. Special insulated hoppers are required for hauling the hot mix, which is produced in accordance with specifications for grain size distribution of the concrete aggregates, filler and bitumen content in the mix, and temperature constraints at the various stages of the process.

Drill core samples are taken to control the in-situ compaction. The density is determined, and the air volume is calculated on the basis of the specific density of solids in the sample. The drill holes are subsequently backfilled with compacted asphaltic concrete or, occasionally, with hot bitumen mastic.

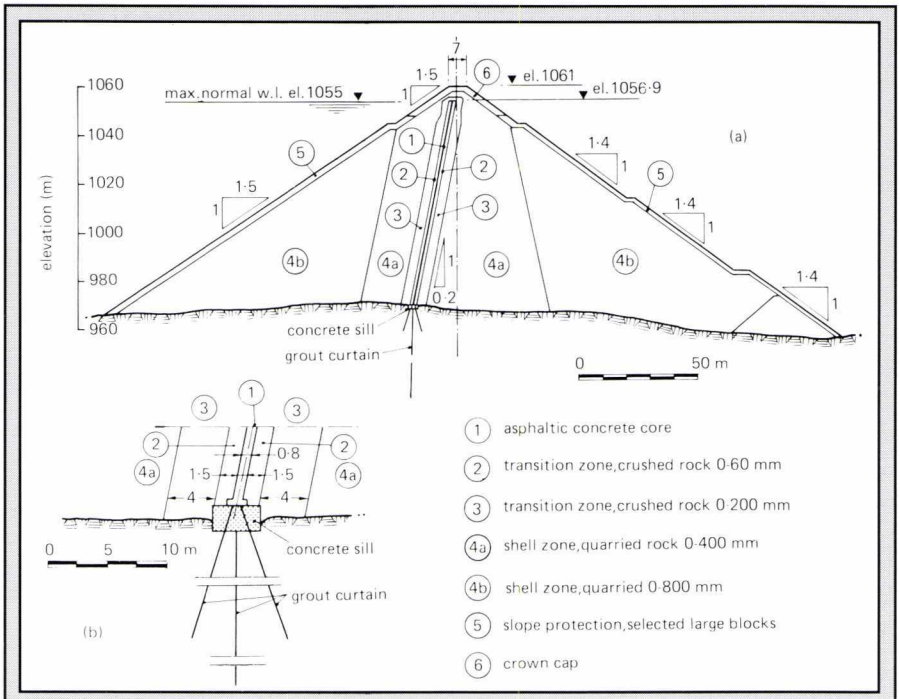


Fig. 1. (a) Cross section of Storvatn dam in Norway, under construction. The inclined core wall is 0.8 m wide at the base and 0.5 m at the top. (b) Detail of base of core. (Courtesy of Statkraft).

Research project to extend applicability of the method

An industry-sponsored research project was recently initiated to develop further the method of using asphaltic concrete core walls in earth- and rockfill dams for various geological, climatic and site conditions. The project began in October 1991 and is expected to finish in December 1992. The aims are to develop criteria for:

- the asphaltic concrete mix design (bitumen content and type, filler, aggregates and potential additives);
- the quality of aggregates which may be accepted for use;
- methods and procedures for laboratory testing of the asphaltic concrete;
- required design analyses for different situations;
- acceptable strain levels in the asphaltic concrete (without causing increased permeability or cracking);
- in-situ control of core compaction by non-destructive test methods;
- specification of construction procedures; and,
- quality management and control.

The thin asphaltic concrete core has to follow and adjust to the movements and deformations imposed by the embankment as a whole. These deformations must be accommodated by the asphaltic concrete without cracking or significant shear dilation (that is, volume expansion) which may lead to increased permeability. As well as the core material being flexible and ductile, it must have sufficient strength to resist the stresses in a high dam without detrimental effects. If the deformation modulus of the asphaltic concrete is similar to the one in the adjacent transition zones, little or no arching (positive or negative) will develop.

Interpretation of field data and design analyses

During the first phase of the research project, field and laboratory data were studied from the best instrumented and monitored dams with asphaltic concrete core walls, such as Finstertal (Austria), Grosse Dhünn (Germany), Storvatn (Norway) and Megget (Scotland). Special attention was paid to strains imposed on the core as a function of dam design, foundation conditions, core inclination, the types of shoulder and transition materials, and compaction procedures.

For the first three of the dams mentioned above, special instrumentation systems were installed to measure changes in the core width during dam construction and operation. Interesting information is therefore now available on the changes in core width over the height of the dam, and the comparison of results sheds light on the core and embankment behaviour, which is different for the three dams.

Field data from the recently completed Norwegian dams (Table I) are being updated and supplemented to assist in the further development of design and construction criteria. The relative merits of vertical versus inclined cores are being specifically studied in connection with the Storglomvatn dam (Table I), and international experience is being reviewed and evaluated.

Laboratory testing

A special laboratory device, called a "plate permeameter", has been developed and built to study core permeability as a function of imposed shear and extensional strains. The test is designed to simulate the flexural deformations which a core wall undergoes during reservoir filling and lowering, and subsequent creep in the embankment. Furthermore, it may simulate the extra flexure and shear caused by any

local imperfections in the core support from the adjacent transition zone.

The apparatus tests an asphaltic concrete circular plate, supported at the edges inside a chamber. While water permeability across the plate is measured, the plate is exposed to bending and shear until cracks occur. These special tests are carried out in addition to more conventional triaxial tests, to establish criteria for acceptable strain levels, beneath which the asphaltic concrete remains virtually impervious and does not crack. Different mix designs are used with various types and contents of bitumen, filler and aggregates, to determine permeability and stress-strain properties as a function of strain level.

The effects of using "lower quality" aggregate material is of special interest. In present practice, criteria for acceptable aggregates stem mainly from the design of road pavements, which exist under very different conditions from those for a dam core. If it can be documented that lower quality aggregate materials may be used, even in embankments with high stresses, the asphaltic concrete core method may also prove competitive in geographic regions where no high quality aggregate materials exist.

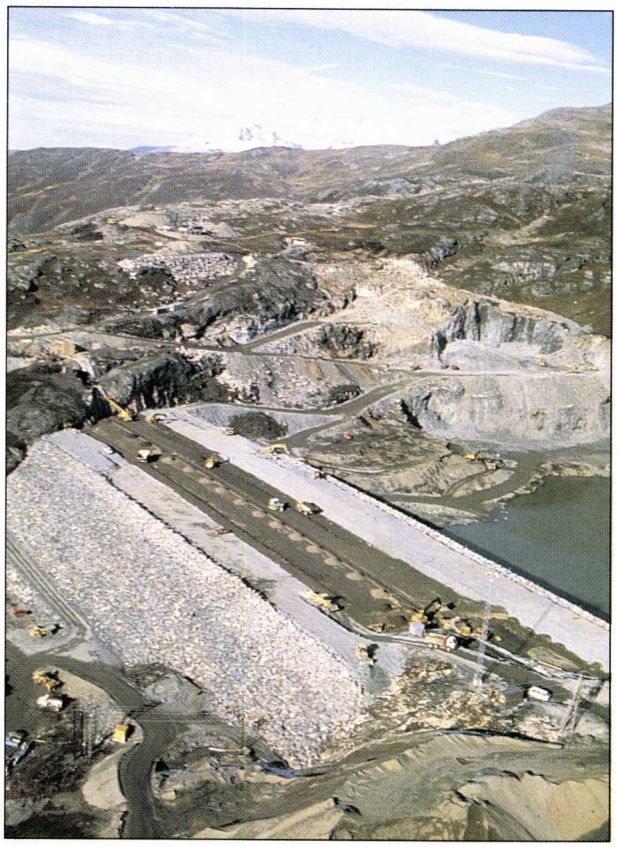
Current practice advocates the use of a grain-size composition of the aggregates which corresponds to the Fuller-Thompson curve. The bitumen content should be just sufficient to saturate the voids at maximum packing (density) of the aggregates with filler. This will be in the vicinity of 6 per cent bitumen by weight, and may be increased by 0.2 to 0.5 per cent to ensure workability and flexibility. It gives a mix which is easy to compact to a porosity below 3 per cent, and is virtually impervious.

The type and content of bitumen have effects on the ability of the asphaltic concrete to "self-heal", should cracking occur. The self-healing ability is of special interest in seismic regions with large earthquakes, and for situations with dams not resting on rock but on soil foundations. Experience shows that cracking in dams may develop because of intense ground shaking or differential foundation settlements, which introduce distortions in the dam body. A somewhat higher bitumen content than is used in current practice, and the use of softer bitumen, may further improve the self-healing ability without compromising any of the other desirable properties of the asphaltic concrete core. Furthermore, the use of softer grade bitumen will reduce the required mixing and placing temperatures, thereby reducing the costs. This aspect is to be studied.

Field control of compacted asphaltic concrete

Special field and laboratory experiments are planned to calibrate the use of non-destructive tests for in-situ control of asphaltic concrete porosity after compaction. The aim is to reduce the number of drill core samples required for adequate construction control. However, improvements in these non-destructive techniques and methods are necessary to increase current reliability for use in field control practice. A combination of coring and non-destructive testing will improve the continuity of construction operation as well as quality control. □

An asphaltic concrete core under construction.



Berdalsvatn dam with a vertical asphaltic concrete core 0.5 m wide, under construction in the Jotunheimen mountains, Norway. (Courtesy of A/S Veidekke).

Acknowledgment

The research programme is sponsored by the private Norwegian contractor A/S Veidekke, the state owned Statkraft (Norwegian Energy Corporation) and the Royal Norwegian Council for Scientific and Industrial Research. This support is gratefully acknowledged. The author thanks the sponsor representatives Svein Huse and Helge Saxegaard (Veidekke) and Leif E. Karlsen (Statkraft) for their comments and contributions to the manuscript.

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The Storvatn Dam. A Rockfill Dam with a Central Core of Asphaltic Concrete

By
Asbjørn Arnevik, Bjørn Kjærnsli, and Sven Walbø

SUMMARY

The considerations evaluated to elect the type of impervious element in the Storvatn Dam are described as well as the design of the Storvatn Dam as a rock fill dam with a central core of asphaltic concrete. Furthermore, the construction and control of the asphaltic work are dealt with. The performance of the dam concerning leakage and deformation after partly filling of the reservoir is given for one of three monitored cross sections.

RÉSUMÉ

Les considérations sur le choix du type d'élément imperméable pour le barrage de Storvatn sont décrites, de même que le dimensionnement du barrage en enrochement avec noyau central en béton bitumineux. De plus, l'article traite de la construction et du contrôle du noyau bitumeneux. Le comportement du barrage relatif aux infiltrations et aux déformations durant le remplissage partiel du réservoir est décrit pour l'une des trois sections transversales observées.

INTRODUCTION

Storvatn Dam and 3 other large and 9 small dams of various types form the Blåsjø Reservoir which is part of the Ulla-Førre scheme, owned by the State Power Board of Norway. The location of this scheme is in South West Norway, about 60 km east of the city of Stavanger, in an area of fjords, rivers, brooks, lakes, and mountains. Two power stations, two pumped power stations and one pump station account for a net production of approximately 4500 GWh in an average year. The Blåsjø Reservoir, with a capacity of 3.1×10^9 m^3, is situated on a mountain plateau and the maximum storage level is 1055.0 m a.s.l. The Storvatn Dam will be completed in 1987.

DESIGN

In the late sixties Storvatn Dam as well as the other large embankment dams around Blåsjø reservoir was designed as a rockfill dam with a central core of moraine, which is a traditional Norwegian rockfill dam. The volume of the dam was 10×10^6 m^3 with a maximum height of 90 m.

Rockfill could be quarried at the site, the hauling distance of morainic material and filter material was however 42 km and 28 km respectively, which was greater than for the other dams. The large cost of transportation including extra cost to build mountain roads to allow the heavy transport called for an alternative design. This request was also supported by the fact that the volume of borrow material was limited. If Storvatn therefore could leave the moraine to be used in the other dams their thin cores could be widened and thereby made safer.

Rockfill dams with the following impervious elements were accordingly examined:

- Facing of concrete or asphaltic concrete.
- Central core of asphaltic concrete or crushed rock.

To decide upon these different alternatives the following points of consideration were evaluated:

- Construction cost.
- Value of water stored during construction.
- Sensitivity to severe weather condition during construction.
- Behaviour of earlier completed dams.

The regulation at Blåsjø amounts to 300%, i.e. it would take three years of average precipitation to fill the reservoir, and the financial value of water which could be stored during construction would be of upmost importance. A rockfill dam with an upstream facing cannot store water during construction to the same degree as a dam with a central core and an estimate of cost and benefit favoured clearly the latter type. The final choice was therefore whether the central core should consist of asphaltic concrete or crushed rock.

The rock at the site consists mainly of granitic gneiss and test results showed that quarried rock could be crushed down to a grain size distribution similar to moraine. The crushed rock was sufficiently impervious and behaved in laboratory testing very much like a

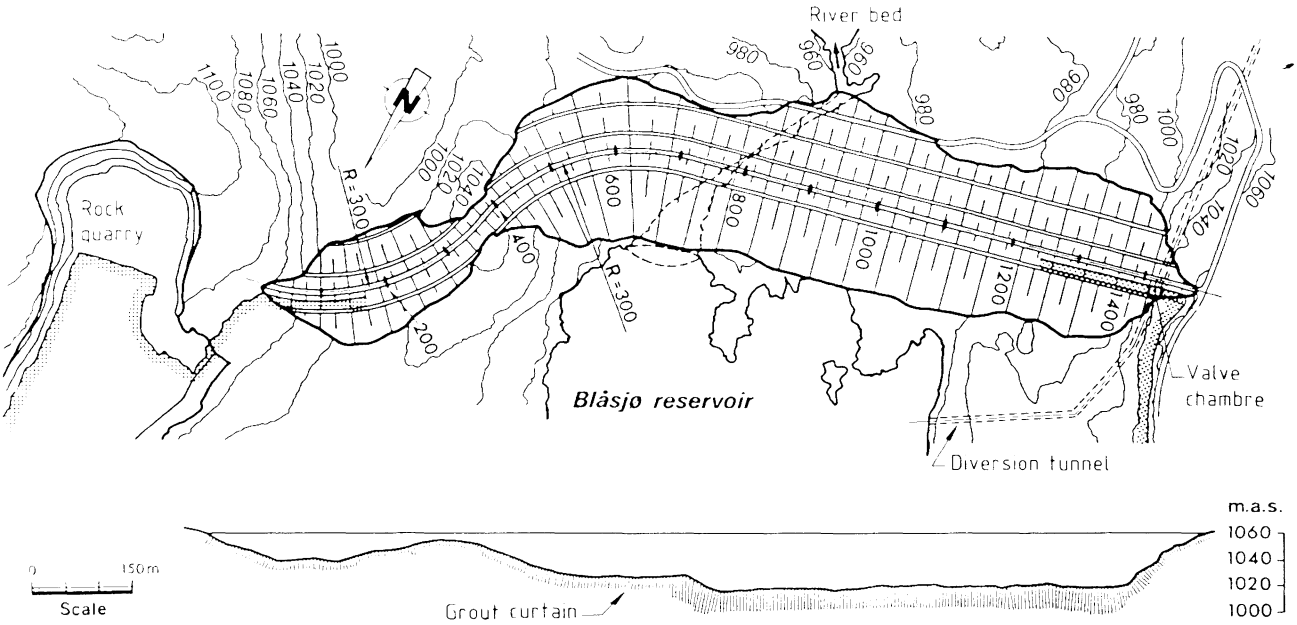

Fig. 1. Plan and longitudinal section.

coarse grained moraine. It could therefore be considered as suitable core material. It was also shown that crushed rock would be suitable as aggregates in asphaltic concrete.

Cost estimate showed that at equal cost the maximum gradient through the core of crushed rock would be 3 compared to the chosen maximum gradient through the asphaltic core equal to 100. The crushing of 0.75×10^6 m³ of rock down to the grain size of a moraine had no predecessor in Norway whereas the production of 50,000 m³ of asphaltic concrete over several years could be handled with well known equipment. Furthermore it was thought that the construction of the core of asphaltic concrete was least sensitive to bad weather.

In the late seventies the choice was made. The Storvatn dam should be a rockfill dam with a central core of asphaltic concrete. As the overall decision was made, the further design involved location of the dam axis, foundation and cross section of the dam.

Location and alignment of the dam axis

Generally the dam axis as a first choice is located in such a way that the volume of the dam will be a minimum. If no extra volume is required the dam axis will be curved convex to the reservoir.

At the site of Storvatn the axis giving the minimum volume is partly straight and partly curved concave and convex to the reservoir, Fig. 1. A straight axis giving a minimal volume had to cross the natural lake. The height of the dam would be approximately 10 m higher, than for the first mentioned axis and the corresponding volume about 10^6 m³ larger. For a dam curved convex to the reservoir the difference would be even larger. Avoiding the concave curvature of the dam required an extra cost of approximately 10%.

The question was therefore what harm would a concave curvature do to the dam? It is well known that an internal core in a rockfill dam is pushed downstream during the first complete filling of the reservoir. This displacement would create tensile strain in the core of concave curvature and could, depending on the extent of strain, lead to cracks through the core. Final element analysis showed however that the tensile strain to be expected in the concave curvature would be less than at the steep right abutment. The alignment of the dam as shown on Fig. 1 was decided upon.

Foundation

It was deemed obvious that the asphaltic core should be founded on a concrete structure in a rock trench. Furthermore, this structure should be anchored into rock and rock grouting should be performed. A main question was, however, whether this concrete structure could be a simple concrete sill or should be a concrete gallery.

The extra cost of erecting a gallery was estimated to exceed 10% of the total cost of the dam. Erecting a gallery would possibly require a prolongation of time of construction by one year. Would the advantages of a gallery be worth the extra construction and time cost?

Generally it is accepted that a concrete gallery may serve as a grouting gallery and as an inspection gallery.

The advantage of carrying out the primary construction grouting from a gallery was looked upon as minor or nil. The advantage of a grouting gallery would therefore depend on the need of supplementary grouting during the lifetime of the dam. Based on the result of ten exploratory diamond drilled holes to a depth of more than 50 m the likelihood that the foreseen grout work would need future repair was looked upon as very little. In any case a possible future increase in perme-

ability of the rock foundation was evaluated to be of little economic significance and could be reduced by grouting from tunnels which eventually would have to be excavated under the dam.

The grouting work carried out involved 2080 boreholes making up 32,140 metres and a grout take of 180 tonnes. The cost of this work corresponds to approximately 15% of the cost of a gallery in excess of a concrete sill. Where the rock foundation therefore consists of good rock the cost of a grouting gallery seems to be an unreasonable high insurance premium.

An inspection gallery would enable the location of leakage through the core. With less accuracy such possible leakage could be located in a much cheaper way. It was furthermore considered that such leakage anyway had to be reduced by grouting of holes drilled from the top of the dam upstream of the core.

On the advice of the consulting engineer the owner decided to omit the concrete gallery and let the asphaltic core rest on a concrete sill. This sill was to be cast in a rock trench, reinforced and anchored. The width of the sill varied between 4 and 5 m. The minimum depth of the trench was set at 0.5 m, the minimum thickness of the sill to 0.75 m, and the sill should not protrude more than 1.5 m above the adjacent rock surface.

Omitting the gallery made it necessary to collect and measure the seepage downstream of the core where it may be located within eight separate lengths of the dam.

Cross section

The cross section of the dam was designed with the aim to minimize the displacement and deformation within practical and economical margins. With this aim the stability of the structure along potential slip surfaces is of no concern.

The asphaltic core as previously mentioned is founded on a concrete sill. The core wall is inclined, producing a favourable transfer of the water load to the rock foundation. Even the top of the core wall is situated upstream of the centre line of the dam, leaving a relative large proportion of the rock fill as a support for the water load, Fig. 2.

The thickness of the core wall varies in steps of 0.1 m from 0.8 m to 0.5 m and at no level is it less than 1% of the water head at each level. The core wall rests on a cushion of asphaltic concrete placed on top of the concrete sill, height and width 0.4 m and 1.5 m respectively. The interface between the asphaltic cushion and the concrete sill is cleaned by sand blasting, primed and coated with asphaltic mastic.

The asphaltic concrete in the core wall is specified to be placed in layers of 0.2 m thickness. Each layer is displaced 4 cm in relation to the foregoing layer to obtain the prescribed inclination of the core wall. The thickness of the wall is defined as the width of the interface between each layer. Adjacent to the core wall a zone (2) consisting of crushed rock 0–60 mm is placed in layers of 0.2 m, and compacted simultaneously with the asphaltic concrete by vibration.

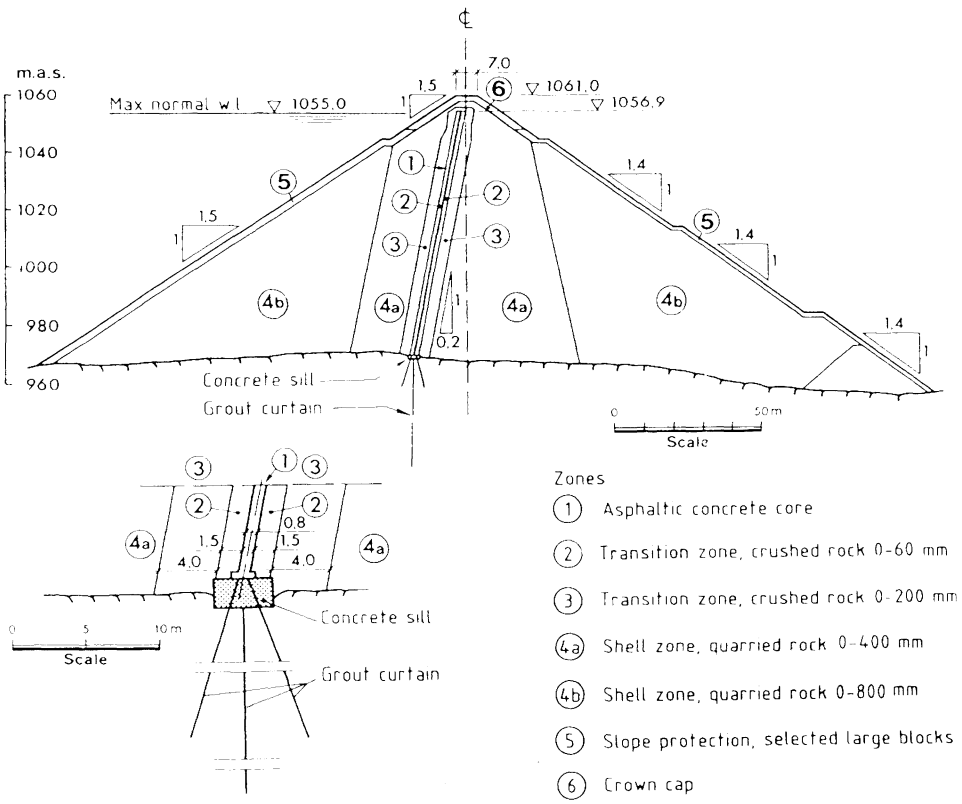

Fig. 2. Cross section.

Between this zone and the supporting fill of blasted rock a transition zone (3) of processed rock 0–200 mm is placed to secure a good filter. This zone is placed in layers of 0.4 m and compacted by vibratory rollers. To avoid local deformation of the asphaltic core a high degree of uniform quality of zones 2 and 3 was specified.

To reduce the deformation in general the central part of the supporting fill is thought to be decisive. More compaction effort is therefore specified for zone 4a than for 4b. The zone 4a is placed in layers of 0.8 m, sluiced and compacted by vibration whereas the rock in zone 4b is placed in layers of 1.6 m and compacted by vibration without sluicing. The slope protection upstream as well as downstream consist of blocks, the weight being approx. 1.5 tonnes, placed by back-hoe.

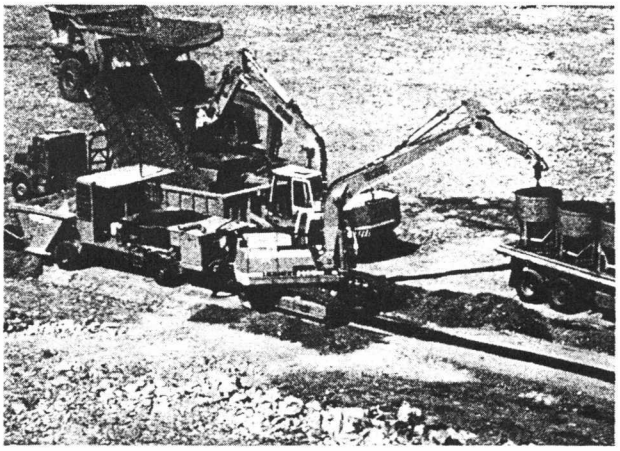

Fig. 4. The core under construction.

Mix design of asphaltic concrete

As part of the design the mix of asphaltic concrete must be specified. The main requirement is that the asphaltic concrete shall be practically watertight. Further more, the flexibility and workability of the mix shall be favourable.

As asphaltic concrete consists of stone aggregates and bitumen the mix must be specified as to:

- Quality and grain size distribution of aggregates
- Quality and content of bitumen
- Temperature at which the aggregates and bitumen shall be mixed and compacted
- Upper limit of allowable air void ratio (air porosity) of compacted asphaltic concrete to secure sufficient imperviousness of the core.

Based on laboratory prepared specimen the preliminary mix design was specified in the bid documents. In these documents it was however required that the contractor should carry out additional testing on asphaltic concrete produced by his plant at the site. The contractor had the right and obligation to suggest adjustment of the specified mix before starting the construction of the asphaltic concrete core.

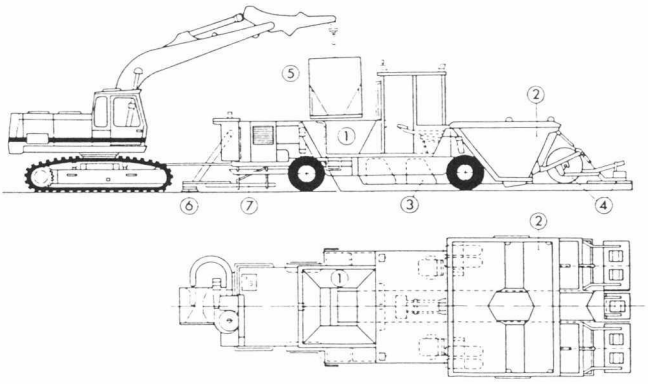

Fig. 3. Placing equipment. (1) Asphalt silo. (2) Silo for zone 2 material. (3) Asphalt precompaction. (4) Precompaction zone 2. (5) Asphalt transport container. (6) Vacuum cleaner. (7) Infrared heater.

CONSTRUCTION AND CONTROL OF THE ASPHALTIC CONCRETE CORE

Construction

The construction season at Storvatn is normally between mid-May and mid-October, due to the altitude and heavy precipitation in the area. Snowdrifts up to 15 m normally prevented the access roads to the dams in the area from being opened before the first part of May. In the summer, heavy rain was prevailing, and the annual precipitation in the area is between 2500 mm and 3000 mm. The Storvatn dam is quite large for Norwegian conditions, with a total volume of app. 10×10^6 m^3. The asphaltic core volume is app. 48 000 m^3, with an area of 79,000 m^2.

The asphalt plant for the production of the asphalt concrete for the core was erected downstream the eastern part of the dam, giving transport distances within the range of 1–3 km.

The zone 2 material was produced in a crushing plant located on the same site as the asphalt plant. The aggregates for the asphalt concrete were produced by the same plant.

To avoid cooling of the hot mix, the asphalt concrete was transported in insulated containers, each carrying approximately 4 tonnes, while the zone 2 materials were transported by ordinary trucks.

The materials were then loaded into the machine placing the core by two slightly modified excavators, which at the same time were pulling the core machine and an intermediate container for zone 2 material, respectively (Figs 3 and 4).

The core placing equipment was designed and constructed especially for this job, and some special considerations had to be taken, because:

- construction had to take place almost independent of the weather conditions,
- very small tolerances were allowed regarding deviation from the centre line, as the dam core was sloping 5:1,

- there was a demand that the surface of the previous layer could be inspected, if so wished, immediately before the next layer was placed,
- high initial compaction of the asphaltic was desired to reduce the core/zone 2 interaction.

To reduce the need for manual labour many of the functions were automated.

Monitoring sideways was performed by laser beams on the straight section of the dam. The receiver is connected directly and rigidly to the front wheels of the paver. In the curved part of the dam sideways monitoring was steered by metal ribbon glued to the surface. Constant height level was maintained due to the use of a plane laser, and the receiver for level control is mounted on top of the machine, to avoid disturbances from other construction machinery on site.

To reduce the risk of failure due to unattended operation the control panel had light and audio signals for «out-of-line».

Hand placing was normally carried out in connection with the two first layers along the concrete sill, which have an extended width, and at each end of a layer.

The construction took place with one placing unit during the first two construction seasons, but as the crest length increased, a second machine was brought into use as well. As seen from Fig. 1, the dam was constructed as two separate dams during the third season and part of the fourth season.

Each layer was placed in lifts of 20 cm. The actual production rate had to be adapted to the placing of the embankment and varied between 1 and 3 layers per day.

The placing equipment was fitted with a two-stage vibrating screed, for initial compaction of the asphalt concrete, while the zone 2 material was pre-compacted by a static roller and vibratory plate, connected to the back part of the machine. Both zones were then compacted by vibratory rollers to the specified density (Fig. 5).

Quality assurance and quality control

When using asphalt concrete as the impervious element, quality assurance is most important. The core,

Fig. 5. Final compaction.

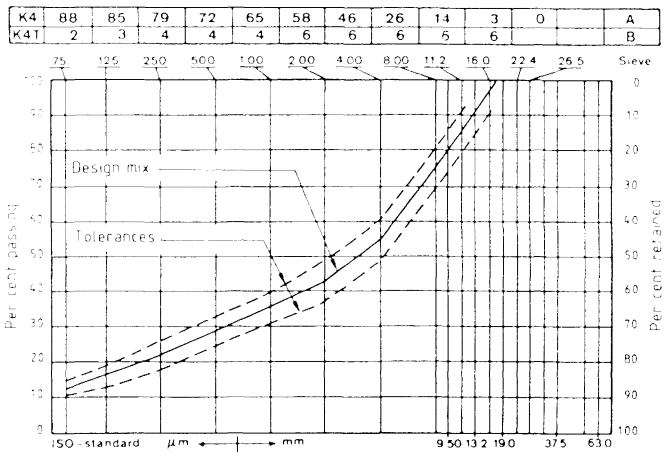

Fig. 6. Designed grain size distribution.

being relatively thin, is hidden shortly after being placed. Further control is made almost impossible, and therefore it is necessary to perform a satisfactory quality control in every stage of the production procedure, from monitoring the mixing plant and placing operations to sampling of raw materials as well as the finished core.

To ensure a good quality in the mixing process, a computer controlled asphalt plant was installed. Reports from the computer were displayed on the operator's screen, and compared with the designed grain size distribution (Fig. 6). The most useful and widely used display for monitoring the mix quality was the "Charge report". A full report was printed at optional intervals, and a short form report was printed every time a complete batch was emptied into the mixing chamber. The report provided the set point and the actual batchweight of each material in the mix. The deviation from the setpoints could then be assessed and proper action immediately taken, if necessary.

This also gave the opportunity of controlling the mean and standard deviation of the production, both daily and weekly.

The current production capacity and fuel oil consumption were also shown and so were the temperatures:

a) in the drum outlet
b) in the aggregates scales and
c) in the stack pipe.

Binder content was also shown in percentage.

The computer alerted the operator if the proportion of any material deviated outside preset limits. Such alarms intercepted the production and required a reaction from the operator who either rejected the batch or corrected it before returning control to the computer.

Mechanical properties of aggregates were determined daily.

Attention was continuously given to the shape and strength of the aggregates. These properties were in

some measure dependent on the current settings of the crushing plant. Strongly abrasive rock material led to frequent adjustment of crusher settings. Primarily, however, the material strength is given by mineralogy and crystalline structure. To prevent weak materials from being used in the construction, aggregates produced within a certain period of time were always put aside in a separate deposit until satisfactory tests results were confirmed. If not, such material was rejected and used for other purposes.

Samples from the production plant were examined by extraction analyses twice a day. The results normally confirmed the computer's Charge Report. In fact, statistics indicate that the plant computer controlled the bitumen content with a higher accuracy than the tests performed in the laboratory could measure (Table 1).

Wear and damage of the screens will only be detected through laboratory investigation, however. These factors will gradually or more abruptly cause an altered situation in the mixing plant, and will normally be detected by the operators of the plant, but not always. Thus the laboratory tests function as a third safety check and brings in a vital element of know-how in the Quality Assurance.

From the mix samples Marshall specimens were made and the density determined. Based on specific density measurements of each material fraction in the mix the content of air voids was calculated. Then the first indication on how the mix would perform during compaction was obtained.

Permeability measurements were also in the test programme. The results, however, confirmed that the asphaltic concrete with the required density is watertight, and the test was therefore omitted when air voids tests were taken.

The temperature is an important feature in the handling and control of hot mix, because of alteration of binder characteristics. In addition to the temperature of the drum outlet, which reflects the heat on the aggregate surface, and in the sieves where the heat is distributed more evenly through the aggregates, measuring continued during loading, transport, placing and compaction.

Table 1. Weekly standard deviation of mix parametres (per cent).

Bitumen	Filler	2 mm	4 mm	8 mm	Void content
0.0587	0.0089	0.184	0.213	0.140	0.35
0.0622	0.103	0.233	0.270	0.146	0.37
0.0380	0.133	0.195	0.286	0.201	0.37
0.0611	0.122	0.409	0.576	0.262	0.40
0.0544	0.095	0.300	0.254	0.151	0.35
0.0500	0.086	0.220	0.141	0.123	0.36
0.0843	0.102	0.392	0.263	0.177	0.43
0.0585	0.068	0.220	0.221	0.226	0.36
0.0692	0.122	0.454	0.619	0.266	0.38
0.0597	0.165	0.409	0.326	0.203	0.33
0.0717	0.132	0.443	0.492	0.284	0.39

Table 2. Control programme.

Testing of aggregates	Testing of bitumen
- Grading of fractions (stone, sand and filler) - Gradation curve for mix - Filler properties by Ridgen test - Mechanical properties	- Penetration - Kinematic viscosity 60 c, 135 C - Ring and Ball (optional) - Content of insoluble material - Content of volatiles - Change in properties after heating - Density
Testing of asphalt mix	**Testing of placed core**
- Bitumen content - Gradation curve - Marshall compactability - Void content - Permeability on Marshall specimen	- Temperature - Geometry - Permeability (on drilled cores) - Void content - Visual control - Compaction control (non-destructive)

A complete list of test carried out is shown in Table 2.

Surface control before applying a new layer

Water and dust are normally appearing on the surface. Water is usually present as a result of rainfall but may also appear in connection with sluicing of the inner rockfill zone. Visual inspection for the appearance of water presence was carried out and any water present removed by a vacuum cleaner at the front of the placing machine. Any remaining moisture was taken care of by the infra-red heater.

Compaction control

With construction of an asphaltic concrete core in a rockfill dam, compaction control is of supreme importance. The compactibility of the mix was determined daily. The results had to be presented quickly in order to stop the work if some irregularity should occur. For every fifth layer and every 200 m section of the dam, cores of 100 mm diameter and 40 cm depth were taken. Additional cores were drilled on spots where visual control gave reason for suspicion. All cores were subject to tests for void content.

The core length of 40 cm made it possible to examine the joint between two layers. Joints in general seem to be as tight as the layer itself as long as careful inspection of the previous layer takes care of unclean spots. All inspected layers have proved to be properly bonded to each other.

Non-destructive test methods have been tried in addition to the ordinary control programme. The density has been determined by means of a nuclear frequency counter. This equipment provides the void content within ten minutes after compaction. However, these results have shown to be unreliable due to the influence of the heat in the asphalt layer.

PERFORMANCE

At the end of the 1986 construction season the fill reached the elevation of 1059 m.a.s.l. which is two metres below the designed crest level and two metres above the top of the core. The highest reservoir level so far reached is 1042 m.a.s.l. which is 67 m above natural level and 13 m below full storage level.

Leakage

The leakage water is collected between the core foundation and walls erected 10–20 m further downstream. Within this area the leakage is registered automatically and remotely for three sections along the dam, A, B, C. The leakage may be registered manually for eight separate sections.

At the highest storage level so far reached, the total leakage is registered at approx. 10 l/s. There is however reason to believe that some of the registered leakage-water does not originate from the reservoir. A cross valley ground water flow was observed within section B before raising the waterlevel and the measurements show that the leakage decreased by up to 2 l/s during the cold winter months despite of the constant reservoir level. Several months with temperature well below freezing will most certainly reduce a possible ground water flow.

Leakage registered in April after winter relatively constant reservoir level is tabulated below:

Table 3. Leakage registered in spring before snow-melting and a new construction season.

Date	Storage Level m.a.s.	Registered leakage l/s			
		Section			Total
		A 20-350	B 350-1155	C 1155-1450	
850425	1004.7		6.7	0.2	6.9
860406	1025.2	0.2	8.0	1.5	9.7
870412	1041.1	0.1	7.8	2.2	10.1

The registered leakage within section B is not consistent with the reservoir level and support the assumption that part of the collected water does not originate from the reservoir. The inconsistencies within section A are thought to be caused by the inaccuracy of the remote readings.

Despite uncertainty of the size of the real leakage it is satisfying that the leakage only increased by 3 l/s with an increase in the storage level of 36 m.

Deformations

Inclinometer tubes are installed at three cross sections. The result of measurement in section 940 shall be described in the following. For this section three horizontal, two vertical and one inclined tubes are installed, Fig. 7.

The displacements measured in these tubes at the end of the construction season of 1986 are shown on the same figure. At the time of measurement the top of the fill has reached the elevation of 1059 m.a.s.

In general the displacement measured are considered to be small. The maximum displacement of the asphaltic concrete, as measured in the inclined tubes immediately downstream of the core, is about 0.21 m, horizontally 0.12 m and vertically 0.18 m.

The measurement in the horizontal tube at elevation 1016 shows a maximum vertical displacement of 0.34 m and horizontally 0.12 m in the downstream direction.

The shape of the deflection curve of the horizontal tubes at elevation 986 and 1016 show clearly that the central part of the rockfill is far less compressible than the outer part. The central part, Zone 4a, was placed in layers of 0.8 m, sluiced and vibrated whereas the outer part 4b was placed in layers of 1.6 m and vibrated. If no extra effort had been put into zone 4a in relation to zone 4b the deflection of the core wall might have been about twice as much.

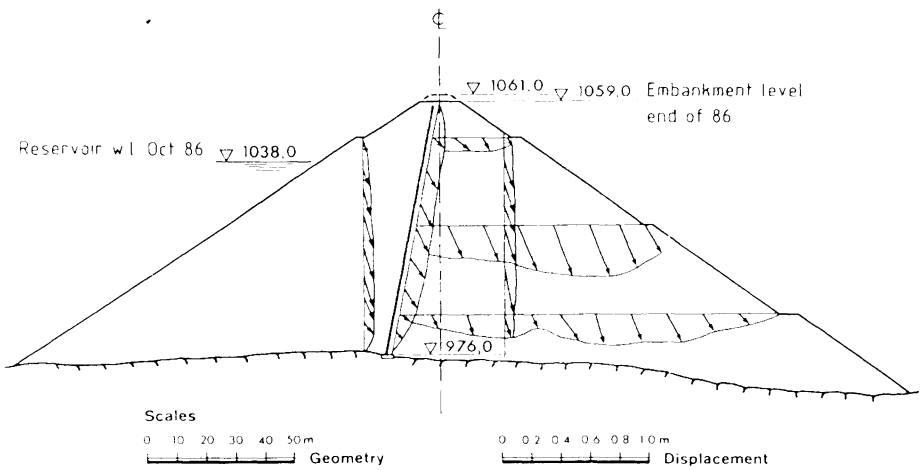

Fig. 7. Registered displacement in section 940, October 86.

Fig. 8. Season 1986. Construction level approximate 1048.0. Rock quarry in the background.

CONCLUSION

The decision to build the Storvatn dam as a rockfill with a central core of asphalt concrete was based on consideration described under chapter 2. The experiences gained during construction seem to prove that the assumptions made during design work are fulfilled. It can be stated that the construction of the asphaltic concrete core is less sensitive to severe weather conditions than the construction of a core of morainic materials. So far the behaviour of the dam is up to the expectation.

The experiences so far obtained has resulted in the construction of three other large dams of the same type of construction and the design of still more. The Storvatn type of dam is certainly a real alternative to the more traditional Norwegian rockfill type with a core of moraine.

Behaviour of Storvatn Dam, Norway – A Case of Prediction versus Performance

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SUMMARY

Storvatn Dam is a well instrumented, 90 m high rockfill structure with an asphaltic concrete core, located in Western Norway. A series of linear finite element analyses of its behaviour was carried out on three cross sections under plane strain conditions, simulating its true construction sequence and simultaneous filling of the reservoir. Material parameters were derived from large scale laboratory triaxial and oedometer tests and from field plate bearing tests. The mathematical model was calibrated by back analysing the early construction stages of the dam and comparing the calculated movements against field measurements. Subsequently the model was used to make predictions of the behaviour of the dam during filling of the reservoir up to full reservoir level. The analyses showed that the simple linear model may be used effectively to simulate the complex behaviour of the dam and to calculate the strains and movements within the dam.

1. INTRODUCTION

Located 80 km northeast of Stavanger in Western Norway and completed in 1987, Storvatn Dam is presently the largest asphaltic concrete core rockfill dam in the world. It has a crest length of 1460 m (comprising an 800 m long straight section and a 660 m long gentle S-curve), a maximum height of 90 m and a maximum base width of 470 m. Figure 1 illustrates the plan, the longitudinal section and the cross section of the dam. The asphaltic core is 800 mm wide at the base and 500 mm wide at the top, and is placed at a slope of 1 (Vertical) to 0.2 (Horizontal).

The dam is well instrumented at three cross sections with 357 instrument points comprising various types of inclinometers, extensometers, pressure cells, survey monuments and seepage weirs. Much of the instrumentation is provided to observe and monitor the movements of the asphaltic core.

2. CONSTRUCTION OF THE DAM

The construction of the Storvatn Dam was carried out conventionally except for the placement of the asphaltic concrete core and its adjacent filter zones. In general the rockfill for the various zones was dumped on to the advancing layer, spread by bulldozers or backhoes near the crest of the dam, and compacted with vibratory rollers in accordance with the schedule shown in Fig. 2.

The asphaltic concrete core and the two adjacent filter zones were placed in 200 mm layers simultaneously by a specially designed machine. This was followed by three 2-tonne selfpropelled vibratory rollers working in a predeterminated parallel formation to ensure uniform, balanced and simultaneous compaction of all three zones whilst the temperature of the hot asphaltic concrete was still between 180°C and 160°C. Six passes were required to achieve the stipulated compaction of the asphaltic concrete core (< 3 per cent air voids), consequently the same number of passes was used for the filter zones.

Great care was taken to ensure proper bonding between successive layers of asphaltic concrete. The contact surface was cleaned to remove free water and impurities by means of an industrial vacuum cleaner. The contact surface was further reheated by means of infrared heaters to facilitate proper adhesion between the layers.

Quality control of the rockfill was generally maintained by visual inspection. The main emphasis was placed on ensuring that rockfill after compaction complied with the appropriate grading envelope, layer thickness, compaction criteria and that particle segregation was avoided. Periodic tests were carried out to determine rockfill properties such as particle grading, density, compressibility and shear strength.

Due to the winter conditions prevailing in the mountains of Western Norway, the season for dam construction is limited to only 6 months and generally extends from May to October. For this reason, construction of Storvatn Dam embankment had to be carried out over 7 years (7 construction seasons) from 1981 to 1987. Reservoir filling began when the embankment reached EL. 1002.0 in 1984 season, and since then the reservoir level generally followed the construction progress, to reach EL. 1040.0 by October 1986. The 1986 season

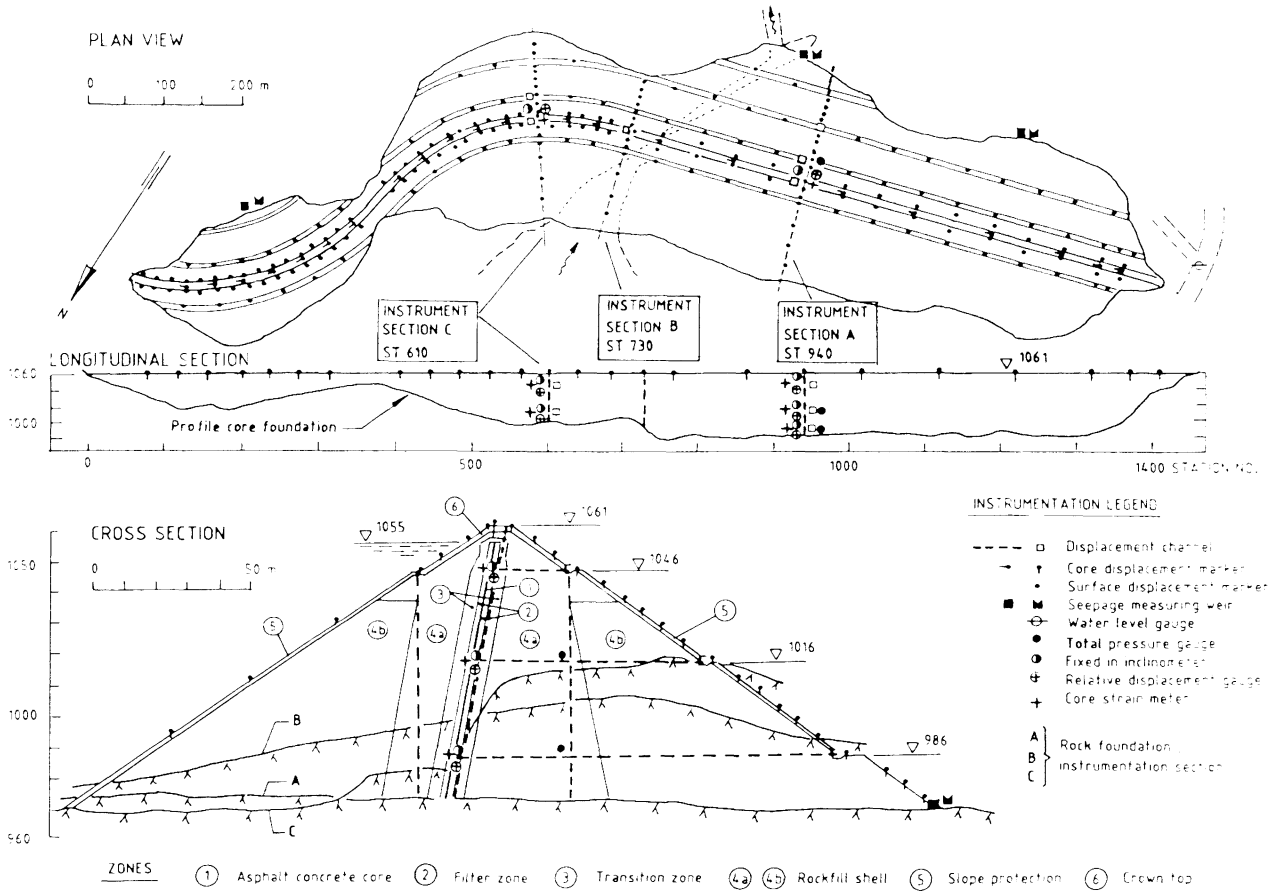

Fig. 1. Storvatn Dam – General arrangement and instrumentation layout.

effectively marks the end of embankment construction, though some 2.5 m of capping material had to be placed during 1987 season to complete the construction of the crest of the dam. For more details about the construction of the dam, see Arnevik et al. (1988).

3. INSTRUMENTATION

The object of the instrumentation programme set up for Storvatn Dam was to determine the behaviour of the embankment and specifically its asphaltic concrete core and adjacent supporting materials. It was considered important to establish if the asphaltic concrete core would be strained beyond 2–3 percent as laboratory tests had demonstrated that such strain levels could result in increased permeability of the core material. The straining of the core might be either vertical compressive or lateral expansive depending on the relative compressibility of the core and surrounding materials.

The instrumentation programme for Storvatn Dam consists of three instrumented cross sections comprising: 12 inclined, vertical or horizontal, inclinometer casings, 28 extensometers for strain measurements in the asphaltic concrete, 10 extensometers for detecting relative movements between the core and the filter zone, 10 special devices for detecting shear deformation in the filter, 10 pressure cells for measurement of stresses in the rockfill, and 3 automatic recording seepage weirs. Additionally, 284 survey monuments are located at regular intervals and at various levels along the embankment.

The locations of these instruments are shown in Fig. 1.

4. ANALYSIS PROCEDURE

Both linear and non-linear finite element analyses were performed. The procedure used for approximating material behaviour is by successive increments, in which the loading is divided into a number of small increments, within which the material behaviour is assumed to be linear. After each increment, the modulus values are re-evaluated in accordance with the stresses in the element, so that the non-linear relationship is approximated by a series of linear relationships.

The incremental stress-strain relationship for an isotropic material under plane strain conditions is expressed in the following form:

$$\begin{pmatrix} \Delta\sigma_x \\ \Delta\sigma_y \\ \Delta\tau_{xy} \end{pmatrix} = \frac{3B}{9B - E} \begin{pmatrix} (3B + E) & (3B - E) & 0 \\ (3B - E) & (3B + E) & 0 \\ 0 & 0 & E \end{pmatrix} \begin{pmatrix} \Delta\epsilon_x \\ \Delta\epsilon_y \\ \Delta\gamma_{xy} \end{pmatrix} \quad (1)$$

in which $\Delta\sigma$ and $\Delta\tau$ are stress increments, $\Delta\epsilon$ and $\Delta\gamma$ are strain increments, and E and B are Young's modulus and bulk modulus respectively.

A simple, stress independent linear material model was used for the determination of Young's modulus and bulk modulus from oedometer tests.

The expressions take the form:

$$E = \frac{M_{oed}\,(1 + 2K_o)\,(1 - K_o)}{(1 + K_o)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

$$B = \frac{M_{oed}\,(1 + 2K_o)}{3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

in which M_{oed} is the oedometric modulus and K_o is coefficient of earth pressure at rest.

The analysis procedure adopted in this investigation takes into account the effects of submerging the upstream rockfill zones during filling of the reservoir. The procedure used for calculating these effects is that described by Nobari and Duncan (1972). The forces due to filling of the reservoir are calculated for filling increments equal to construction layer thickness. These forces may be applied to the dam together with forces due to weight of the elements added during construction of upper layers of the dam, thereby simulating the reservoir filling simultaneously with construction. After the dam construction is completed, the reservoir filling is considered as a "load case after construction". The entire analysis is thus simulated in sequential loading form.

The new computer program developed at the Norwegian Geotechnical Institute (Adikari, 1987) was used for the analysis. The program (NAD-87) performs plane strain, linear and non-linear finite element analyses of zoned embankment dams, using the sequential loading method proposed by Clough and Woodward (1967). The effects of reservoir filling on stresses and movements in the embankment are accounted for in the finite element model. Stress-strain relationships are expressed in terms of Young's modulus and bulk modulus, enabling volume change characteristics of the material to be incorporated into the finite element analysis more easily.

ZONE	MATERIAL	Layer thickness m	Vibr.roller static weight in tons	Number of passes
1	Asphaltic concrete (crushed aggregates 0 – 16 mm)	0.2	> 2	6
2	Crushed rock (0-40 mm) Alt. gravel	0.2	> 1-2	6
3	Well graded rock material d < 200 mm	0.4	> 6-8	4
4a	Well graded rock material d < 400 mm	0.8	> 13	8 (sluized)
4b	Well graded rock material d < 800 mm	1.6	> 13	8
5	Selected large stones > 0.5 m³	Individually placed by backhoe		
6	Selected large stones > 1.0 m³	Individually placed by backhoe		
7	Coarse rockfill d_{90} > 900 mm, d_{10} > 100 mm	Dumped in 4 m lifts		

Fig. 2. Specification for placement fill materials.

5. LABORATORY AND FIELD TESTING OF CONSTRUCTION MATERIALS

Rockfill from Storvatn Dam has been tested both in the laboratory and in the field to obtain strength parameters and modulus values for stability and finite element analyses of the dam. Laboratory tests on rockfill are described in NGI (1984). Field tests comprising grading analyses, density tests and plate bearing tests are described in NGI (1986).

The laboratory test programme (conducted in 1984) consisted of fourteen triaxial tests and six oedometer tests. Five of the triaxial tests were carried out in NGI's large vacuum triaxial cell with a specimen diameter of 625 mm and a height of 1250 mm. Conventional triaxial equipment with a specimen diameter of 102 mm and a height of 200 mm was used for the remaining nine triaxial tests. The six oedometer tests were performed in a fixed ring oedometer with specimen diameter of 500 mm and height 250 mm.

6. DETERMINATION OF MATERIAL PARAMETERS

The derivation of material parameters for the finite element analysis described in this paper was carried out in two stages. In the first stage, the results of laboratory triaxial and oedometer tests were used directly in the finite element analyses, and the calculated movements were compared with measurements taken at the end of the 1986 construction season. The parameters were derived in accordance with the method described by Charles (1976).

In the second stage, the material parameters were modified in order to match the finite element model results more closely with the field measurements. The asphaltic concrete core material was found to have a comparatively low Young's modulus, but the measured strains at the end of construction were very low, implying that the bulk modulus (and the Poisson's ratio) was rather high. Based on field measurements, the rockfill in zone 4a was found to be considerably stiffer than the material in zone 4b. After making allowances for such observations, the final set of material parameters was chosen for the analyses. These values are shown in Table I.

Table I. Storvatn dam – Material parameters.

Parameter		Unit	Zone (Mat No.)								
			1	2		3		4a (4)		4b (5)	
				Moist	Sat.	Moist	Sat.	Moist	Sat.	Moist	Sat.
Unit weight	γ	kNm^{-3}	24.5	21.5	23.6	21.5	23.65	20.5	22.5	20.0	22.5
Strength parameters	c	kPa	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	φ	Deg	35.0	49.0	47.0	49.0	47.0	47.0	45.0	45.0	43.0
	$\Delta\varphi$	Deg	0.0	7.0	7.0	7.0	7.0	7.0	7.0	7.0	7.0
Coeff. E.P. at Rest	K_0	–	0.50	0.25	0.25	0.25	0.25	0.25	0.25	0.40	0.40
Porosity	n	–	0.02	0.21	0.21	0.215	0.215	0.22	0.22	0.25	0.25
Isotropic col. coeff.	β	$m^2kN^{-1}(\times 10^4)$	0.0	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Young's modulus	E	MPa	100	300	280	180	170	90	80	35	30
Bulk modulus	B	MPa	1700	165	150	100	90	50	40	27	20

Atmospheric Pressure (P_a) = 101.4 kPa

7. FINITE ELEMENT ANALYSIS OF STORVATN DAM

Both linear and non-linear stress strain relationships were used in the finite element analysis carried out for this project. In each case plane strain conditions were considered to be appropriate. Analyses were carried out for each of the three instrumented cross sections shown in Fig. 1. However, only the results obtained from the linear analysis for cross section A are presented in this paper. Complete details of the analyses performed on other sections are found in NGI (1987).

Filling of the reservoir simultaneously with the construction of the embankment is common practice in Norway. In the case of Storvatn Dam, filling of the reservoir began when the embankment level was at EL 1002.0. From then on the reservoir level followed embankment construction, as shown schematically in Fig. 3. The periods of construction shutdown have coincided with periods during which no or limited reservoir filling has taken place. For the purpose of this analysis, the reservoir level at EL 1040.0 is taken as the reservoir level at the end of embankment construction. Similarly, the reservoir level at EL 1055.0 is taken as the reservoir

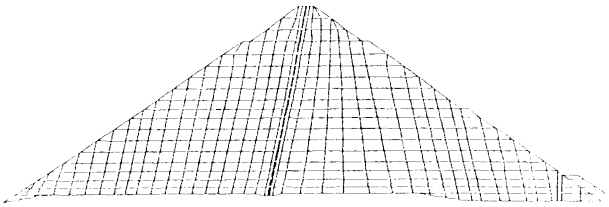

Fig. 4. Finite element mesh for Section A.

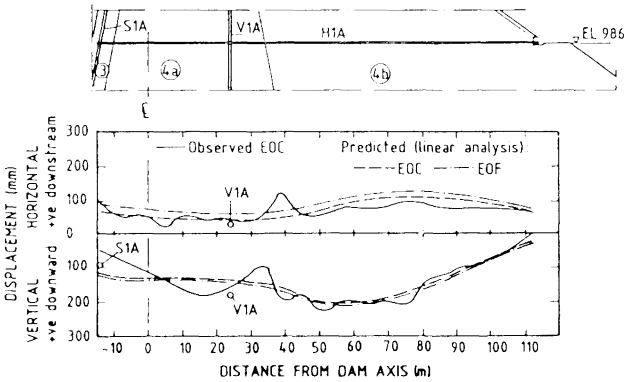

Fig. 5. Displacement profiles at horizontal channel H1A.

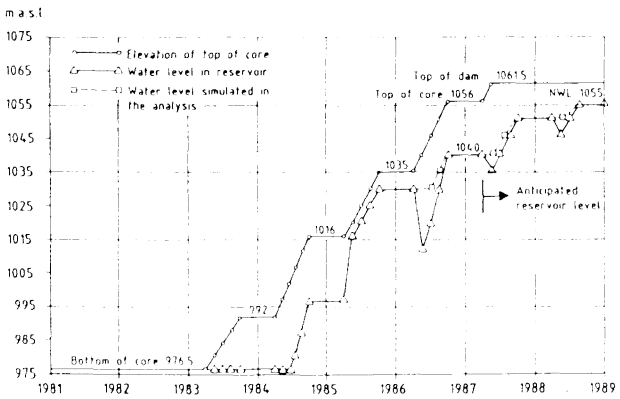

Fig. 3. Simulation of construction sequence and reservoir filling.

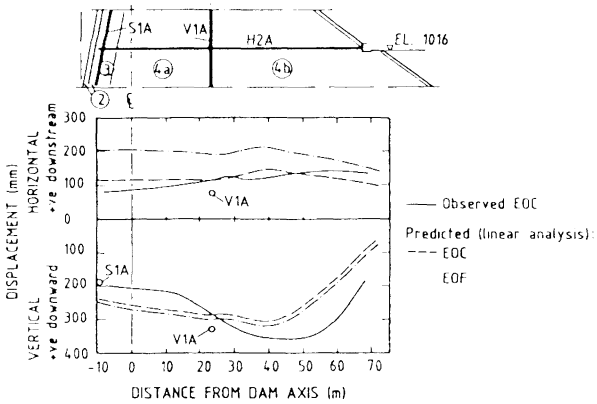

Fig. 6. Displacement profiles at horizontal channel H2A.

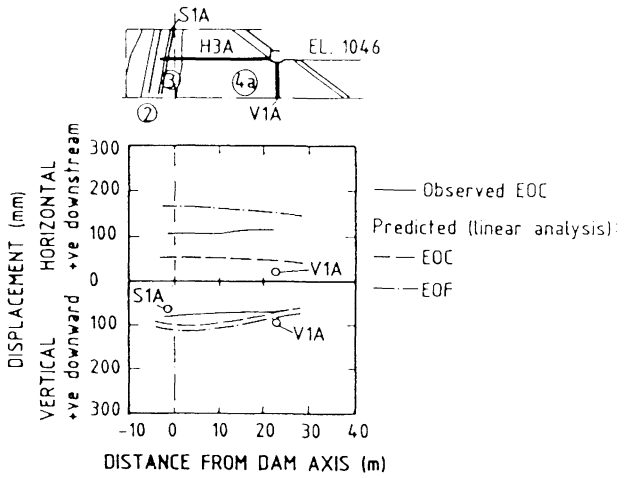

Fig. 7. Displacement profiles at horizontal channel H3A.

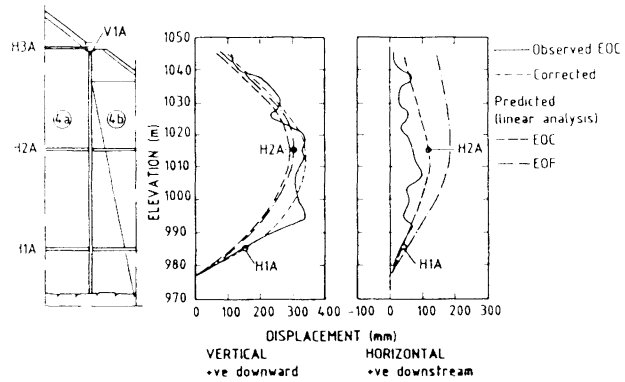

Fig. 8. Displacement profiles at vertical channel V1A.

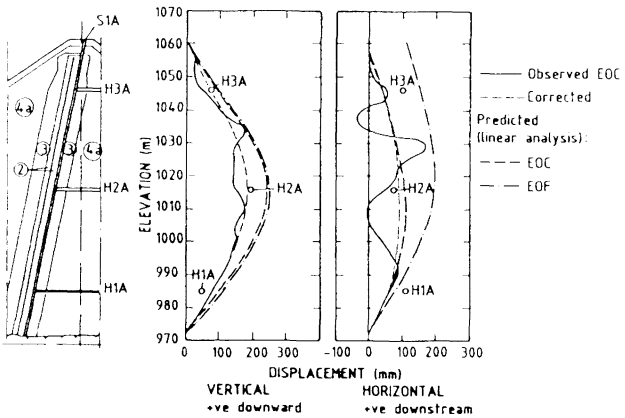

Fig. 9. Displacement profiles at vertical channel V2A.

level at the end of reservoir filling. The rise in reservoir level from EL 1040.0 to EL 1055.0 is simulated in two increments, and the resulting forces are applied to the completed structure in the form of "load cases after construction".

The finite element mesh used to model the cross section of the embankment (Fig. 4) has been formed of

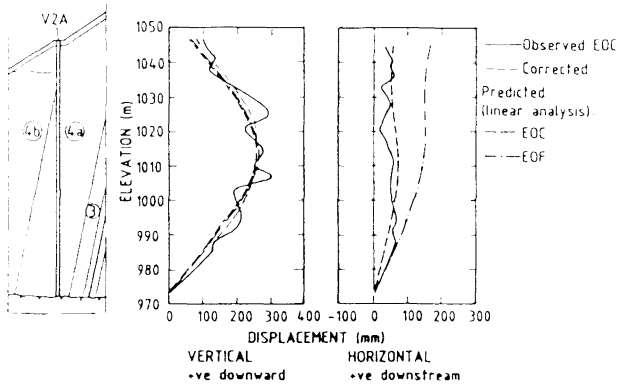

Fig. 10. Displacement profiles at inclined channel S1A.

four-node, linear isoparametric quadrilateral elements, as described by Wilson et al. (1971).

Where triangular elements are used, the 3rd and 4th nodes of the quadrilateral are merged together, so that they become special cases of the quadrilateral.

The mesh has been built up in 20 layers to simulate the construction in 20 lifts, and the nodal points have been chosen to coincide with zonal boundaries, and as far as practicable, with instrument locations. The foundation was not modelled because the bedrock was considered to be effectively rigid in comparison with the rockfill and as such was not expected to influence displacements within the embankment.

7.1 *Vertical and Horizontal Displacements*

Profiles of measured and calculated displacements are plotted for the six movement measuring channels installed in Section A, as shown in Figs 5 to 10. For simplicity of presentation the same scales are used in all plots. The displacement scale is exaggerated to 100 times the horizontal distance scale. Field measurements taken on 9th October 1986 are used in all plots and are taken as the measurements for the end of construction loading stage. Where available, measurements from other channels that have common observation points, are specifically shown in these profiles.

The reasonable agreement between the finite element model calculations and the field observations for the end of construction loading stage, achieved by parameter modification, forms the basis for the predictions made for the end of reservoir filling loading stage. Provided the reservoir filling takes place as shown schematically in Fig. 3, the end of filling loading stage for the dam should be reached by 1989.

The displacements measured in Storvatn Dam are smaller than those originally estimated. However, they are of the same magnitude as those measured in Finstertal Dam, an Austrian dam in many aspects similar to Storvatn Dam (Schwab, 1984). At Storvatn Dam the maximum observed settlement in horizontal channel H2A is 350 mm, measured approximately 40 m downstream of the dam axis. The depth of rockfill and the

depth of overburden at this point are approximately 40 m and 20 m respectively. The vertical strain in the rockfill in this region amounts to approximately 1 per cent. The maximum horizontal displacement is also measured in horizontal channel H2A near the same location where the maximum settlement is measured (50 m downstream of the dam axis, and is 135 mm.

In order to present a comprehensive picture of the displacements predicted by the finite element analysis, contours of settlements and horizontal movements for the end of reservoir filling loading stage have been prepared. These contour plots are shown in Figs 11 and 12. It can be seen that the displacements are assymmetrical about the dam axis.

The maximum predicted settlements occur at mid height with the values in the rockfill zones upstream of the asphaltic core being somewhat larger than for the downstream zones. The values of predicted horizontal displacement on the downstream side of the core are larger than those on the upstream, with maximum values occurring in the mid region of the downstream filter zone. The presence of the water load on the upstream side of the asphaltic concrete core is considered to be the primary reason for this behaviour.

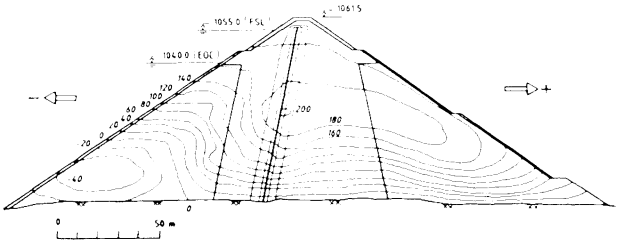

Fig. 12. Predicted horizontal displacement in mm at end of reservoir filling.

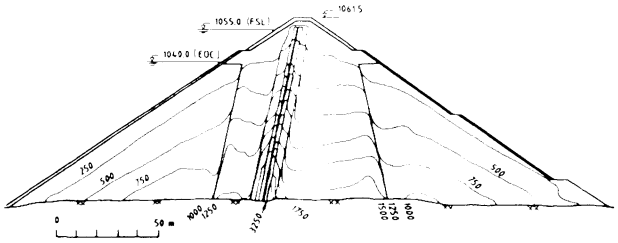

Fig. 13. Predicted major principal stress in kPa at end of reservoir filling.

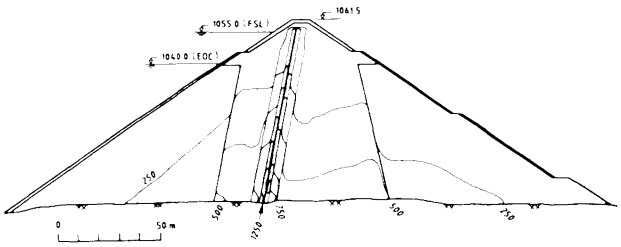

Fig. 14. Predicted maximum shear stress in kPa at end of reservoir filling.

7.2 *Normal and Shear Stresses*

Contours of predicted major principal stresses for the end of reservoir filling loading condition are presented in Fig. 13. The contours tend to be sub-parallel to the side slopes in the 4b rockfill zone. High stress concentrations exist in the region of asphaltic concrete core and adjacent filters, due to high bulk modulus of the core and high shear modulus of the filters.

The contours of maximum shear stresses are shown in Fig. 14. The asphaltic concrete core assembly is seen to exhibit the highest shear stress which is as expected.

The earth pressures measured by Gloetzl pressure cells at two locations in the downstream rockfill zone are compared with the calculated total stresses in Table II. At both locations the measured vertical earth pressures are much lower than calculated. The readings from earth pressure cells require an initial correction to account for the low stiffness of the fill material compacted adjacent to the cells. This should be done by calibrating the readings against the total stresses calculated in the finite element analysis for the early construction stages of the dam. This correction has not been made to the measurements shown in Table II.

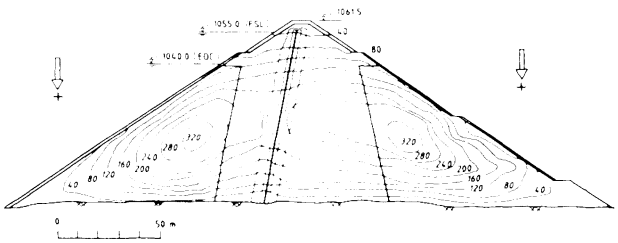

Fig. 11. Predicted vertical displacement in mm at end of reservoir filling.

Table II. Measured and computed total stresses Section A - End of construction.

Cell No.	Earth Pressure measured on 6th October 1987 (kPa)		Computed Total Stress (kPa)		Over burden Pressure (kPa)
	σ_x	σ_y	σ_x	σ_y	γh
G1 (EL. 986, element No. 172)	347	794	314	1260	1509
G2 (EL. 1016, element No. 365)	246	320	126	710	612

7.3 *Strains in the asphaltic core*

Strains in the asphaltic core are measured at 3 locations in Section A. Table III summarizes the measured and calculated strains at these locations. In all cases, the measured as well as the calculated strains are small, the maximum being of the order of 1 per cent. It is noted that in the lower portions of the core the measured strains are compressive in both horizontal and vertical directions, whereas the calculated are compressive vertical and tensile horizontal. In the upper portions of the core the signs of the measured and calculated strains agree. The important observation here is that the strain levels are well below the 2 and 3 per cent level that could cause increased permeability of the asphaltic concrete core.

8. CONCLUSIONS

A series of finite element studies were carried out to analyse the behaviour of the Storvatn Dam embankment. The finite element results have been compared with actual field measurements. The study has enabled the authors to predict the behaviour of the embankment during the final phases of construction and reservoir filling. Based on the total programme of work carried out the following conclusions are presented.

The stress-strain characteristics of the fill materials used in the Storvatn embankment were in general found to be approximately linear in the low stress-strain range as determined from laboratory and field tests. The measurements of internal displacements indicated that the strains prevailing inside the embankment were small, and less than 1 per cent. It is therefore possible to model the material behaviour in linear form in the finite element analysis and to fairly accurately calculate movements within the dam.

After calibrating the finite element model by back-analysing the maximum section of the dam for the construction stage using field measurements taken on that section (A), the model was used for the prediction of displacements and stresses at the end of construction on two other instrumented sections (B and C). Subsequently the model was used to make predictions on the behaviour of the dam during filling of the reservoir up to full reservoir level.

The predictions made on Sections B and C have also produced results that are in general agreement with the measurements taken. Judging by the performance of the model, it is concluded that the predictions made on the behaviour of the dam during first filling to full reservoir level are fairly reliable. The predicted additional horizontal deformation of the dam's crest at Section A due to the filling of the reservoir from EL 1040.0 to EL 1055.0 is 120 mm. (Full reservoir level is expected to be reached during summer 1989.)

The analyses showed that the stiffness of zone 4a rockfill was approximately 3 times higher than that of zone 4b rockfill, although the laboratory and field tests indicated the two rockfills to be of rather similar stiffness. The rockfill in zone 4a was sluiced with water and compacted in 0.8 m layers whereas zone 4b was compacted dry in 1.6 m layers. As intended the difference in construction procedures have resulted in a stiffer rockfill zone 4a and consequently, this has minimized displacements in the region of the asphaltic concrete core. The plate bearing tests did not reveal a difference between the two zones because the tests measured only the stiffness of the rockfill near the top surface of a layer where the efficiency of the vibratory roller would be similar for both zones. Where the layer

Table III. Measured and computed strains in the asphaltic core, Section A – End of construction.

Extensiometer No.	Location	Strain Measured on 3rd November 1986						Computed Strain (%)	
		Horizontal Relative Displacement (mm)		Horizontal Strain (%) ϵ_x	Vertical Relative Displacement (mm)		Vertical Strain (%) ϵ_y	ϵ_x	ϵ_y
			Average			Average			
H1A1	EL. 986, element No. 166	+ 1.4	+ 1.8	+ 4.47	–	–	–	– 0.96	–
H1A2		+ 2.2							
V1A1					+ 0.5				
V1A2		–	–	–	+ 0.6	+ 5.5	+ 2.75	–	+ 1.03
H2A1	EL. 1016, element No. 358	– 0.2	+ 0.5	+ 0.015	–	–	–	– 0.48	–
H2A2		+ 0.3							
V2A1					+ 0.7				
V2A2		–	–	–	+ 1.0	+ 0.85	+ 0.43	–	+ 0.51
H3A1	EL. 1046, element No. 477	– 0.8	– 0.75	– 0.27	–	–	–	– 0.08	–
H3A2		– 0.7							
V3A1					+ 0.6				
V3A2		–	–	–	+ 1.2	+ 0.9	+ 0.45	–	+ 0.09

Note: Compressive strain positive.

thickness is limited to 0.8 m this upper portion is recompacted through compaction of the next layer. The depth efficiency of the vibratory roller is however not sufficient to recompact through a 1.6 m layer.

Analyses of this type, in addition to predicting the future behaviour of the dam, have two other useful functions. They aid in the design of future rockfill dams of similar type and in the design of instrumentation systems for such dams by indicating critical locations for observations and suitable instrument types. Once a comparison between observed and calculated behaviour has been established, the analysis may also be used to examine the behaviour of the dam at locations where no instruments are installed.

Overall, some differences between the measured and predicted behaviour were observed. The inaccuracies in predictions arise from two sources: i.e. in the approxi- [illegible] model, and in the difficulties experienced in deriving representative material parameters. The stress paths followed in the dam are, in some elements, far from oedometer or triaxial test conditions. In addition, the anisotropy in material properties and the variations in fill density cause departure from the analytical model. However, based on our experience with the analyses for Storvatn Dam, one may conclude that seemingly more realistic non-linear constitutive models do not provide improvements over the much simpler linear analyses when applied properly.

9. ACKNOWLEDGEMENTS

The extensive instrumentation and monitoring programme at Storvatn Dam has provided an excellent record of information on the behaviour of a large rockfill dam. On behalf of the dam engineering profession the authors acknowledge the owner of the dam, Statkraft, for funding the instrumentation and monitoring programme, without which this study would not have been possible.

The work described in this paper was carried out during 1986/87 at the Norwegian Geotechnical Institute as an extension of a contract with Statkraft for engineering investigations in respect of Storvatn Dam. During this period the principal author worked at NGI as a research fellow. He is grateful to the Royal Norwegian Council for Scientific and Industrial Research (NTNF) and the Rural Water Commission of Victoria for their support during the fellowship period.

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APPENDIX 8

Hints on problems related to gate vibrations.

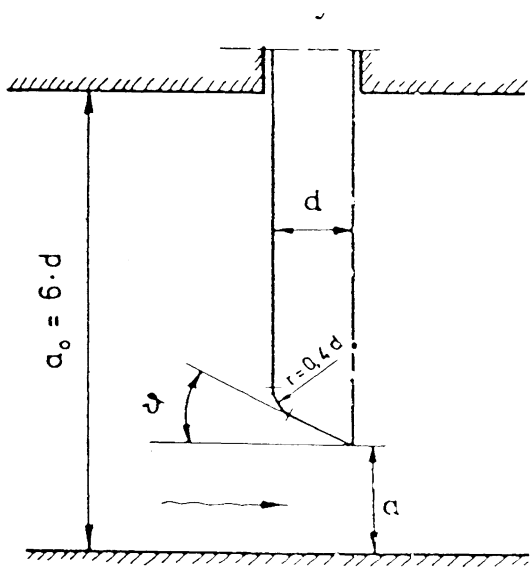
* Some hints on problems related to gate vibrations.
  By J. F. Nicolaisen.

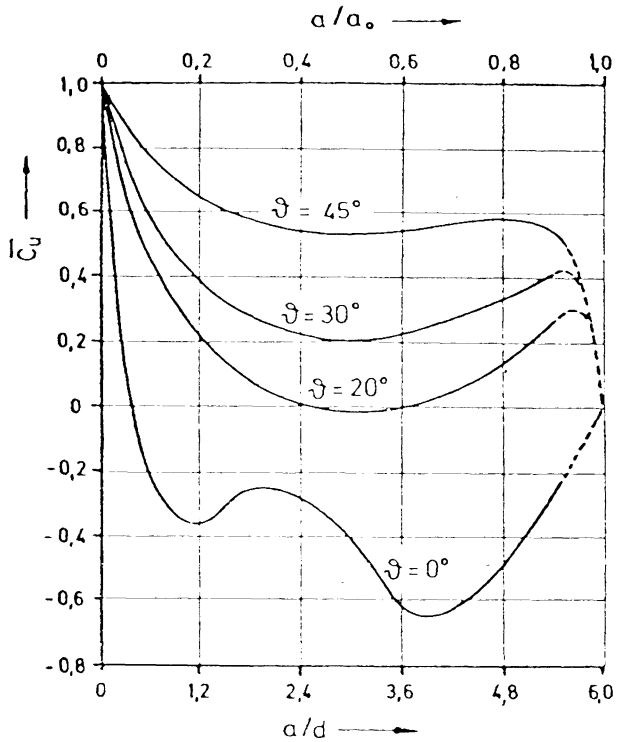
Some hints for evaluation of the danger of vibrations and cavitations at high-pressure-gates.

Jan Fredrik Nicolaisen, Senior Engineer, NVE

The gate at Peruca dam, Croatia, is designed for operation in maximum open and completely closed position. However, the present situation requires more flexibility, with the gate in intermediate positions.
The purpose of these illustrations is merely to give ideas to local engineers in evaluating the gate, rather than to give exact solutions.

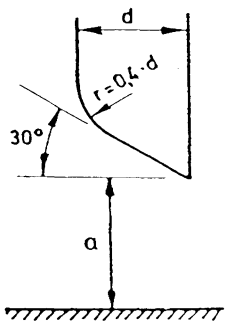
Vertical lifting-pressure from water stream: The lifting-pressure on the gate varies with the angle between the bottom profile of the gate, and the horizontal water-way:

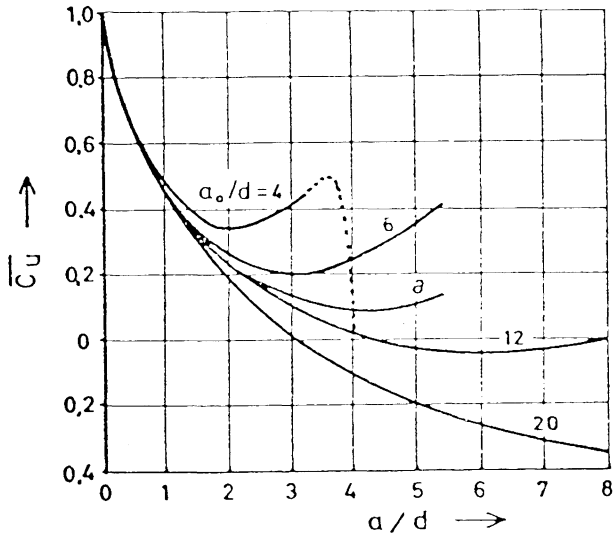




The figure above indicates that bigger angle gives larger lifting-force.

The lifting pressure also varies with different thickness of the gate:
a = The opening-distance a_o = maximum opening distance

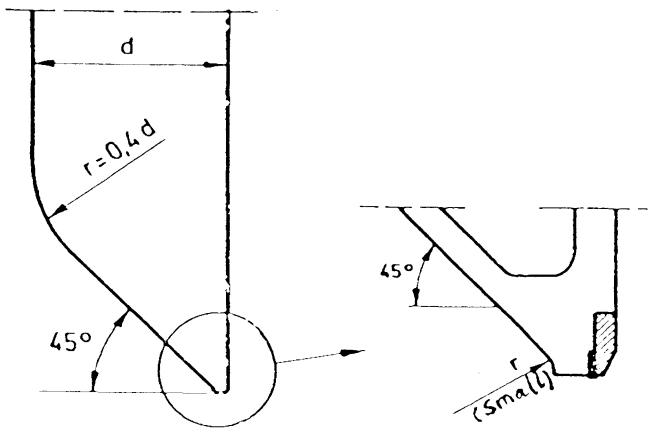




The figure shows that the minimum lifting-pressure come when the gate is about half-open, regardless the thicknes of the gate. Thinner gates give smaller lifting-pressure than thicker.

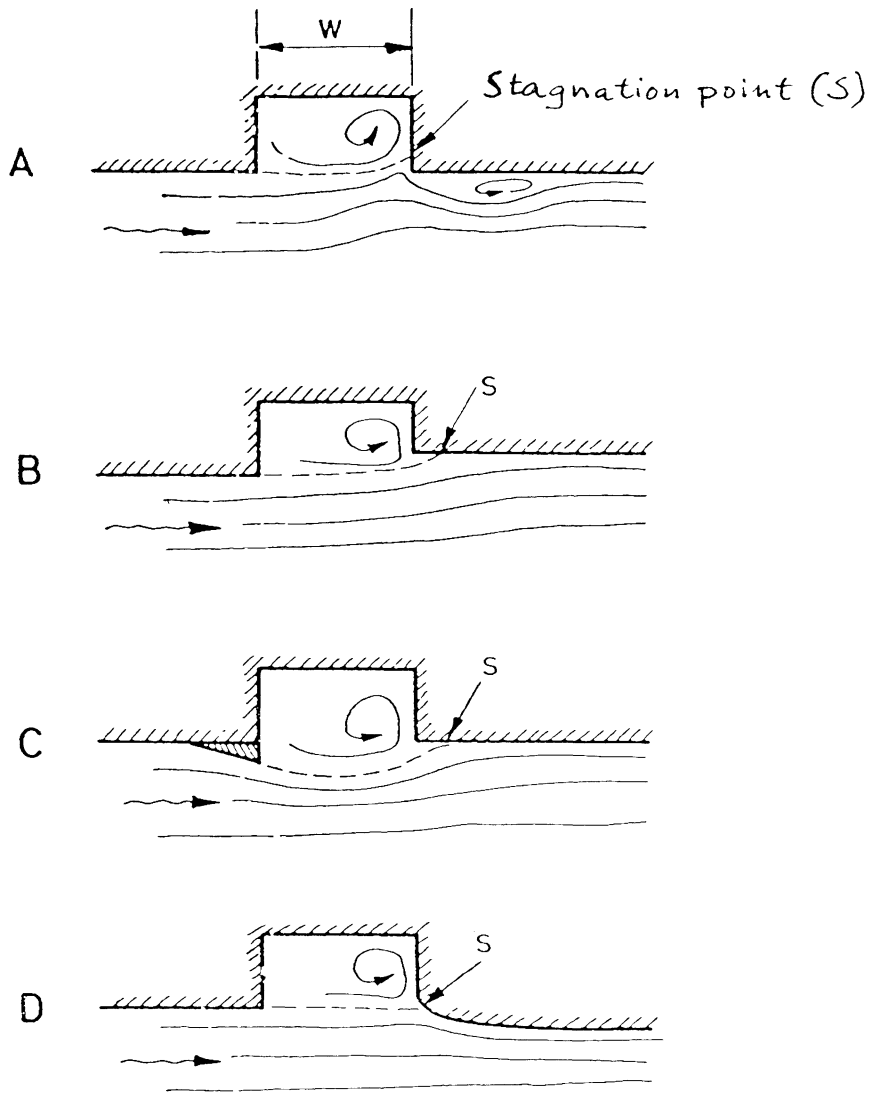
Proposed shape of a gate-bottom-profile:

This profile is proposed by U.S. CORPS OF ENGINEERS and widely used in Norway.:



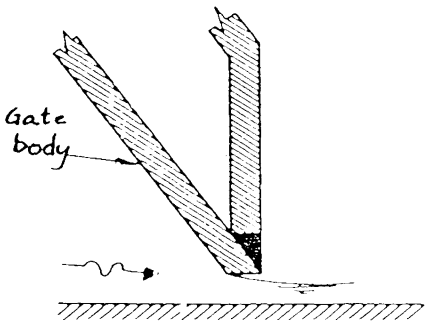
The profile gives great lifting-pressure, and **a stable downstream waterseparation with small vibration/cavitation-problems.**

Stream-pattern in diffenent gate-niches:

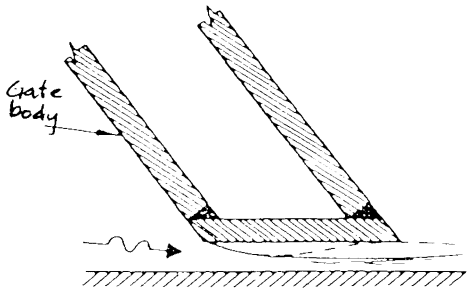


A Situation A is the worst. B, C and D is better.

The importance of a well defined release-edge can not be enough overestimated:



A: Stable stream



B: Unstable stream

APPENDIX 9

Spillway philosophy

* Opinions on ungated vs gated spillway for embankment dams.
  By E. Kleivan, and I. Torblaa.

COMMISSION INTERNATIONALE
DES GRANDS BARRAGES

Seizième Congrès
des Grands Barrages
San Francisco, 1988

OPINIONS ON UNGATED VS GATED SPILLWAY FOR EMBANKMENT DAMS (*)

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Chief Engineers, Ingenior A.B. Berdal A/S
Partner of Norconsult A.S.
NORWAY

1. INTRODUCTION

Even if dams are not being designed for eternity, it can be expected that a large number of the dams built in this century will still be here and serving a purpose some 100 years from now.

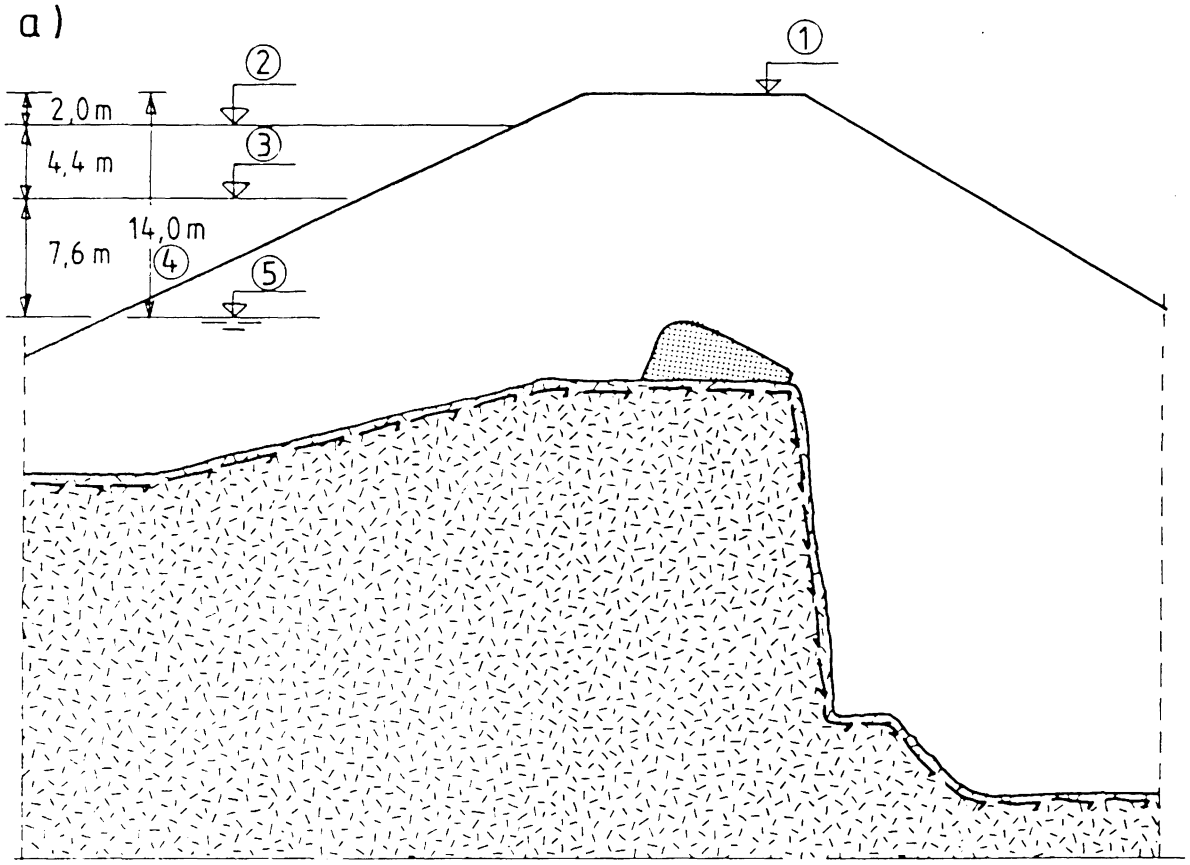
The longevity of embankment dams, more vulnerable to the consequences of overtopping during major flood events than the concrete dams, is likely to be affected to a large extent by the spillway arrangements — on whether gated or ungated.

Since large reductions in the height and volume of an embankment can be the result of designing the spillway with gates, see example in Fig. 1, where a height reduction of 6 m can be achieved, the ensuing cost saving may tempt designers to choose that solution. This apparent cost benefit may, however, be reduced or even reversed when safety considerations are taken into account.

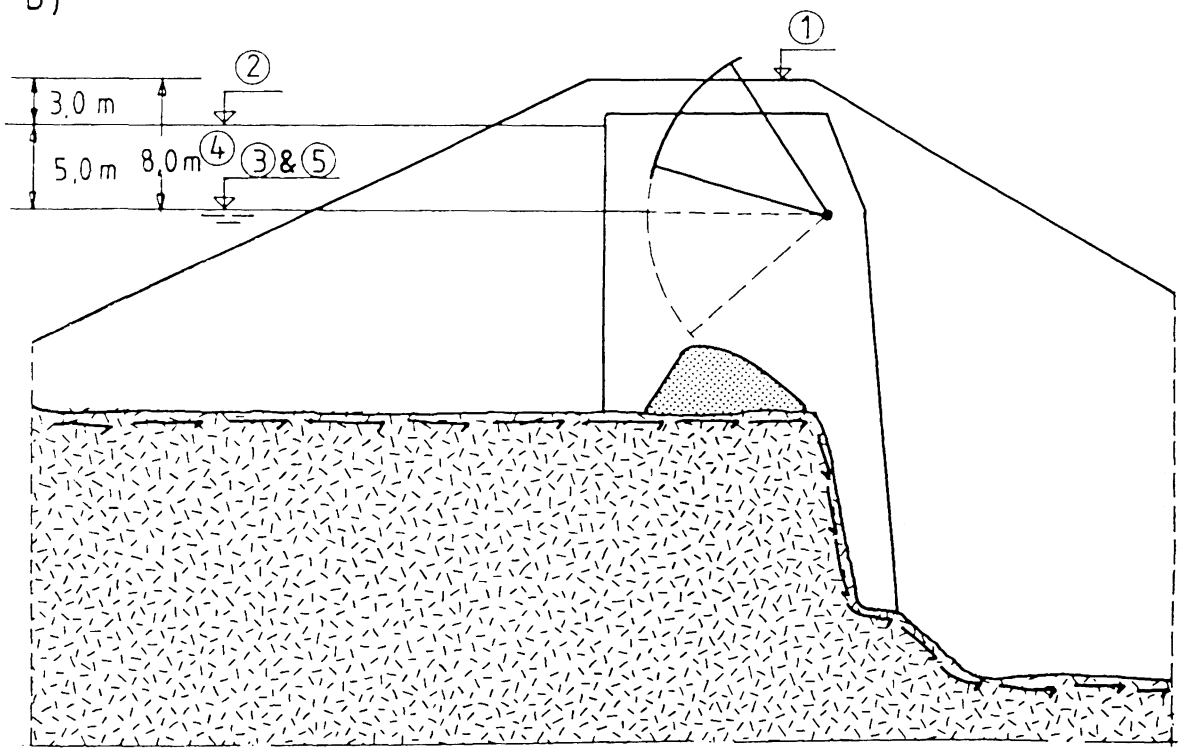
Based on literature published on the subject of controlled and uncontrolled spillways it may be deducted that the present trend is away from gating of the spillways, at least in some parts of the world. Ref. [1], [2] and [3].

(*) *Points de vue sur les évacuateurs de crue sans vannes comparés à des évacuateurs avec vannes pour les barrages en remblai.*

a)



b)



2. SAFETY CONSIDERATIONS

When it comes to dam safety, the gated spillways have several disadvantages, such as :

— Malfunctioning of the gates may result in overtopping of the dam and dam failure.

— Faulty operation of the gates causing artificial floods and inundation of downstream areas.

— Possible failure in communications, power supply and availability of manpower in an extreme flood situation may cause overtopping and damage even if the gates are well maintained and fully operable.

— The vulnerability to destruction through sabotage.

Some recent examples of malfunctioning of gates are the following :

— Euclides da Cunha Dam (1977, Brazil). Dam failure caused by overtopping at least partly due to a bridge being taken by the flood, blocking access to the dam for the crew on their way to open the spillway gates. This dam failure triggered the failure of facilities at another dam further downstream.

— Machhu Dam II (1979, India). Dam failure due to overtopping, which caused heavy losses including human lives. Three out of eighteen gates were out of order due to electrical failure.

Fig. 1

Practical example on how the height of an embankment dam is influenced by the spillway solution

Exemple pratique de l'influence exercée sur la hauteur d'un barrage en remblai par le type d'évacuateur de crue retenu

a) Ungated version, resulting in a dam crest of 14 m above HWL	*a) Sans vanne, donnant un niveau de crête situé à 14 m au-dessus de la retenue normale*
b) Gated version, resulting in a dam crest 8 m above HWL	*b) Avec vanne, donnant un niveau de crête situé à 8 m au-dessus de la retenue normale*
1. Dam crest	*1. Crête du barrage*
2. PMF (Probable Maximum Flood)	*2. Crue maximale probable*
3. Design Flood	*3. Crue de projet*
4. Freeboard above RWL	*4. Revanche au-dessus de la retenue normale (R.N.)*
5. RWL (Retention Water Level)	*5. R.N. (Niveau normal de retenue)*

— Hirakud Reservoir (1980, India). Heavy unseasonal flood, caused by over-reaction on the part of a gate operator, resulting in material losses.

— Tous Dam (1982, Spain). Dam failure attributable to overtopping because power failure made it impossible to open the spillway gates. Losses included 40 human lives.

— Noppikoski Dam (1985, Sweden). Dam failure due to overtopping caused by jamming of one out of two gates, resulting in material losses.

— Lutufallet Dam (1986, Norway). Part of an embankment dam had to be cleared away by blasting, due to a jamming spillway gate.

In a 10-point plan for dam safety proposed by Mr. Pierre Londe in 1982, the points 4 and 5 deal with spillways :

4. In designing spillways and other reservoir outlets, utilise the best hydrological methods with a clear appreciation of the catastrophic consequences of overspilling the main embankment. Fuse plug spillways provide a good safety measure if correctly designed.

5. With a gated spillway, very detailed and strict operating rules are required for safety. Any possible human or mechanical failure has to be anticipated and emergency arrangements made to overcome it.

If a gated spillway is preferred, for instance in a region with severe earthquake conditions, it is obvious that an extra spillway with emergency fuse plug is required.

A brief review of the literature published on the subject of controlled and uncontrolled spillways reveals that in Australia, the trend since 1955 has clearly been away from gated spillways at large dams. According to Ref. [1], only 25 % of the total number of spillways built between 1955 and 1972 were gated, with the percentage dropping to 19 % for spillways built between 1970 and 1972. A similar count from Austria, Ref. [2] shows that only 10 % of all embankment dams in that country were built with gated spillways.

The ungated spillway will never fail to discharge even when unattended, and under the most extreme conditions such as earthquake damage together with power failure and broken communication lines, or under panic conditions during large floods.

3. RISKS AND CONSEQUENCES

According to statistics from ICOLD, referred in ref. [3], which includes more than 10 000 dams higher than 15 m, 216 of these dams have failed. The diagrammes in Fig. 2 and Fig. 3 inform in more detail about the dam failures.

Fig. 2 shows the relative importance of the causes of failure, on top for concrete dams, in the middle for fill dams and at the bottom for all dam types together. It is interesting to note that the three main causes of failure — overtopping, foundation defects and piping — have all about the same rate of incidence.

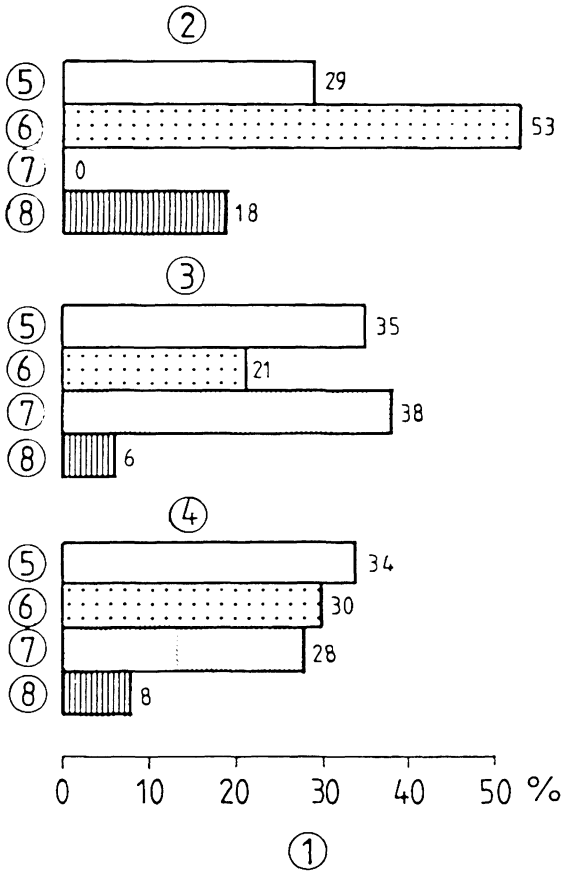


Fig. 2

Causes of dam failures, not including failures during construction and acts of war

Causes de rupture des barrages, ne comprenant pas les ruptures pendant la construction ou à la suite d'actes de guerre

1. Percent of failures
2. Concrete dams
3. Embankment dams
4. All dam types
5. Overtopping
6. Foundation
7. Piping and seepage
8. Others

1. Pourcentage de ruptures de barrages
2. Barrages en béton
3. Barrages en remblai
4. Tous types de barrages
5. Submersion
6. Fondation du barrage
7. Renard et infiltration
8. Autres causes

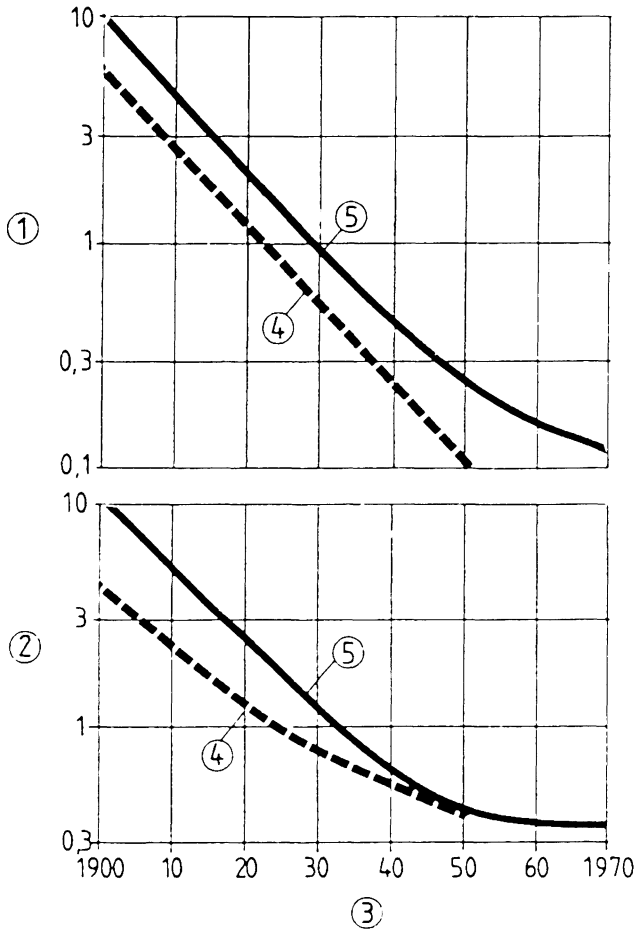


Fig. 3

Risk for dam failure, not including failures during construction and acts of war (ref. [3])

Risque de rupture des barrages, ne comprenant pas les ruptures pendant la construction ou à la suite d'actes de guerre (réf. [3])

1. Percent of all dams in operation
2. Percent of dams built simultaneously
3. Year
4. Concrete dams
5. Embankment dams

1. Pourcentage de tous les barrages en service
2. Pourcentage des barrages construits simultanément
3. Année
4. Barrages en béton
5. Barrages en remblai

Fig. 3 shows the improvement of the rate of failure over the period 1900-1970, separately for concrete and fill dams. The upper graph gives in logarithmic scale the percentage of failed dams in relation to all dams in operation or at risk at a given time. The lower graph gives the proportion of the built dams which later failed and shows :

a) the dramatic (at least tenfold) improvement in safety since the beginning of the century,

b) that modern fill and concrete dams are about equally safe.

It can be concluded that according to statistics the risk for failure of a new embankment dam is approximately 0.4 %, or one out of 250. One third of the failures will be caused by overtopping, but with reference to the previous chapter, it seems probable that the majority of the overtopping cases are explained by malfunctioning of gates. As an estimate the risk for overtopping can probably be set to one out of 1 000.

When evaluating risks, also the consequences should be considered. Fig. 4 presents a relation between stored water height and maximum flood due to dam failure. The diagram is based on observations from 15 dam failures and theoretical estimates.

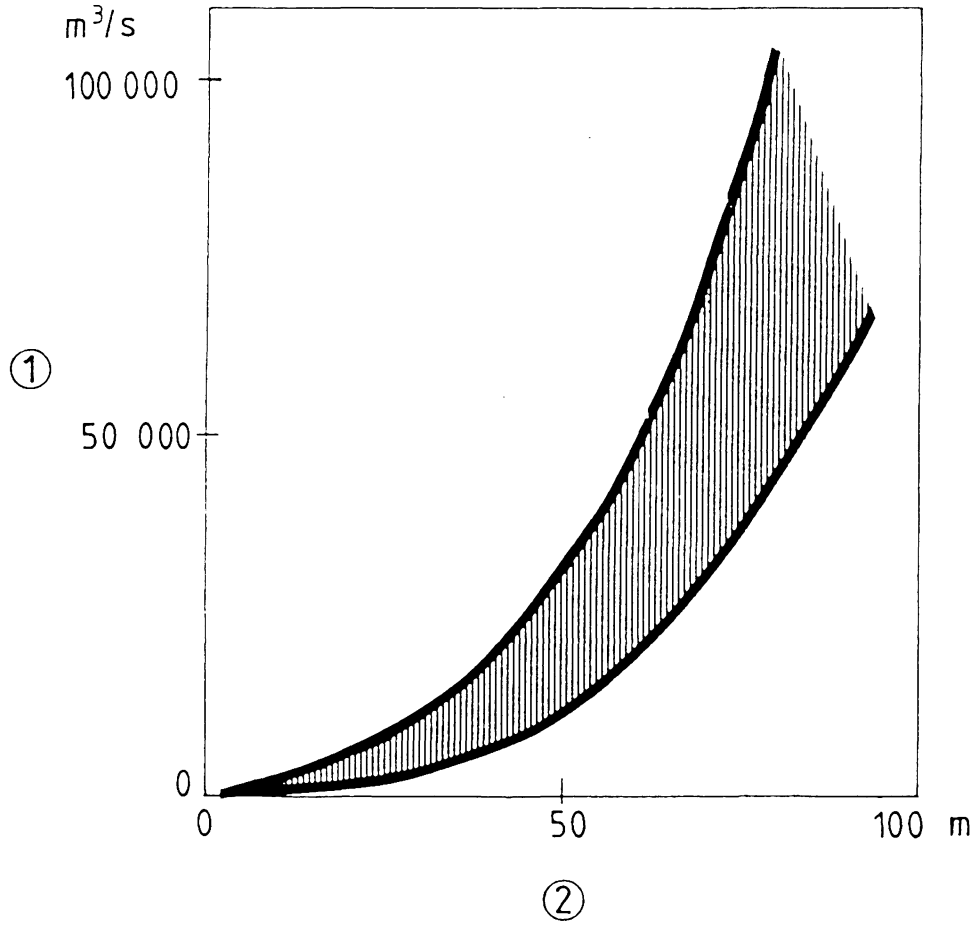


Fig. 4

Relation between stored water height and maximum flood due to dam failure (after G.W. Kirkpatrick)

Relation entre la hauteur de la retenue et la crue maximale due à la rupture du barrage (d'après G.W. Kirkpatrick)

1. Maximum flood, m³/s
2. Stored water height, m.

1. Crue maximale, m³/s
2. Hauteur de la retenue en mètres

In their Bulletin of 1986 on Design of Spillways for Dams, ref. [4], the ICOLD Committee on the hydraulics of dams have reviewed experiences from a total of 182 existing spillways around the world, and made some clear statements regarding the question of gating. Some key excerpts are the following :

— « Gates are optional on surface spillways. This is a distinct advantage in that an ungated spillway is preferable when local conditions such as high seismic activity, lack of confidence in maintenance and/or operating skills, short peaking time of the inflow hydrograph, remoteness of the site and difficulty of access mean that there are doubts as to the dependability of the gates and the way they will be operated » (paragr. 3.2.1.).

— « The seismicity of the area and confidence in the operating system are the prime factors in deciding whether it is appropriate to design a gated structure » (paragr. 2.2.2.).

— « Regarding reliability of operation, the designer must weigh the risk of one or more gates failing to open when the flood arrives because of power failure to the hoisting mechanism or a gate or gates jamming through faulty maintenance. He must also consider the possibility of human error in opening the gates at the wrong time, or too late because the operating rules are misconstrued. The operator must have ready access to the gate controls at all times. It must be realised that en exceptionally large flood may cause panic. If there is the slightest doubt on the reliability of gate operation or the competence of the operating staff, the wise choice is for an ungated spillway » (paragr. 2.2.2.).

— « Standby diesel-electric generators should be provided if power failures are likely » (paragr. 3.4.).

— « Regardless of how reliable gate operation is, it is often stipulated, sometimes by national regulations, to design the spillway to prevent overtopping of the dam with one or more gates failing to open. Gates can also be designed for overtopping. This leads to a larger number of gates or the use of an emergency spillway (uncontrolled overspill, breaching dyke or exploding plug) » (paragr. 3.2.2.).

4. COSTS

As stated in the introduction, the saving for the gated alternative come from the reduction in embankment volume due to the lower crest level.

If, however, the safety situation is such that the gating of the spillway calls for an emergency fuseplug-type spillway in addition, the cost of that extra spillway will reduce or even exceed the saving, depending on the local conditions.

With gated spillways in seismic areas it is also prudent to endeavor reducing the risk for jamming, by setting the gates inside stiff, one-piece frames. The net result is a larger number of moderately sized gates rather than a few very large ones, paragr. 3.4. in ref. (4). This will further penalize the costs of that alternative.

For a proposed project in Burma, where the downstream area for the 130 m high dam is a 3-400 km long flood plain with a large population, the consultant recommended an ungated spillway based on safety considerations. The cost for that alternative incidentally came out as 4 MUSD (or 3.5 % of the dam cost) less expensive than the gated one, mainly due to the cost of the emergency fuse-plug required for the gated alternative.

5. CONCLUSIONS

In cases where the consequences of a dam failure or of faulty operation of the gates may become severe, costs alone should never be made the deciding argument in the choise between a gated or an ungated spillway — where the latter alternative clearly provides a safer existence for the population in the downstream area it should be selected in any case.

Gated spillways should be contemplated only when :

— the consequences of a dam failure is less severe and the cost saving is significant,

— the higher flood rise in the reservoir implied by the ungated alternative is of great inconvenience.

In all other cases ungated spillways should be preferred.

The authors have had the fortune of seeing a dam in Central Burma dating from the year 1200, still in serviceable condition. Needless to say, the spillway was ungated. It is the guess of the authors that if this was not the case, the dam would not still be there.

REFERENCES

1. Johnsen R.B. (1973) : « Spillway types in Australia and factors affecting their choice ». ICOLD, Madrid 1973, vol. II, pp. 833-852.

2. Simmler, H. (1979) : « Spillway design principles in Austria ». ICOLD, New Delhi 1979, vol. III, pp. 885-898.

3. Schnitter, N.J. (1979) : Discussion 52, Q. 49. ICOLD, New Delhi 1979, vol. V, pp. 488-493.

4. ICOLD Committee on the hydraulics of dams : « Bulletin on Design of Spillways for Dams, 1986 ».

SUMMARY

The reduced dam height and volume obtained by the gating of the spillway may tempt designers to rule out the ungated alternative.

A closer look at the safety aspects for the two alternatives, together with a list of recent accidents related to gated spillways, should give rise to second thoughts.

Risks and consequences of an overtopping related to a gated spillway is discussed, and an example given of a cost comparison of gated/ungated alternatives.

The trend worldwide, judging from the somewhat meagre statistics available, seems to be away from the gated spillway.

The conclusion is clearly in favour of ungated spillways when there is a freedom of choice.

RÉSUMÉ

L'évacuateur de crue avec vannes entraîne une réduction de la hauteur et du volume du barrage, ce qui pourrait inciter à rejeter la variante évacuateur sans vanne.

Toutefois, on devrait être amené à une conclusion différente lorsqu'on examine de plus près ces deux types d'ouvrage du point de vue de leur sécurité et lorsqu'on tient compte de tous les accidents liés aux évacuateurs avec vannes survenus récemment.

Ce rapport traite en détail des risques et des conséquences de submersions liées aux évacuateurs avec vannes et donne ensuite un exemple de comparaison des coûts des solutions avec/sans vanne.

On constate, à la lumière des statistiques assez maigres dont on dispose actuellement, une tendance, dans le monde, à s'éloigner de l'évacuateur équipé de vannes.

On conclut en se prononçant nettement en faveur des évacuateurs sans vanne lorsqu'il est possible de choisir entre les deux types d'ouvrage.

APPENDIX 10

Throughflow and
stability problems in
rockfill dams.

* Chapter 4 and 6 from the book "Rockfill Dams"
by B. Kjærnsli, T. Valstad, and K. Høeg.

Book no. 10 in the series HYDROPOWER DEVELOPMENT.

4.
LOADS ARISING FROM WATER AND WIND

Water in the reservoir constitutes the basic external load and generates the effects which may inflict damage or at worst, ultimate failure of the dam. In addition come body forces due to embankment weight and potential earthquake loads.

The water head imposes external loads acting on the impervious element, and submerged fills are subjected to uplift. Flow through the dam and adjacent ground sets up pore pressures and seepage forces acting internally, and waves exert loads on the upstream slope of the dam.

4.1 Seepage Forces and Pore Pressures

Versatile finite element modelling and numerical analyses may be applied to compute steady state and transient pore pressures and seepage forces in an embankment dam and its foundation (Pinder and Grey, 1977). However, so far in design practice, the method of "flow nets" has mainly been used, as even an approximate flow net may give the designer a good appreciation of the flow situation.

A flow net is a two-dimensional, graphic representation of the hydraulic condition and is composed of two sets of curves, equipotential lines and flowlines. Equipotential lines designate points of equal potential. For laminar flow the potential at any point is the sum of the elevation of the point above a reference level and the actual pore pressure at the point, expressed in terms of the piezometric height, i.e. the water column above the point. Hence, the equipotential lines join points in the profile for which the piezometric elevation is the same (Figure 4.1).

The flowlines depict the direction of flow paths through the profile. The area between two adjacent flowlines represents a flow channel. A flow net may be produced on the basis of analytical computations, by means of physical models or analog computers or, most often, graphically by hand sketching. The computer can also convert numerical values from finite element analysis into graphical flow nets, simplifying the interpretation of results (Christian, 1980).

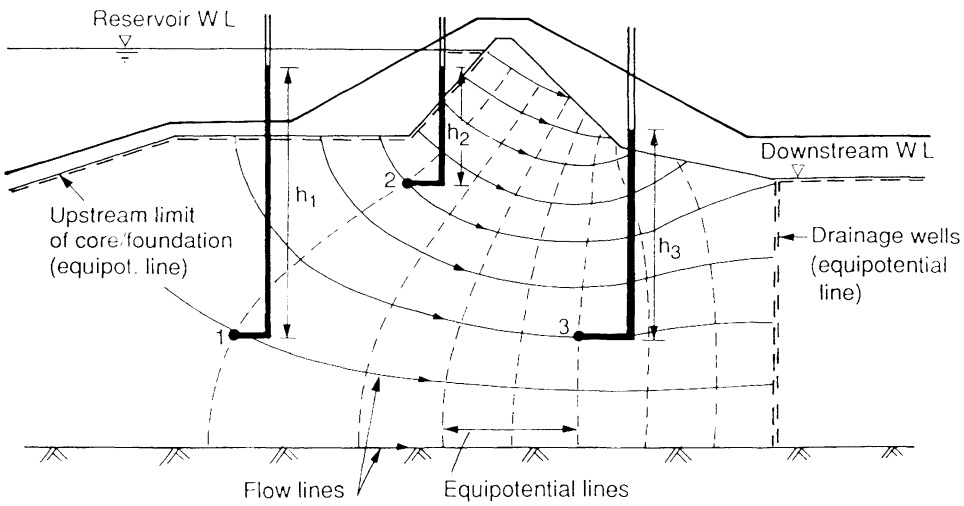


Fig. 4.1
Typical flow net for an embankment dam. Core and foundation are assumed to have same permeability properties.

Under isotropic conditions, i.e. same permeability in all directions, the equipotential lines and the flowlines will intersect each other orthogonally. In practice, the flow net is formed such that the drop in potential from every equipotential line to the next is constant, and each flow channel transports an equal amount of water. These preconditions imply a constant ratio between length and width of each "rectangle" in the flow net (Figure 4.1).

When drawing flow nets, it is first essential to fix the equipotential lines and flow lines given by the boundary conditions. Thus, in Figure 4.1, the flowline along a boundary of watertight rock as well as the equipotential line along the upstream boundary of the impervious core, are given. Likewise, the downstream drainage well which is under hydrostatic pressure, represents an equipotential line. The fact that flowlines and equipotential lines in Figure 4.1 appear in equal numbers is purely incidental. For a more comprehensive introduction to the concept of flow nets the book by Cedergren (1989) is recommended.

By definition, the potential and pore pressure at any point in the section can be determined from the flow net. Hence, the average gradients of flow from one point to another inside the mesh can be determined by measuring the distance between them. The average gradient is defined as the ratio of potential difference to corresponding flow path length.

The drop in potential corresponds to the energy lost over the flow distance between the points. This energy is transferred by friction to the particle matrix through which the water flows, generating a so-called seepage force acting in the direction of the flow. The seepage force per unit volume of the particle matrix is proportional to the flow gradient and the unit weight of water:

$$j = i \cdot \gamma_w$$

where i: hydraulic gradient (dimensionless)

γ_w: unit weight of water

The seepage force exerts an effective pressure in the direction of the flow-line and creates internal stresses that must be added to the effective stresses of the no-flow hydrostatic situation. This is an important principle described for a simple case below.

Figure 4.2 illustrates a situation with vertical, upward exit flow downstream from the embankment toe. In a point at depth, z, below the ground surface a pore pressure, $u = \gamma_w (z + \Delta h)$, exists. The free water level is assumed to be at ground level. The potential drop over the distance, z, equals, Δh, and the average gradient $i = \Delta h/z$.

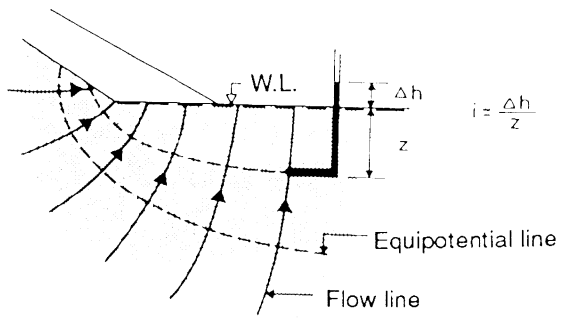


Fig. 4.2
Example of flow net determination of exit gradient. To determine the effective stress situation, use submerged unit weight combined with seepage forces or total unit weight combined with pore pressures determined from flow net.

The vertical effective stress at the same point in the no-flow hydrostatic state is, $\gamma' z$, where, γ', is the submerged unit weight of the material. As the seepage force here acts in the opposite direction of the gravity force, it must be subtracted, and the resulting effective stress at the point in the state of flow becomes:

$$\sigma' = \gamma' z - i\gamma_w z = \gamma' z - \gamma_w \Delta h$$

Alternatively, the effective stress may be calculated as the difference between total stress and pore pressure at any point:

$$\sigma' = \sigma - u = \gamma z - \gamma_w (z + \Delta h) = \gamma' z - \gamma_w \Delta h \quad \text{(q.e.d.)}$$

From this equation, the so-called critical vertical gradient, i_c, at which effective stress at the point would be zero, and at which material particles would become suspended in water, is determined:

$$i_c = \frac{\gamma'}{\gamma_w}$$

To prevent this unstable situation from occurring, the ground surface downstream should be loaded with an inverted filter which increases the effective stresses beneath the surface and stops the erosion and piping. This was the case for the Mänika Dam described in Section 1.3.

From a flow net pore pressures and seepage forces needed for embankment deformation and stability analyses, can be determined. The quantity of seepage through the core can also be calculated by means of a flow net as described in Section 6.4.

4.2 Outflow Forces at Dam Toe

Under normal operating conditions the downstream supporting fill and toe of the dam will not be exposed to seepage forces, and often no requirements are specified for the drainage capacity of the dam. Ample drainage capacity is, however, essential for the safety of the dam if overtopping or big leaks should accidentally occur. This situation must be studied and constitutes a part of the overall risk analysis for the dam.

Outflow of water at the toe of the dam is sketched in Figure 4.3. Depending on run-off intensity, the water may exit on the slope at a certain level above the base determined by the permeability of the fill. The fill is subjected to seepage forces and pore water pressures which reduce the stability along potential sliding planes inside the fill. The overflowed part of the slope is further exposed to surface unravelling and erosion stone by stone.

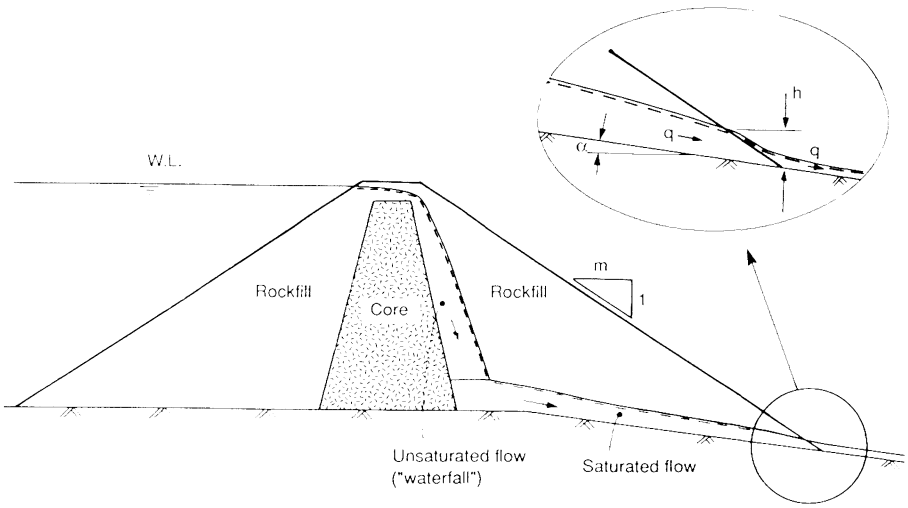


Fig. 4.3
Outflow of water at the toe of the dam. The flow gradient, i, can be estimated by means of even rather approximate flow nets.

Flow through coarse rock fills will generally be turbulent. The flow velocity is calculated as being:

$$v = \sqrt{k_t\, i}$$

where v : discharge velocity;
k_t : permeability at turbulent flow (e.g. m^2/s^2);
i : hydraulic gradient (dimensionless).

The turbulent permeability may roughly be estimated from the equation:

$$k_t = \frac{1}{\beta_0} \frac{n^3}{1-n} g\, d_t$$

where β_0: grain shape factor (β_0 = 3.6 for quarried rock);
n : porosity of the fill;
g : acceleration of gravity; and
d_t : significant particle size.

In well graded materials the significant particle size is approximated by $d_t = 1.7\, d_{10}$. For narrowly graded materials is similarly used, $d_t = d_{50}$. The

sieve opening diameters, d_{10} and d_{50}, represent the sieve openings through which 10% and 50% respectively of the material would pass (by weight).

The porosity, n, may be in the range 20-30% for well graded and well compacted material and 35-40% for narrowly graded material. The permeability, k_t (m^2/s^2), would be in the order of 0.1 d_{10} (m) for the rockfill supporting shoulder and 0.2 d_{50} (m) for the cover stones in the toe and the slope protection. The permeability of the rockfill shoulder can easily be less than 1% of the permeabilities of the protective cover layer of large stones, and will be substantially reduced with increasing content of fines.

The water exits in the slope at an elevation, h, given by the cross-sectional area required for the actual flow (Figure 4.3):

$$h = \frac{q}{\sqrt{k_t i}} \frac{1}{(1 - m \tan \alpha) \cos \alpha}$$

where h : elevation above the reference level;
q : specific flow of water (e.g. m^2/s);
m : slope inclination ratio, $m = \tan \beta$; and
α : slope angle of rock surface in direction of the flow.

The surface of the dam above the tip of the toe and below elevation, h, will be overflowed. The specific flow, q, has a threshold value for stability of the slope depending on the size and weight of the stones, the foundation inclination, α, the effective angle of friction, ϕ', and the slope inclination ratio, m. For a given total outflow, Q, the specific flow, q, will be determined by the shape and width of the dam at the toe. The specific flow may increase dramatically by narrowing gorge effects towards the toe.

The stone sizes required for toe and slope facing, and the critical flow intensity, q, are estimated by means of the empirical equation and diagram presented in Section 6.6.

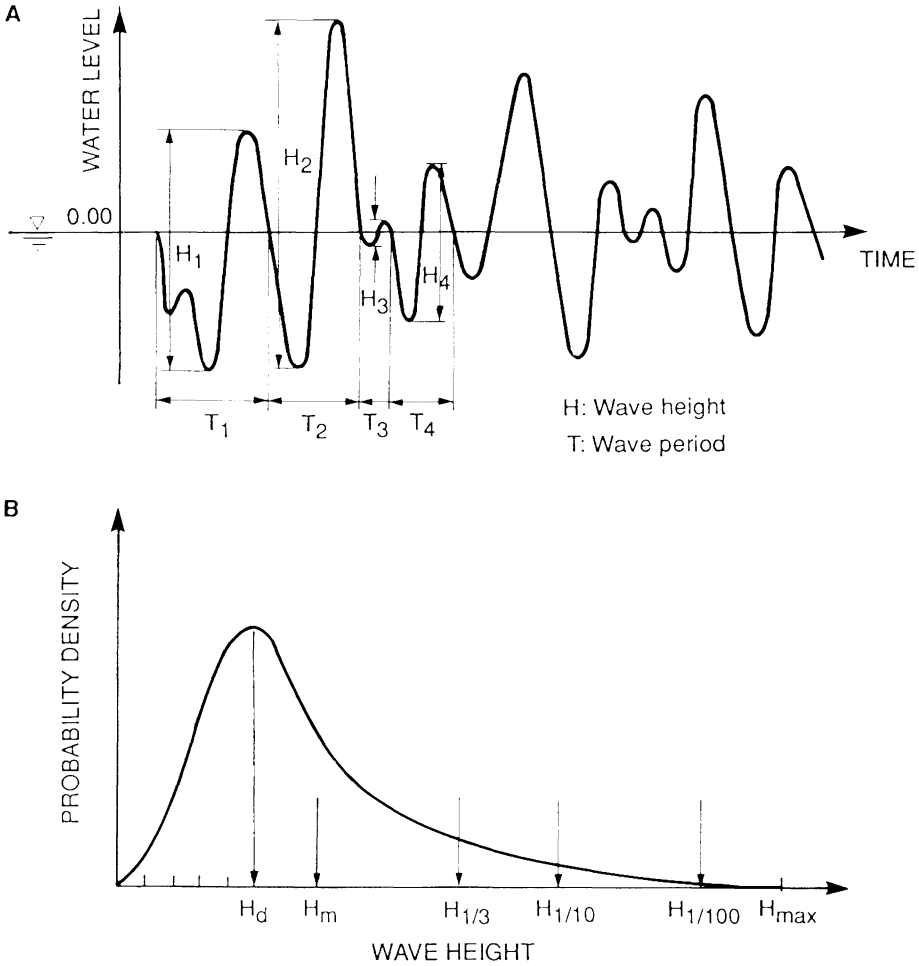
4.3 Wind Generated Waves

Characteristics of the waves on the reservoir are needed for design of slope protection and freeboard. Wave height and steepness are input data to design rock armoured slope protection against wave erosion. Wind tide and wave run-up are input data required for design of freeboard allowance above maximum flood water level. Waves generated by wind will vary in height even if the wind is of constant force and direction. An example of how the wave height and period vary within a wave train and the corresponding probability density function are given in Figure 4.4.

The irregular motion of wind generated waves is illustrated by the probability density function for the wave height in Figure 4.4b. The motion is characterized by:

H_s: significant wave height; and
T_p: peak period.

The significant wave height is here defined as the mean height of the 1/3 highest waves in a wave train. The peak wave period is the period for a wave corresponding to the peak of the wave height probability density function. The relationship between heights and periods are given in Figure 4.4c.



C

Significant wave defined here as mean of 33 percentage highest waves:	$H_{1/3} = 1.00\ H_s$
Most frequent wave (modal wave height):	$H_d = 0.50\ H_s$
Mean wave height:	$H_m = 0.63\ H_s$
Mean of 10 percentage highest waves:	$H_{1/10} = 1.27\ H_s$
Mean of 1 percentage highest waves:	$H_{1/100} = 1.67\ H_s$
Maximum highest wave:	$H_{max} \approx 2.00\ H_s$
Peak wave period:	T_p
Significant wave period:	$T_s = 0.95\ T_p$
Mean wave period:	$T_z = 0.71\ T_p$

Fig. 4.4
Characteristics of wind generated waves
A) A typical train of waves
B) Probability density function for wave height
C) Statistical relationships for waves based on Rayleigh distribution (After Pilarczyk, 1990)

At the design stage the wave height and period will have to be calculated from observations of the mean wind velocity and duration blowing over the reservoir fetch in front of the planned dam. The governing parameters are:

U: average wind velocity,
t : duration of the wind, i.e. the period over which the wind velocity is averaged, and
F: fetch, i.e. the overwater distance from the shore opposite of the dam in the direction the wind is blowing.

Wind observations are rarely available at a dam site. In most cases, however, the wind regime may be estimated from observations at meteorological stations in the vicinity. If wind occurs only in gusts of a few minutes, this will limit the height of the generated waves. Due to insufficient data the calculation of wave height is usually based on the assumption that the average wind is blowing long enough for the waves to be fully developed. The limitation of the fetch on most reservoirs yields fully developed waves for wind duration between 15 minutes to 2 hours. Thus, for establishing the wind regime the extreme values of the wind velocity for a duration of up to 2 hours will need to be determined.

In many countries the national meteorological bureau has published wind maps, showing wind velocity and direction that can be used to estimate the wind regime at dam sites. An example of such maps for the coastal regions of Norway are given by Børresen (1987). If no such maps are available, the wind at the dam site may be correlated with the wind at meteorological stations in the vicinity. Simultaneous observations at the dam site and at the meteorological station for a certain period allow a transfer function to be established. The transfer function must account for the topographical changes due to the dam structure and filling of the reservoir as smoothening of the terrain reduces surface friction. The procedure for such a correlation is given in Manual (1991).

The wind velocity for design of slope protection and freeboard can be made for a return period of 50 years. In Norwegian practice a steady and lasting wind of 30 m/s has been assumed when no observations are available at the site. In projects where wind effects become important for design decisions, wind observations should be performed at the dam site.

Most reservoirs are narrow in width compared to the fetch in front of the dam. The computational formula for wave height, however, is based upon a wide reservoir, i.e. no lateral restrictions. In that case the effective fetch equals the overwater distance. For reservoirs of limited width the effective fetch depends on the shape of the reservoir. A method for determining the effective fetch is presented in Figure 4.5 (Saville et al., 1962).

The effective fetch is determined as the average distance in the direction of the wind within a 90° sector centred about the wind direction:

$$F_e = \frac{\sum_{-45^\circ}^{+45^\circ} R_i \cos^2 \alpha_i}{\sum_{-45^\circ}^{+45^\circ} \cos \alpha_i}$$

where: R_i : radial distance from the shore line to the dam; and
α_i : angle between wind and radial direction.

The shore line at maximum reservoir level is used for determination of the radial distances. The most unfavourable wind direction is assumed to coincide with the orientation of the maximum effective fetch.

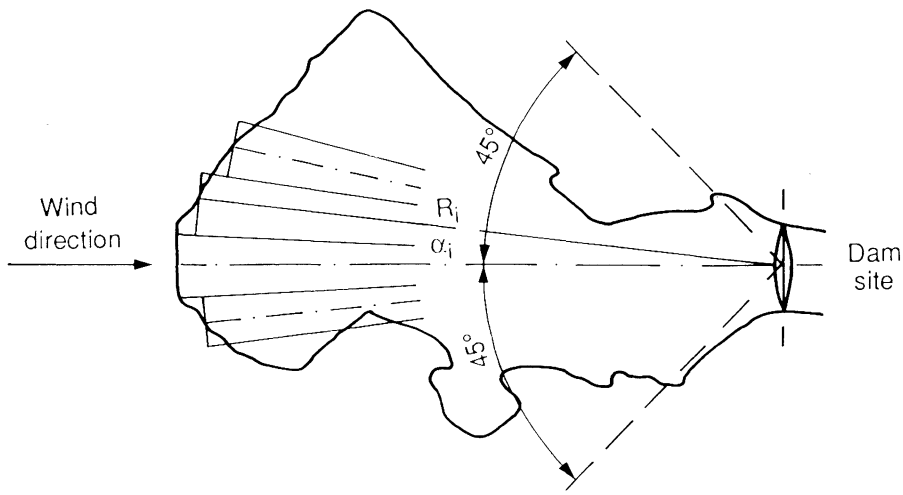


Fig. 4.5
Method for determination of effective fetch

A set of non-dimensional formulae are applied for calculation of wave characteristics, significant wave height, H_s, peak period, T_p, and minimum wind duration, T. For fully developed waves they are:

$$\frac{gH_s}{u_*^2} = 0.0506 \left[\frac{gF_e}{u_*^2}\right]^{\frac{1}{2}}$$

$$\frac{gT_p}{u_*} = 0.903 \left[\frac{gF_e}{u_*^2}\right]^{\frac{1}{3}}$$

$$\frac{gT}{u_*} = 23 \left[\frac{gF_e}{u_*^2}\right]^{\frac{2}{3}}$$

where g : acceleration of gravity; and
u_* : friction velocity.

The friction velocity is calculated from the wind velocity, U_{10} at 10 m height above ground surface by the empirical formula:

$$u_*^2 = C_D\, U_{10}^2 = (0.0008 + 0.000065\, U_{10})\, U_{10}^2$$

The formula is only valid for friction and wind velocities given in metre per second.

4.4 Wave Run-Up and Wind Tide

Waves hitting the upstream slope will run up to a height above the mean water level as shown in Figure 4.6.

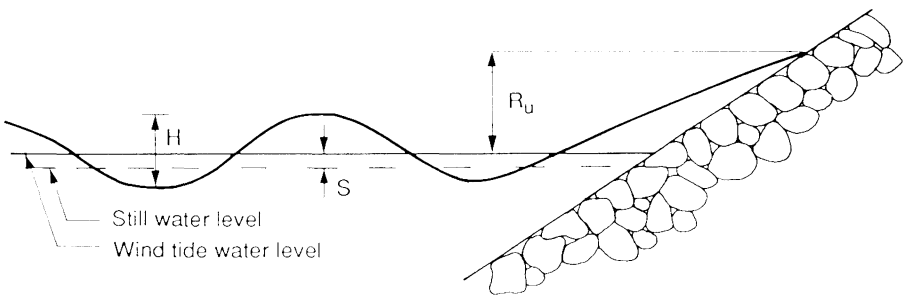


Fig. 4.6
Sketch for wave height, H, wind tide, S, and run-up, R_u, at the upstream slope of an embankment dam

Run-up exceeded only by 2% of the waves in a storm is given by:

$$R_u = rH_s$$

where R_u: run-up, vertical height above still water level;
r: attenuation factor; and
H_s: significant wave height.

The attenuation factor, r, depends on the Iribarren number for the wave slope geometry:

$$I_r = \tan\beta\sqrt{\frac{L_o}{H_s}} = T_p \tan\beta\sqrt{\frac{g}{2\pi H_s}}$$

where I_r: Iribarren number;
ß: slope angle;
L_o: deep water wave length;
T_p: peak wave period; and
g: acceleration of gravity.

The Iribarren number is an expression for the type of wave breaking at the slope:

$I_r < 0.5$	spilling wave
$0.5 < I_r < 2.0$	plunging wave
$2.0 < I_r < 2.6$	plunging or collapsing wave
$2.6 < I_r < 3.1$	collapsing or surging wave
$3.1 < I_r$	collapsing wave

For most embankment dams the Iribarren number will be between 0.5 and 2.0. In Figure 4.7 are given attenuation factors for impermeable and smooth, impermeable and rough, and permeable and rough slopes (Bruun, 1985).

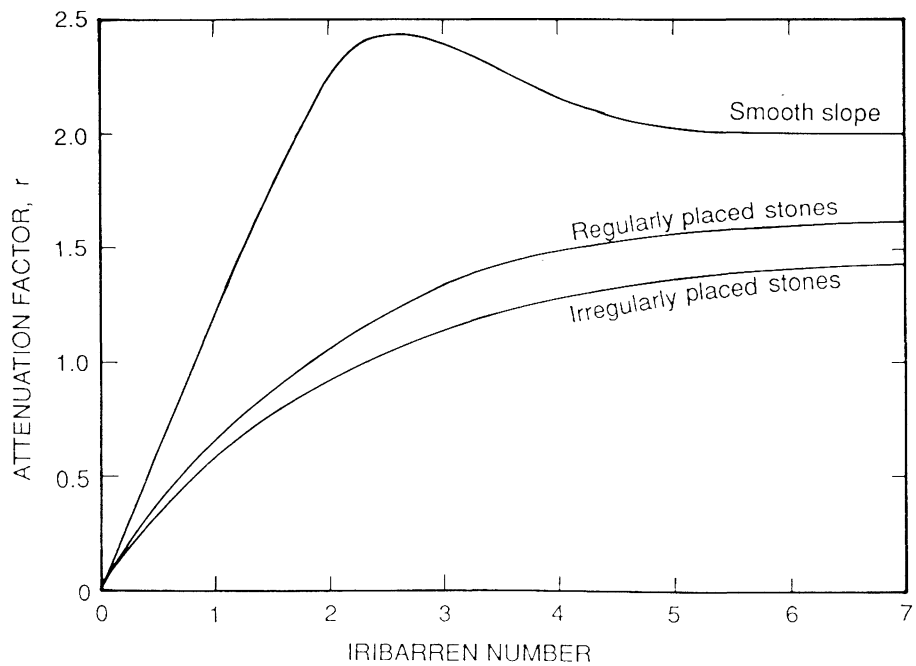


Fig. 4.7
Run-up attenuation factor for typical embankment slopes

Wind tide must be accounted for in shallow reservoirs. The rise of the water due to the shear of persistent wind, the wind tide, S, is estimated from the following equation:

$$S = \frac{U^2 F_e}{4800\, D}$$

where S: wind tide in metres;
U: wind velocity in metres per second;
F_e: fetch in kilometres;
D: average depth of the reservoir along the fetch in metres.

For deep reservoirs the wind tide will be of insignificant size compared to the run-up.

6.
DESIGN AND ANALYSES

Design of rockfill dams must be adapted to geological and topographical conditions at the site. Under any given circumstances, however, alternative solutions will be feasible, and other factors may determine the final choice.

6.1 Choice of Impervious Element

The type of impervious element, that is, the selection of material and its position in the dam, is the first and foremost consideration in the planning of an embankment dam (Kramer, 1988).

When soil of suitable quality is available within an economic distance from the site, the dam should generally be designed with an impervious core of soil. The width of the core has to be decided. Soil cores with widths of half the water head or more at that elevalation, are considered "wide", whereas cores of widths of one third of the water head or less, are considered "narrow". In general, a wide core is considered to be safer than a narrow one. The volume of available material, or their technical properties such as rate of consolidation, may, however, restrict the width of the core.

A wide core may be given a central, symmetrical and vertical position in the dam. A narrow core should preferably be given an inclined position sloping towards the upstream side. The inclined position produces safer stress conditions and thus reduces the risk of arching and cracking in the core. However, a sloping core of low shear strength may create stability problems for the upstream embankment slope. The width and position of the core must be chosen with due consideration to the topography of the site. If the dam is set in a narrow gorge with steep valley slopes, the core should be wide and vertical.

In economic terms moraine cores have proved preferable to other types in most Norwegian dams. However, where moraine is not available, or is in short supply, other impervious materials have to be used. Clay, silty materials and crushed stone ground to sufficient fineness may be used for impervious cores following the same guidelines as for moraine (Chapter 3).

Concrete may be used for waterproofing. In low dams it may be placed in a central vertical wall. The wall should be comparatively thin, approximately

30 cm, reinforced on both sides and appropriately jointed to reduce cracking, both during settling and curing, and during the subsequent placing of fills and later impoundment. Schober (1988) reports the construction of a central concrete core with a bitumen layer on either side to reduce the interface friction and stresses in the core.

For dams of more than 10 to 15 m height a frontal concrete deck is considered preferable to a central concrete wall. A frontal deck should be at least 30 cm thick. Dual side reinforcements, and vertical and perimeter joints must be used to allow for the anticipated movements of the embankment with respect to the fixed concrete plinth boundary of the concrete deck. For dams of this type, reference is made to the publications by Cook and Sherard (1985,1987).

Bituminous impervious elements are well suited to embankment dams. Compacted asphaltic concrete may be applied in a central core wall or in a frontal deck (Arnevik et al., 1988; ICOLD, 1992). Existing central walls have a maximum thickness of about 1% of the water head and minimum thickness 50 cm. A frontal deck should consist of at least two layers each of 6-7 cm thickness.

A central wall of pebble size stones saturated with bitumen may serve as impervious element. The voids of the uniformly graded stone material are filled with hot bitumen. Adjacent filters must, therefore, be impermeable to bitumen under the actual temperature conditions, i.e. the filters must have a sufficient fines content to prevent any flow of bitumen from the wall.

In special cases sealing by means of wooden planking have been used. A wall or deck of wooden planks should consist of two layers of impregnated groove and tongue planed planks of at least three inch thickness. Sometimes an impervious sheet material is placed between wooden plank layers.

Single or multi-layered membranes of synthetic plastic or rubber sheets welded together may serve as impervious elements in minor dams. So far such synthetic membranes have not been extensively used, but geosynthetics in general are rapidly gaining ground in geotechnical engineering.

When the dam is founded on rock, any of the above mentioned sealing elements may serve the purpose. If, however, the dam is to be founded on overburden deposits of considerable thickness, an earth core would be preferable, as it will tolerate and adjust to more severe shear distortions than any other alternative.

6.2 Layout and Foundation Preparation

The foundation of a dam is characterized by topography, load bearing capacity, compressibility and permeability. Design of the embankment and measures to control seepage and pore water pressures in the foundation must take these characteristics into account; they must suit the ground conditions. Dam foundations for practical purposes, fall into two main categories: Solid rock or loose ground. Then there are a number of conditions in-between.

In Norway, rock foundations are generally of sufficient strength and rigidity for any type of dam. With the exception of rare cases where water soluble limestone has led to cavities (karst), rock foundations can be made sufficiently watertight by regular grouting. As a rule a grout curtain is extended down to a depth of not less than one third of the height of the dam, or to a minimum of 6-10 m below the rock surface.

If the permeability of the rock at these depths is still too large, grouting is automatically carried on until satisfactory rock is encountered; or to a maximum depth of two thirds of the height of the dam. It should be observed that it is difficult to establish a sufficiently impervious contact between the grout curtain and the core when the core base is narrow. A wide core is therefore preferable if the foundation rock is of poor quality.

If the surface rock is of exceptionally poor quality, weathered or affected by permafrost, it may have to be removed by excavating a trench down to unaffected or unweathered rock. A core foundation trench must have sloping sides (for example 1:1) and be sufficiently wide (4-5 m at the bottom) to prevent arching across the trench and the ensuing loss of vertical soil stress in the core. Low total stresses in the trench may lead to hydraulic fracturing at this location (Chapter 5). The bottom and sides of the trench must be prepared and treated as required for the core foundation.

At dam sites where the bedrock is covered by overburden to great depths, the bearing capacity, compressibility and permeability of the overburden material will be of decisive importance for the project. In these circumstances, dams with impervious cores of moraine, clay or silty soil are to be preferred. If the overburden consists of material of low bearing capacity such as clay, an earthfill dam may be the best choice. Such ground would require gentle embankment slopes and no advantage can be taken of the high shear strength of rockfill. Steeply sloping rockfill dams are therefore better suited to high-strength foundations such as rock, consolidated moraine or gravel.

In Norway, at altitudes where large dams are sited, the overburden normally consists of materials other than clay, and mostly of moraine or gravelly fluvial deposits. Whenever it is technically feasible and economically acceptable, the core should be extended down to bedrock by excavating a wide trench with gently sloping sides. The load bearing and draining capacities, as well as the compressibility of the unexcavated overburden are important for the stability and deformations of the dam.

If the depth of overburden is so great as to make trenching down to bedrock uneconomical, the sealing of the overburden then becomes of major concern in the project. Figure 1.6 in Chapter 1 gives examples of Norwegian rockfill dams with and without foundation trenches down to bedrock. With the exception of the Mänika and Nerskogen Dams, all dams without a trench are founded on sufficiently impermeable moraine. At Mänika and Nerskogen Dams, adequate sealing was achieved by extending the core with an impervious blanket covering the ground into the reservoir up-

stream. Stability has been secured by a ballasting filter blanket extending downstream from the embankment toe.

If the overburden consists of thick deposits of gravel or coarse sand, sealing by means of grouting must be considered. Grout curtains have been successfully constructed to depths greater than 100 m under high dams. For intermediate depths, where open trenching is not feasible or economical, cut-off walls have been established in permeable deposits by the use of the slurry trench method. The deep, vertical slit trench is during excavation filled with a stabilizing slurry which is subsequently displaced by a permanent cut-off wall material such as concrete or clay.

For dams founded on soil overburden it is essential to secure free drainage of any water from seepage and leaks through the foundation. Seepage water must not be allowed to set up high pore pressures, which could threaten the stability of the dam or cause erosion in loose materials underneath the dam. To meet both requirements, the drainage system, usually combining filters, wells and ditches as necessary, should be designed to trap the water immediately downstream of the cut-off wall. This will secure a free exit and safe outlets for the water collected. Additionally, a ballast filter downstream of the dam may be appropriate. The need for such measures is analysed by means of finite element computer models or flow nets drawn by hand (Chapter 4). To monitor the actual drainage conditions, pore pressures in the foundation are measured by means of piezometers or standpipes as presented in Chapter 8.

Concrete structures in conjunction with embankments should preferably be founded on rock. Concrete surfaces abutting the earth core must be free of protruding ledges and should be inclined away from the core to minimize the risk of cracks along the soil-concrete interface. Inclinations in the order 10:1 have proved satisfactory.

Special attention should be paid to interior culverts which have been the cause of failures of several dams internationally. Culverts should preferably be cast in rock trenches with the top more or less on level with the surrounding rock surface. Culverts may also be set on the rock surface, but the part of the culvert protruding into the core should have sloping sides, for instance 1:1, to avoid a local zone of low soil stresses susceptible to hydraulic splitting in the core.

The alignment of the dam is chosen with a view to obtaining minimum dam volume. The dam axis should preferably be convex towards the upstream side. The topography of the site may, however, be such that an axis convex towards the downstream side would entail significantly less volume. The savings from a volume reduction have to be compared with the cost of measures required to cope with the increased risk of cracking in the dam (Arnevik et al., 1988).

The likelihood of unfavourable deformations and possible cracking in the core is analysed from the topography of the site by studying maps supple-

mented with cross and longitudinal profiles of special interest. The cross-valley profile should be smooth without precipices or abruptly changing slopes. Overhanging or steep rock walls are to be avoided or flattened by blasting. Local narrow gorges and deep clefts must be backfilled with concrete (Chapter 7).

Balancing the core along a high ridge with the supporting shoulder fill sitting on rock slopes dipping to one side or both, may create longitudinal cracks in the core. The horizontal stresses in the core are also reduced, rendering the core susceptible to transverse cracking by hydraulic fracturing.

6.3 Need for Foundation Gallery?

In many countries it is common practice to build a gallery under the impervious core. The purpose is inspection and drainage control and to facilitate grouting, if required, of the foundation and possibly of the core after the embankment is completed. The gallery may consist of a concrete culvert built in a trench under and along the dam axis or a tunnel in rock at some greater depth in the foundation. None of the Norwegian large embankment dams have such a gallery. The cost-benefit effect has in general not been found favourable, and the gallery has not been deemed necessary or desirable from a safety point of view. The 90 m high Storvatn Dam with an asphaltic concrete core will be used as a case study in the following discussion (see Figure 5.11).

It was decided that the asphaltic core should be founded on a concrete structure in a shallow rock trench. Furthermore, this structure should be anchored into the rock foundation, and rock grouting should be performed. A main question was, however, whether this concrete structure should be a simple concrete sill (plinth) or a concrete gallery.

The extra cost of erecting a gallery was estimated to ca. 10% of the total cost of the dam, and erecting a gallery would possibly require a prolongation of the time of construction by one year. Would the advantages of a gallery be worth the extra construction and time cost?

The advantage of carrying out the primary construction grouting from a gallery was looked upon as minor or nil. The advantage of a grouting gallery would therefore depend on the need for supplementary (secondary) grouting during the lifetime of the dam. Based on the results from ten exploratory diamond drilled holes to a depth of more than 50 m, the likelihood that the foundation grout work would need future repair, was looked upon as very little. In any case a possible future increase in permeability of the rock foundation was evaluated to be of little economic significance and could be reduced by grouting from tunnels driven under the dam, if really required. If leaks should develop through the asphaltic concrete core, it would not be any easier to grout from a gallery than from the top of the dam. A gallery under the core was not required for the instrumentation and collection system to measure and localize potential leakage in a reliable manner. Thus, in the final design, the decision was made not to include a gallery.

However, for the planned 120 m high Storglomvatn Dam (Table7.1), a gallery is incorporated. The dam rests partly on a moderately karstic rock foundation, and it is very difficult to predict the amount of seepage under the dam. Internationally, there are several reports of dams on karstic foundations where leakage problems have developed. For the Storglomvatn Dam a tunnel has been designed at 25-40 m depth in the karstic rock region under the core base to facilitate grouting, if required.

6.4 Seepage Estimates

The interpretation and use of flow nets was briefly presented in Chapter 4. From a flow net with a mesh consisting of "squares", the quantity of seepage through a 1-metre long section of the dam is given by the equation (Figure 4.1):

$$q = \frac{n_f}{n_d} kH$$

where n_f : number of flow channels;
n_d : number of equipotential drops;
k : permeability; and
H : total potential drop from upstream to downstream

The ratio, n_f/n_d , represents a geometrical form factor. This ratio is determined fairly accurately from even very approximate flow nets. The main uncertainty in the computation stems from the determination of the permeability, k, for which estimates may easily vary by a factor of 10. Moreover it may be much greater in the horizontal than in the vertical direction depending on the character of the core material and the way it is placed and compacted. Flow nets which account for anisotropic permeability and flow across interfaces between different zones, are presented by Cedergren (1989).

In order to compute total seepage through the dam, it must be divided longitudinally into representative sections and flow nets drawn for each section. The total seepage is calculated by summing the seepage for all sections (Figure 6.1).

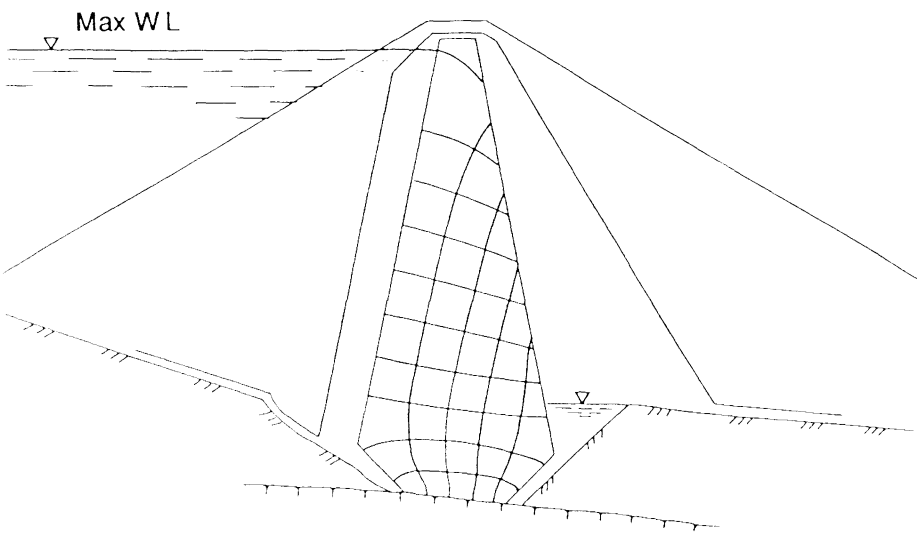


Fig. 6.1
Flow net for a moraine core in a rockfill dam

6.5 Freeboard and Camber

The top of the dam should have sufficient freeboard to prevent waves from splashing over the dam at highest flood water level. The required minimum freeboard is determined as the sum of the maxima for flood water rise, wind tide, wave run-up and possible seiche effect (Chapter 4).

The core should have sufficient freeboard to prevent overtopping. As the effects of waves and run-up are attenuated before the water reaches the core, the freeboard of the central core relative to the design flood water level is commonly stipulated equal to 0.5 metres. Similarly, the freeboard to the top of the dam relative to the design flood water level has been set equal to 2.5 metres as a minimum.

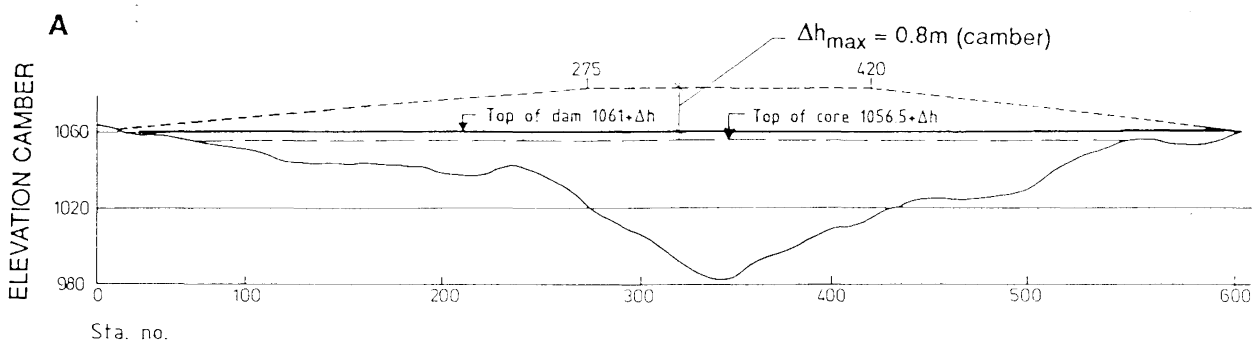
For dams with frontal deck, the deck must have a freeboard of sufficient height to prevent waves from washing over it. Often the deck is topped off with a vertical wave-breaking wall, mainly to save costs. The freeboard required in such cases is set with reference to the top of the wave breaker and may have to be checked by model tests.

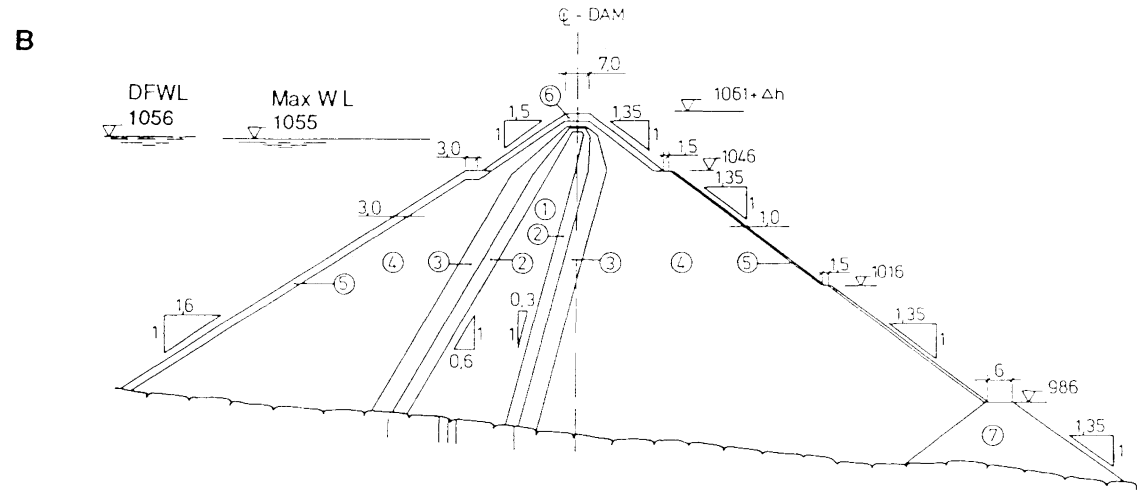
Several failures of minor embankment dams in Norway and abroad have been due to insufficient freeboards. The main cause has frequently been an underestimate of the rise of water levels during high floods. Freeboard requirements, therefore, should be estimated with ample safety margins, which will depend on the reliability of the hydrological statistics available and the type of dam involved. Earth fill dams require higher freeboards against flowing water and overtopping than do the less vulnerable rockfill dams. Higher freeboard improves safety against overflow damage, and will also augment compressional stresses and thus reduce the risk of cracks in the top of the core.

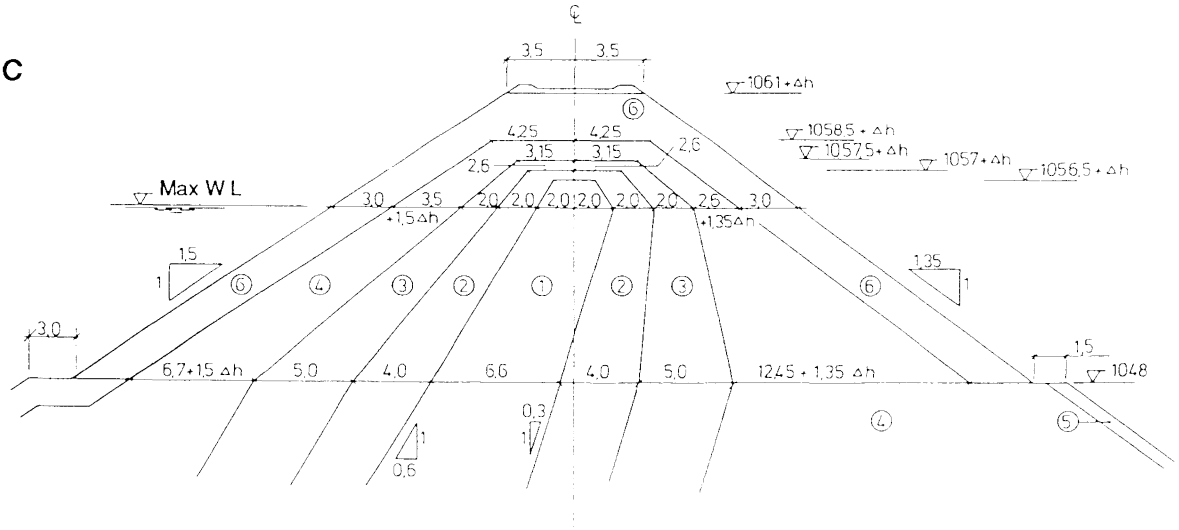
Settlements in embankment dams continue after the dam is completed. To ensure that required freeboard levels remain unaffected even after many years of operation, a dam is built with a sufficient camber to compensate for future settlement. A camber, expressed as a fixed percentage of the dam height, is appropriate for dams with a fairly smooth cross-valley profile without steeply inclined abutments. The relative settlement may, however, vary considerably over a cross-valley profile with steep or abruptly changing inclinations as shown in Figure 5.6. Relatively large settlements are likely to take place above the steepest part of the profile. In such cases the camber should be stipulated in fixed dimensions over the cross-valley profile, rather than in percentages (Figure 6.2).

The camber should cover expected settlements allowing a certain margin of safety. For well compacted fills of good quality rock, founded on rock, a camber of about 0.5-1% would be satisfactory. For fills not compacted or of lower quality rock, the camber should be 1.5-2%. The top of the core is given the same camber as the top of the dam.

The camber must be considered so that the outer slopes are kept to the designed inclination and coincide with the designed width of the top of dam







DFWL = Design Flood Water Level
Max WL = Max. Regulated Water Level (Retention Water Level)

Fig. 6.2 Example of specification of cross-sectional dimensions and camber for a rockfill dam; Δh = camber

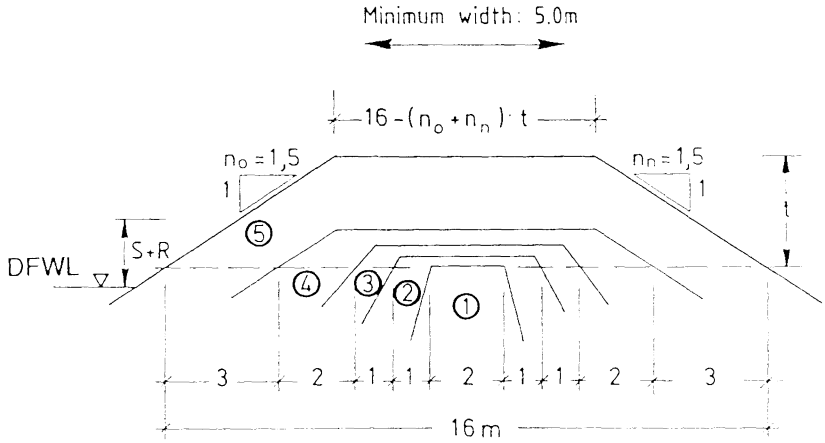
A) Longitudinal section B) Cross-section C) Crown details

without steepening the upper part of the slopes. It is preferable to assign the increase in width to the supporting shoulder fill, zone 4, and to keep the widths of all other zones independent of camber as illustrated in Figure 6.2.

For dams founded on compressible materials, the camber must cover total combined settlements of the fill and the underlying foundation material.

An increase in the width at the top of the dam, similar to an increase in freeboard, enhances the safety of the dam. The width at the top is, however, mainly dictated by practical considerations. It must be wide enough to allow proper placement and compaction of the fill and to accommodate planned traffic when the dam is completed. The width required may in practice depend somewhat on the length and the height of the dam, but usually is at least 5 metres. If a public road is involved, the relevant regulations must be observed and complied with. In any event, a road should be made at the top of the dam for inspection and maintenance.

In zoned rockfill dams with a moraine core and several zones of different gradings on both sides, the width is dictated by the working space requirements at the top of core level. Figure 6.3 shows an example of dam crown dimensions and construction lines. With the top of the dam 2 metres above the top of the core, the minimum width at the top of dam will be 10 metres. This is because, at the core top level, a minimum width of 16 metres is required to accommodate the nine separate zones.



SYMBOLS:

t	= min. 2.0m
DFWL	= Design Flood Water Level
n_o and n_n	= Upstream and downstream slopes
S + R	= Surge and wave run-up

DAM ZONES:

①	Core	④	Shoulder
②	Filter	⑤	Crown cap
③	Transition		

Fig. 6.3
Example of dam crown design resulting from the widths of the separate zones, slope inclinations and height of freeboard

If the height is increased by 2 m, the top of the dam would only be 4 m wide, which is insufficient in most cases. The width of the top of the dam

must therefore be set in accordance with the actual freeboard and working space requirements. General or special safety aspects and future traffic may require a wider crown.

6.6 Accidental Leakage and Required Drainage Capacity

Unexpected leaks through or under an embankment cannot be discounted. Filters which effectively prevent erosion are therefore of the greatest importance for the safety of embankment dams. The drainage capacity must be sufficient to let even large leaks pass through freely without creating high pore pressures or seepage forces which can be critical and threaten the stability of the dam.

Criteria for filter materials exist and are discussed in Section 2.3. They are generally based on the required grain-size distribution of the filter material related to the grain-size distribution of the base material to be protected against erosion. In view of their importance for dam safety, it is advisable to use a strict standard for the selection of filter materials as described in Chapter 2. The filter criteria should be met at each transition zone interface. Several filter zones may therefore be necessary to bridge the gradation gap between the earth core and the big stones in the slope protection and toe of the dam.

The drainage capacity denotes the maximum water flow intensity which the downstream fill can sustain without losing stability. The head of water required to cause the maximum flow intensity must not entail such rise of pore water pressures in the downstream supporting shoulder as to create deepseated sliding inside the downstream slope.

Similarly, the drainage capacity of the toe and slope protection indicates the greatest flow intensity which the toe and protection can sustain when

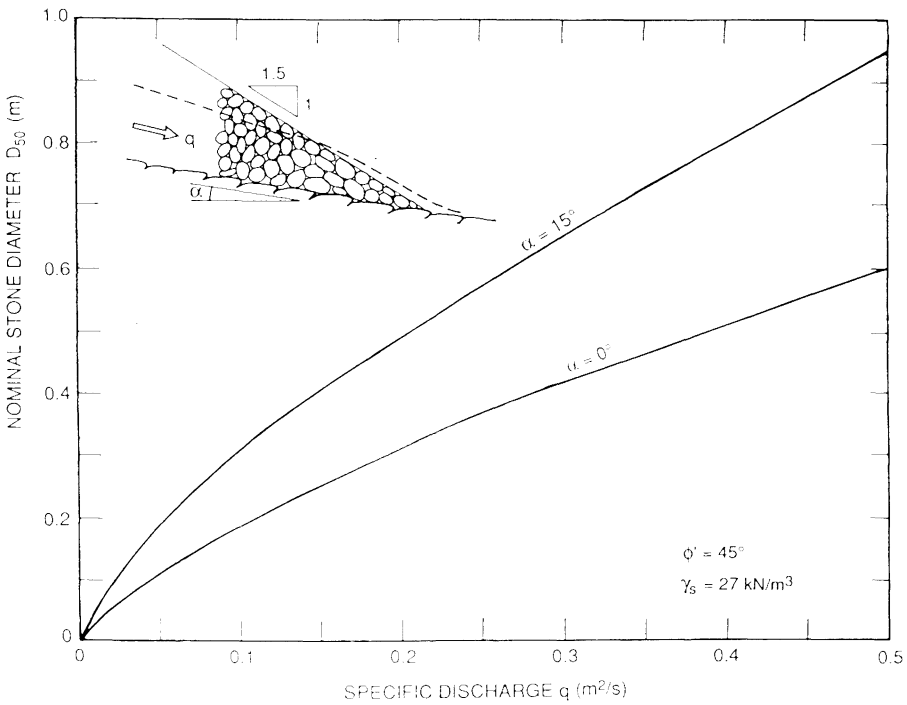


Fig. 6.4 Diagram for design of stones in the toe of the downstream slope (Modified by the authors from diagrams presented by Solvik, 1991)

subjected to flow without dislodgement of single stones from the toe. The critical flow intensity has been determined empirically from model tests. The diagram in Figure 6.4 gives the required size of stones in the toe for the appropriate outflow intensities. The size of stones depends on slope inclination, m, base inclination angle, α, effective angle of friction, ϕ', density of stones, ρ_r, and the shape of the stones.

The ϕ'-value is usually greater than 45°, but can be less on the interface for very smooth base rock surfaces. When exceptionally smooth or unfavourably inclined rock surfaces are encountered, the toe stones should be secured by using ditches excavated into the rock surface, concrete plinths, or similar measures to prevent sliding (Barton and Kjærnsli, 1981).

6.7 Upstream Slope Protection

[illegible] rocks with sufficient weight to withstand the action of waves as ice sheet [illegible] reservoir.

Design of rock armoured slope protection requires wave data for selected directions of wave approach. The wave data will be in the form of significant wave height, H_s, average wave period, T_z, and angle of approach to the slope (Chapter 4).

Design of armour stones has in Norway so far been based on the chart shown in Figure 6.5 giving the median weight of stones placed in an interlocking grid on the upstream slope protection layer. The design wave height, H_d, should be set equal to 1.3 H_s. The chart is based upon a formula developed by Hudson (1958) and experimental data from model tests of riprap on earth embankments (Thomsen et al., 1972).

Recent research has improved calculation of stone weights by taking into account the wave period and the wave breaking mechanism. A general

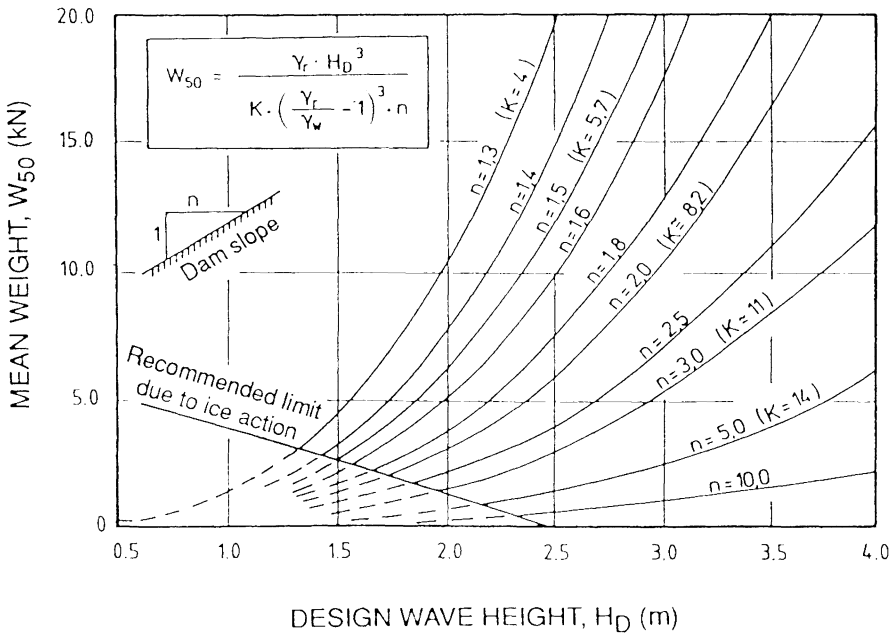


Fig. 6.5
Required weight of stones when individually placed in an interlocking pattern. Dumped stones must weigh twice as much.

stability formula for design developed by Pilarczyk (1990) can be expressed as follows:

$$\frac{H_s}{\Delta D_{n50}} = \frac{1}{F} \Psi K_M \cos \beta$$

where H_s : significant wave height (see Chapter 4);

D_{n50} : nominal diameter of median armour stone;

Δ : relative submerged density, $\Delta = \rho_s / \rho_w - 1$;

F : coefficient of safety against the threshold stability condition;

Ψ : system stability coefficient relative to randomly dumped armour rock;

K_M : stability number for randomly dumped armour rock;

ß : slope angle

The nominal diameter of the cover stones is defined as:

$$D_{n50} = \sqrt[3]{\frac{W_{50}}{\rho_s}}$$

where W_{50} : mean weight of individual stones, equal to the 50 per cent value of the grain-size distribution curve.

The stablity number, K_M , for randomly dumped armour rock is given in Figure 6.6 as a function of the surf similarity parameter (Meer, 1987). The diagram in Figure 6.6a is applicable for rockfill dams with a pervious fill below the armour stones, and the diagram in Figure 6.6b for earthfill dams with a shallow filter and an essentially impervious fill (silt, clay) below the armour stones. The surf similarity parameter, ξ_z, is defined as:

$$\xi_z = \tan \beta \ / \ \sqrt{\frac{2\pi H_s}{g T_z^2}} = 0.40 \tan \beta \ T_z \ \sqrt{\frac{H_s}{g}}$$

The stability of armoured rock can be improved substantially by placing stones in an interlocking grid rather than dumping randomly on the slope. In the stability formula this is accounted for by the system stability coefficient, Ψ. Typical values for individually placed stones are:

$\Psi = 1.0$: reference value for randomly dumped armour stones;

$\Psi = 1.3$: armour stones placed individually in an interlocking grid with the longest axis perpendicular to slope;

$\Psi = 1.5$: armour stones placed in a closely fitting dry masonry.

A Ψ-value of 1.3 can be applied for armour stones placed as illustrated on Photo 7.13.

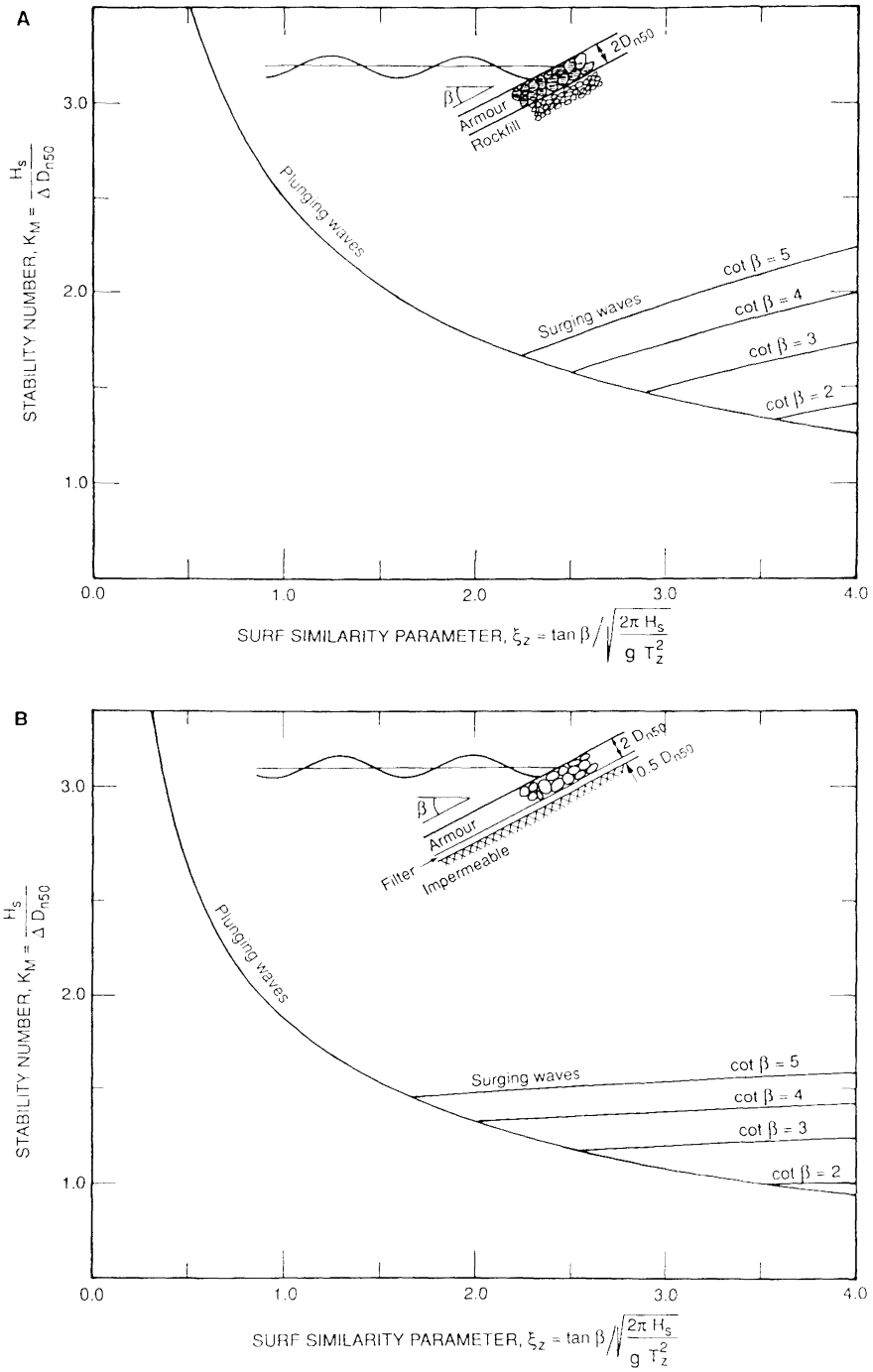


Fig. 6.6
Van der Meer's diagram for stability number, K_M, defining the stability threshold value for randomly dumped armour stones. (After Meer, 1987: using S = 2 and N = 10 000)

A) Permeable base (P = 0.5)

B) Impermeable base (P = 0.1)

The procedure for using the general stability formula is:

1. Given the characteristics of the waves, H_s and T_z, and the slope angle, β, the surf similarity parameter, ξ_z, is calculated.
2. The stability mumber, K_M, is taken from the appropriate diagram.
3. The system stability coefficient, Ψ, is selected according to the placement technique to be applied.
4. The coefficient of safety against the treshold stability condition can for embankment dam slope protection be set to 1.1.
5. The nominal diameter, D_{n50}, and the corresponding median weight, W_{50}, is calculated from the stability formula.

Stones selected and individually placed in a slope protection cover shall be uniformly graded. The ratio, W_{85}/W_{15} , should be less than 10. For control purposes it may be simpler to stipulate the minimum weight instead of the median weight. The safest way may be to stipulate a minimum size equal to the median size determined. However, minimum weight of half the median can be tolerated.

It is impractical to weigh the individual stones. Stone size is characterized by the longest, shortest and intermediate cross-sectional measures of the stone. A given weight or volume of stones may be converted to a corresponding median diameter size by:

$$D = \sqrt[3]{\frac{W}{K\,\rho_s}}$$

where K: shape coefficient for quarried stones;
$K = 0.45$ for elongated stones typical for schistose rocks; and
$K = 0.75$ for cubical stones typical for granitic rocks.

The thickness of the upstream slope wave protection cover should be at least twice the largest stone diameter. The adjacent stone material must be coarse enough to meet the requirements of a base material for the given filter of large uniformly sized stones. Stones of a diameter less than about 1/10 of the diameter of the blocks in the slope protection cover may be washed out through the openings between the blocks.

On reservoirs with small wind fetch surfaces the design wave height would be negligible, and possible ice forces become critical for the dimensioning of stones in the slope protection cover. Figure 6.5 shows the stone sizes required to resist ice forces according to slope inclination as indicated.

6.8 Slope Stability Analyses

A slide in soil materials often leaves a more or less distinct sliding (slip) surface. The body overlying the slip surface moves relative to the underlying material that remains in place. Figure 6.7 shows a typical surface for a slide in a relatively homogeneous earth slope. The sliding surface in this illustration is simplified to a circle segment in a vertical section through the slide. The mechanics of the limiting equilibrium situation for this case is very simple, and the main uncertainties appear in the determination of shear strength along the potential sliding surface.

The safety factor against sliding is commonly defined as the number by which the shear strength is divided in the equilibrium analysis. The probability of a slide occurring depends on the mean safety factor and on the statistical scatter (coefficient of variation) of the shear strength for points along the sliding surface. With a mean safety factor of 1.0, a slide is highly probable, whereas a factor of 1.3 to 1.5 for embankments of fairly well known properties (i.e. coefficient of variation small), represents a very low probability of failure.

$$W \cdot a = \frac{s}{F} \cdot L \cdot r$$

W	Weight of soil
a	Lever arm
s	Shear strenght along failure surface
L	Length of failure surface
r	Radius of failure circle
F	Factor of safety

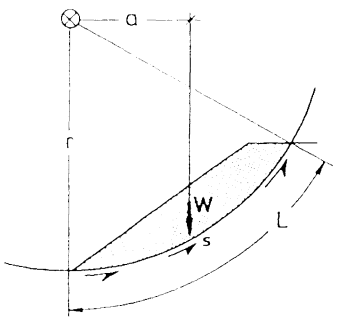


Fig. 6.7
Moment equilibrium assuming a circular sliding surface

Although the circular form is common for fairly homogeneous conditions, the sliding surface may take other forms according to circumstances. Janbu (1973) has given a comprehensive presentation of analyses for more general sliding surface configurations. Multilinear sliding planes should be studied, especially in a rockfill dam with distinct zones of different properties (Figure 6.8). Stability along multilinear sliding planes is analysed by means of force equilibrium equations, the so-called "sliding block" method (Figure 6.9).

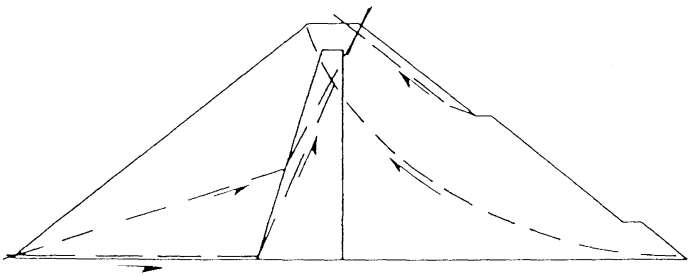


Fig. 6.8
Typical sliding surfaces used for calculation of the stability of a rockfill dam resting on bedrock

The shear strength along the different sliding surfaces are determined as described in Section 3.4. The strength parameters for the rockfill and filter, the core material and the rockfill-base interface must be determined as well as the effective normal stresses on the potential sliding surface. The latter requires estimates of total normal stresses and the pore water pressures at different points in the dam interior.

The stability of an embankment dam must therefore be analysed for several sliding planes in various dam cross-sections, and the numerical analyses are best carried out by means of special computer programs. Figure 6.9 shows potential sliding planes and corresponding "block" arrangements depending on the shear strength of the foundation as compared with that of the embankment. Stability is analysed for the highest of each type of cross-section along the dam axis.

The force vectors in the stability equilibrium equations result from material weight, water pressure as external load, pore water pressure on the sliding planes as internal loads, shear resistance along the sliding planes, and the effective normal forces on the sliding planes. The factor of safety is commonly defined as the number by which both c' and $\tan \phi'$, is divided in the limiting equilibrium analyses.

The external and internal forces for an embankment dam are not constant with time, and stability must be ascertained under various conditions. In practice computations are made for three design situations: end of construction, steady state seepage at full reservoir, and rapid drawdown of the reservoir level.

The end of construction and first impoundment is characterized by the transient pore pressures in the core not yet being adjusted to the impounded water regime. Pore pressures will rise with the increases in overburden load during the construction of the embankment. They will dissipate slowly in keeping with the consolidation properties and width of the core. The transient pore pressures are estimated on the basis of the anticipated embankment construction and reservoir filling rates using two-dimensional consolidation theory (Koppula and Morgenstern, 1982). The parameters needed for the computations, $\bar{B}$ and c_v, are determined from laboratory test results as described in Section 3.4.

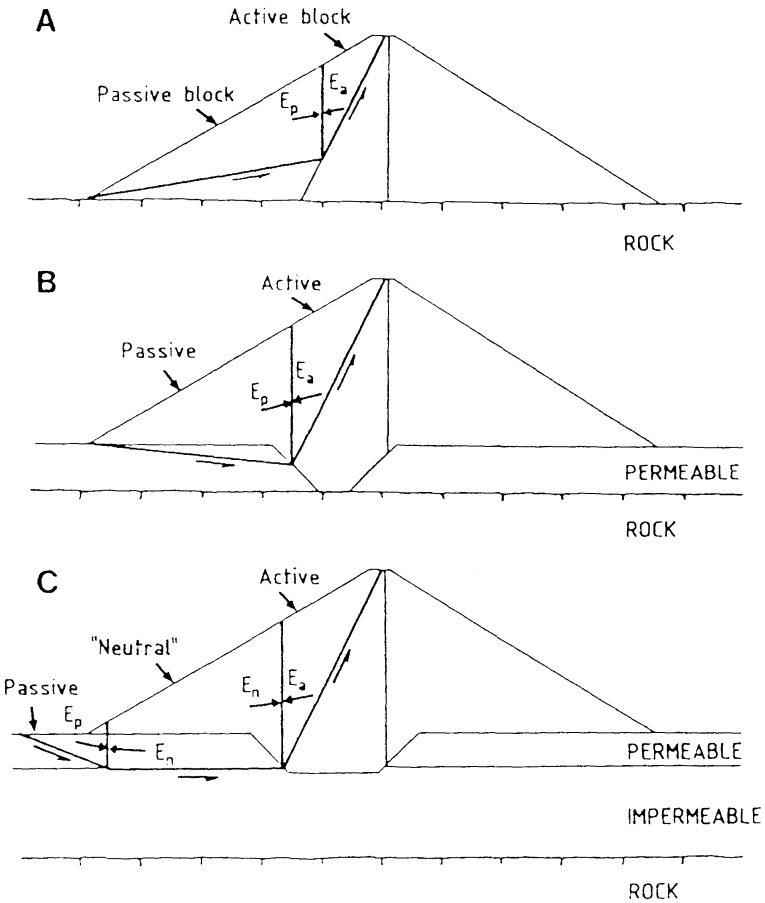


Fig. 6.9
Potential sliding planes and corresponding "sliding blocks" depending on the relative shear strength of foundation and embankment

A) Higher shear strength in the foundation

B) Higher shear strength in the embankment

C) Higher shear strength in supporting shoulder and permeable layer than in an impermeable layer of the foundation

External water pressure is introduced by using a value which corresponds to the water level at which stability is most critical. Internal pore pressures are estimated and may be monitored by piezometers installed during construction. If measurements indicate that too high pore pressure are developing, corrective measures must be taken. These measures may include slowing down the rate of construction, introducing relief drains, flattening outer slopes or reducing core width.

The condition of steady state seepage is said to exist when pore pressures at all locations are fully adjusted to the maximum water level regime. The pore water pressures are determined from flow net or finite element analyses with an external water head at the retension water level.

The condition of rapid drawdown is characterized by the transient pore pressure which results from a sudden drawdown of the reservoir from a maximum water level to a lower critical water level. This may give a critical stability situation. Pore pressures during the drawdown are set equal to the pore pressures for steady state seepage at maximum water level, less the changes in pore pressure due to reduced total overburden stresses. The external water pressure corresponds to the actual drawdown level.

Table 6.1 gives the safety requirements for rockfill dams in Norwegian practice. The table lists the various conditions for which stability analyses are required and the associated minimum factor of safety as defined above.

Under Norwegian design codes (limit state design), safety requirements are expressed through partial coefficients on load and material strength. For the types of stability analyses described above a load coefficient equal 1.0 is prescribed. The material coefficient therefore coincides numerically with the factors of safety listed in Table 6.1.

Table 6.1 Design conditions and required factor of safety against sliding

Design condition	Reservoir level	Sliding in slope	Minimum factor of safety
End of construction and first impoundment	Empty	Upstream	1.3
	Critical level (min. stability)	Downstream	1.5
Steady state seepage	High flood water level	Downstream	1.5
Rapid drawdown	Critical level (min. stability)	Upstream	1.3

6.9 Earthquake Effects

Norway is not in a very seismic region, and analyses have shown that the effects of probable earthquakes have not governed any aspects of the design of Norwegian rockfill dams (Valstad et al., 1991)

In regions subjected to stronger earthquakes, or for embankment dams and foundations more susceptible to earthquake shaking, stability and deformation analyses must include the effects of dynamic and cyclic (repetitive) loads. Due to the cyclic loading, excess pore water pressures may build up in poorly compacted fills or loose saturated foundations, leading to reduced shear strength, lower deformation moduli and the accumulation of perma-

nent (plastic) deformations (Andersen and Høeg, 1991). Strong earthquakes have in some cases caused complete liquefaction of foundations, slopes and embankments. However, compacted rockfill is inherently very resistant to earthquake shaking, and there is no record in the literature of damage to compacted rockfill dams caused by earthquakes.

Simplified stability analyses for dam embankments may be performed by introducing "statically equivalent" inertia forces (using seismic coefficients) into the equilibrium equations described in Section 6.8. The increased pore pressures induced by shaking, and any deterioration of the shear strength parameters, must be considered. It should be realized that the critical equilibrium stage may well occur some time after the earthquake shaking has ceased. Even if no instability and sliding take place, gradually accumulated displacements may have caused cracking to occur in the core. Analyses for such situations are presented in ICOLD (1986), Kuwano and Ishihara (1988) and Gazetas and Dakoulas (1992).

6.10 "Belt and Suspenders" Principle

The calculated safety factor or probability of failure is far from providing an unequivocal answer to the question of the overall safety of the dam. Theoretical calculations on their own can in no way offer a guarantee for the safety of the dam. This view was expressed so well by R. Peck in the 5th Laurits Bjerrum Memorial Lecture which he entitled "Where has all the judgment gone". An excerpt follows and concludes this chapter on design and analyses (Peck,1980):

..... a failure is seldom the consequence of a single shortcoming. Usually there is at least one other defect or deficiency, and the failure occurs where two or more coincide. This inference supports the principle of designing to provide defence in depth, the "belt and suspenders" principle long advocated by Arthur Casagrande. It postulates that if any defensive element in the dam or its foundation should fail to serve its function, there must be one or more additional defensive measures to take its place. Teton Dam is an outstanding example of violation of the principle; the sole line of defense was the core and grout curtain. Under such conditions, it is almost irrelevant to define the precise manner in which the failure started or whether there may have been such deficiencies in construction as a wet seam across the core. The design should have provided adequate defenses against any such defect.

The bedrock treatment appropriate to the geological conditions is a matter of design. It is not an aspect of design susceptible, however, to numerical analysis. Instead, it requires the exercise of judgment, a sense of proportion. When a jointed bedrock foundation is being treated and covered with the first layers of fill – a crucial time with respect to the future performance of the dam – engineers fully acquainted with the design requirements should be present, should have the authority to make decisions on the spot, and should not delegate their authority unless and until they are satisfied that their judgment concerning the particular project has been fully appreciated by their subordinates.

I doubt if guidelines, regulations; or even the best of specifications can take the place of personal interaction between designers and field forces at this stage. Frequently, as a consultant or as a member of a board of consultants, I have walked over a recently exposed foundation or abutment with the field forces, discussing place by place what treatment would be appropriate. Together we studied details of foundation conditions, observed treatment in progress, agreed upon or changed it. In this manner, the consultants evaluated the potential problems and their solutions, and the field forces gained the necessary insight as to what was required.

The type of foundation treatment is not a matter to be determined by a geologist, unless he is truly an engineering geologist. The geologist should investigate the geological characteristics of the foundation and its interface with the dam and make them known to the engineer who will form the judgment concerning treatment. The consequences of the flow of water near the interface, including its effect on the various materials in the dam, are within the realm of the engineer and the decisions regarding treatment are engineering decisions.

The three failures that I have discussed in some detail originated at the interface between dam and foundation. Others, that might equally have been chosen as examples, arose from overlooking or misjudging geological features in the foundation itself. A few had their origin in construction defects in the embankment. They had in common that all were outside the scope of numerical analyses. They would have been prevented if judgment arising from extensive experience had been given full scope in design and construction, and if the design had included multiple defenses. Accordingly, I think it quite possible that the incidence of failure of major dams could be reduced an order of magnitude by focusing attention on the details of design and construction that cannot at least presently be covered by analysis, that perhaps cannot be known until construction is underway, but that require the personal attention of experienced engineers. Research should be directed to those aspects of design and construction in greatest need of improvement; definition of foundation conditions; conditions conducive to internal erosion and the means for controlling it; filter criteria and their practical achievement in construction; prevention and treatment of cracking. The goal of the research should be improved understanding, preferably but not necessarily quantitative.

The literature already has much to say about cracking of earth dams. The emphasis, however, is on the mechanics of producing the initial cracks, an aspect that has recently become at least partly amenable to analysis. The analytical results serve a useful purpose: reduction of cracking can undoubtedly be achieved most successfully if the causes of cracking are understood and avoided. Nevertheless, to accord with the principle of defence in depth, every dam should be designed on the assumption that the core may crack and that the dam should be safe even if it does.

So we reckon with the conclusion that modern dams seldom if ever fail because of incorrect or inadequate numerical analyses. They fail because

inadequate judgment is brought to bear on the problems that, whether anticipated or not, arise in such places as the foundation or the interface between embankment and foundation. Sometimes they disclose themselves only in subtle ways in the use of the observational procedure. Irrespective of specifications, contractual arrangements, possibilities of claims for extras, or delays, these problems must be recognised and solved satisfactorily. If we regard them as second-rate judgment, failures of dams will continue at a probability of 10^{-4} per dam year. As long as the myth persists that only what can be calculated constitutes engineering, engineers will lack incentive or opportunity to apply the best judgment to the crucial problems that cannot be solved by calculation.

Where has the judgment gone? It has gone where the rewards of professional recognition and advancement are greatest – to the design office where the sheer beauty of analysis is often separated from reality. It has gone to the research institutions, into the fascinating effort to idealize the properties of real materials for the purposes of analysis and into the solution of intricate problems of stress distribution and deformation of the idealized materials. The incentive to make a professional reputation leads the best people in these directions.

From a probabilistic point of view, it is logical to assume a base level probability of failure of 10^{-4} per dam year. There is no reason, however, why engineers should be satisfied to consider such a failure rate as the norm. Dams should be designed and constructed not to fail, even if a probability of failure is incorporated into the benefit-cost analysis. Since we know wherein the greatest weaknesses lie, we should be able to devise the means for applying judgment to avoid these weaknesses. If we succeed, we should be able to justify a base level probability of failure no more than perhaps 10^{-5} per dam year. Such an improvement is now within the state of the art. Its achievement does not depend on the acquisition of new knowledge. It depends on our ability to bring the best engineering judgment to bear on problems that are essentially non-quantitative, having solutions that are essentially non-numerical. To develop this judgment and to bring it to bear requires a re-ordering of our present views of what constitutes the highest form of our practice of engineering. Without detracting from the necessity for reasonable and meaningful engineering calculations and from the rewards to those who can carry them out, at least equal professional prestige and responsibility should be accorded men of judgment, even when that judgment is not expressed in numerical form.

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