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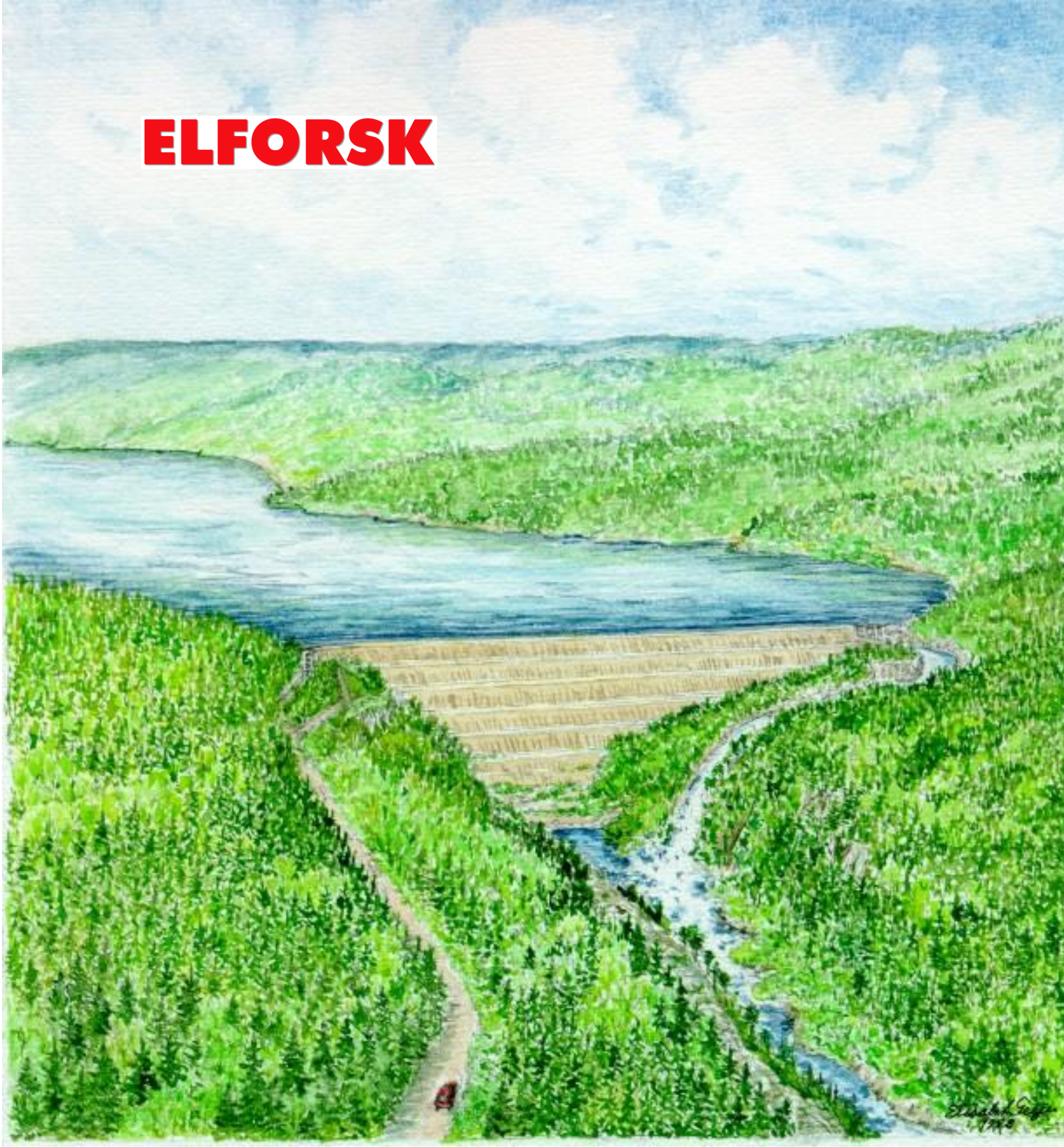

DAMMSÄKERHET

Experiences from Hydraulic Model Tests in Dam Rebuilding Projects

Rapport 10:95

Experiences from Hydraulic Model Tests in Dam Rebuilding Projects

Elforsk rapport 10:95

James Yang, Vattenfall R & D

April 2009

Förord

Denna rapport är ett delresultat inom ”Elforsks ramprogram dammsäkerhet”. Målen för programmet är att långsiktigt stödja branschens policy, dvs att:

- Sannolikheten för dammbrott där människoliv kan vara hotade skall hållas på en så låg nivå att detta hot såvitt möjligt elimineras.
- Konsekvenserna i händelse av dammbrott skall genom god planering såvitt möjligt reduceras.
- Dammsäkerheten skall hållas på en god internationell nivå.

Prioriterade områden är teknisk säkerhet, operativ säkerhet och beredskap samt riskanalys.

Ramprogrammet har en styrgrupp bestående av: Jonas Birkedahl – Fotum, Claes-Olof Brandesten - Vattenfall Vattenkraft, Lars Hammar - Vattenfall Vattenkraft, Anders Isander – E.ON, Martin Johansson – Skellefteå Kraft, Anders Sjödin - Statkraft, Gunnar Sjödin – Vattenregleringsföretagen, Rolf Steiner - Fortum samt Cristian Andersson – Elforsk. Adjungerade till gruppen är också Maria Bartsch, Anna Engström-Meyer och Olle Mill - Svenska Kraftnät.

Stockholm juli 2010

Cristian Andersson
Elforsk AB

Sammanfattning

Med anledning av Flödeskommitténs riktlinjer som ger upphov till högre dimensionerande vattenföringar samt mer strikta dammsäkerhetskrav betingade av RIDAS har ett stort antal dammar byggts om i landet och många andra undergår f.n. fördjupade dammsäkerhetsutvärderingar eller är under ombyggnad.

Under de senaste tio åren har cirka 20 dammanläggningar studerats i planmodeller vid Vattenfall R & D i Älvkarleby, med huvudsyfte att undersöka och utvärdera föreslagna dammsäkerhetshöjande åtgärder. I föreliggande rapport har de under 1998–2008 utförda modellförsöken dokumenterats, och erfarenheter och frågeställningar av intresse kategoriserats.

I samband med ombyggnaden av en damm i fråga kan hydrauliska frågeställningar av angelägenhet beröra erosion i dammslänten uppströms som gränsar till utskov, avbördningsförmåga och överdämning, tillbyggnad av nytt utskov eller modifikationer av befintligt utskov genom tröskelsänkning eller från botten- till överfallsutskov, justeringar av utskovskanaler, anläggning av ny eller ändringar i befintlig energiomvandlare och undersökning av risk för erosion i dammtån och nedströmsliggande älvfåra.

Summary

In the light of the revised design floods and higher dam-safety requirements, many dams in Sweden have been rebuilt and many others are undergoing dam-safety evaluations or an upgrading process.

At Vattenfall R&D, Älvkarleby, about 20 dams have been tested during the past years for suitability of proposed rebuilding measures. In this report, the hydraulic model tests conducted during 1998–2008 are summarized; experiences and hydraulic issues of attention are discussed.

Depending upon the dam in question, hydraulic concerns arising from the dam rebuilding may cover dam-slope erosion close to the spillway, spillway discharge capacity, addition of new spillway or rebuilding of existing spillway by lowering threshold elevation or modifications of bottom outlet to overflow-type spillway, re-shaping spillway channel, enlargement of stilling basin or plunge pool and risk for erosion in the dam toe or in the river channels downstream.

Table of contents

Dams included in the report

As listed in alphabetical order

No	Name of dam	Owner
1	Ajaure	Vattenfall
2	Bergeforsen	
3	Gallejaur	
4	Halvfari	Fortum
5	Harsprånget	Vattenfall
6	Höljes	Fortum
7	Laxede	Vattenfall
8	Letsi	
9	Ligga	
10	Långbjörn	
11	Midskog	
12	Porsi	
13	Rusfors	
14	Satisjaure	
15	Stenkullafors	
16	Storfinnforsen	E.ON
17	Stornorrfors	Vattenfall

As listed per river

River	Name of dams
Ume älv	Ajaure, Rusfors, Stornorrfors
Lule älv	Laxede, Letsi, Ligga, Harsprånget, Porsi, Satisjaure
Ångermanälven	Långbjörn, Stenkullafors, Storfinnforsen
Indalsälven	Bergeforsen, Midskog
Skellefte älv	Gallejaur
Klarälven	Höljes
Ljusnan	Halvfari

Acknowledgements

During the past ten years, about 20 hydraulic models of existing dams and spillways have been built for engineering solutions towards higher dam safety level. In one way or another, many individuals have been involved in and contributed to the accomplishment of the hydraulic model investigations. A partial list is given below.

Vattenfall Vattenkraft	Malte Cederström, Urban Norstedt, Erik Nordström, Peter Viklander, Lars Hammar (before at Elforsk), Niklas Dahlbäck, Claes-Olof Brandesten, Kjell-Åke Wallin, Leif Ask, Stefan Berntsson, Jan Mikaelsson (retired)
E.ON	Carl-Oscar Nilsson, Peter Mattiasson, Anders Isander
Fortum	Rolf Steiner, Gunnar Henriksson, Karl-Erik Löwén, Anders Sjödin (now at Statkraft), Juha Laasonen
WSP	Petter Stenström, Håkan Bond, Anders Halvarsson, Patrik Andersson, Andreas Halvarsson (now at Grontmij)
VPC	Nils Johansson, Karin Hellstadius, Mats Stenmark, Ingvar Ekström (now at Sweco), Åke Nilsson, Anders Nyström
SWECO	Anders Gustafsson, Jörgen Dath, Hans Eriksson, Sofia Lindgren
Sekond	Karl Rytters
HydroTerra	Åke Engström

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Disclaimer

All the design floods of dams and their spillway discharge capacity given in the report refer to approximate data. The reader should resort to the dam owner in question if he or she is interested in the exact figures or related reports from the hydraulic model tests. As a policy, Vattenfall R&D will not distribute any material without consent of its client.

Unless otherwise specified, majority of the photographs inserted in the report were taken either by the author, Vattenfall R&D or Vattenfall Vattenkraft. However, the origin of some pictures can't be traced.

The model tests are stated in either past or present tense, which is mainly due to the citation of previously used documents.

The opinions expressed herein are only those of the author and don't necessarily represent any involved organization's. Any use or interpretation of the materials presented in this report lies in the reader's responsibility.

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May 2009

1 Introduction

1.1 Dam upgrading background

In Sweden, there exist approximately 700 hydropower dams of varying ages. Classified by the ICOLD standard, 142 of them are high dams (>15 m), about 80% of which are of embankment type.

When the dams were constructed, there were no well-established criteria for determination of their design floods. The design flood of a dam was usually based on multiplication of the observed highest historic flood by a safety factor. This corresponded often to 100-year floods or somewhat higher. In certain cases, the frequency analysis method was deployed for extrapolation to 1000-year floods.

In the past few decades, the operation of the hydropower dams has indicated that the method was not up to date and new criteria for design flood have therefore been developed. The new guidelines of design floods were released in 1990. They are based on hydrological modeling rather than frequency analysis and emphasize critical timing of the flood generating factors. The design flood for a dam is obtained through transformation of extreme climatological and hydrological conditions using a conceptual hydrological model. The observed maximum areal rainfall is, on a trial-and-error basis, combined with extreme snowmelt, soil moisture and reservoir operation until the worst flood for the catchment in question is found (Flödeskommittén, 1990).

During the past years, the hydropower sector has conducted extensive investigations for many catchments and dams; the main purpose is to make control calculations of the design floods. The results show that the present guidelines result generally in higher design floods and many existing spillways have to be redesigned for a higher discharge capacity. Either the reservoir storage volume or the spillway discharge capacity or both must increase. Of course an increased water level during a high flood will increase the discharge at this time.

With the guidelines in the background, the hydropower sector has also worked out the guidelines on dam safety (so-called RIDAS) that govern the dam-safety practice in the country. It is stipulated that dams should be constructed and maintained in such a way that a high level of safety is guaranteed. Depending on the incremental losses in case of hypothetical dam failure, dams are classified into two risk categories, i.e. high-hazard (class I) and low-hazard (class II). All risk class-I and even some class-II dams must be designed to discharge the design floods. In general the design flood is lower for class-II dams.

For each dam to be rebuilt, safety evaluations are made, similar to the SEED used in USA. Through the studies, all weaknesses of the dam are focused on. Usually, the spillway is tested by opening its gates as much as is allowed by the operation permit for the dam in order to identify potential problems and risks, especially in the downstream area. Geotechnical investigations of the dam body and its foundation are conducted if necessary. Previous incident reports from dam operation should also be reviewed.

Dams with obvious defects and with high consequences are given priority in the upgrading process. Concerning the sequence of refurbishment for a river with several dams, one should take into consideration such factors as the possibility of flood occurrence during construction and losses in power production due to construction work. Computer modelling can be used for the purpose. A dam is rebuilt to cope with the design flood and also to remedy the existing defects. Damages on the dam are allowed during the extreme flood, but they should absolutely not endanger the safety of the dam.

Several rebuilding alternatives are usually compared. In many cases, it is the rebuilding costs in combination with safety aspects that govern the choice of a technical solution. However, considerations are also given to environmental and social impacts. Alternative rebuilding measures include raise in the impervious core and dam crest, modification of spillway, reinforcement of dam foundation and strengthening of waterways for spillway discharge.

The erosion protection of the upstream slope of a dam is usually upgraded. On the downstream side, a toe berm is often added to make sure that a rock-fill dam will be stable even if there is a rather high leakage through the dam. Often, the stilling basin is rebuilt due to the larger flood magnitude. The dam should be protected against retrogressive erosion and the risk for erosion further downstream is also avoided.

Depending on the nature of the rebuilding, permission is sometimes needed from the Environmental Court. The refurbishment needed for a river basin, including safety evaluations, engineering design, court proceedings, construction and other measures, will take many years to accomplish.

It should be pointed out that the refurbishment going on in Sweden aims only at raising the safety level of its dams. Its purpose is not to increase the power production.

1.2 Physical model testing

The Hydraulic Laboratory at Vattenfall Research & Development, Älvkarleby, has a long tradition in doing physical model tests for hydropower stations and dams, Figure 1.1. The earliest hydraulic model testing dates back from 1943-44 when the hydroelectric development program in Sweden was stepped up. The laboratory from 1943 was extended several times in the 1950s. The existing building was from 1953.

The hydraulic laboratory, excluding laboratories for nuclear, turbine, material and petroleum research, has up to 2000 m^2 testing space and is equipped with sufficient pumping capacity and test facilities for diversified kinds of tests inclusive of other industrial applications, Figure 1.2.

Figure 1.1 Building of Vattenfall R&D, Älvkarleby, in which the Hydraulic Laboratory is housed (Alf Linderheim)

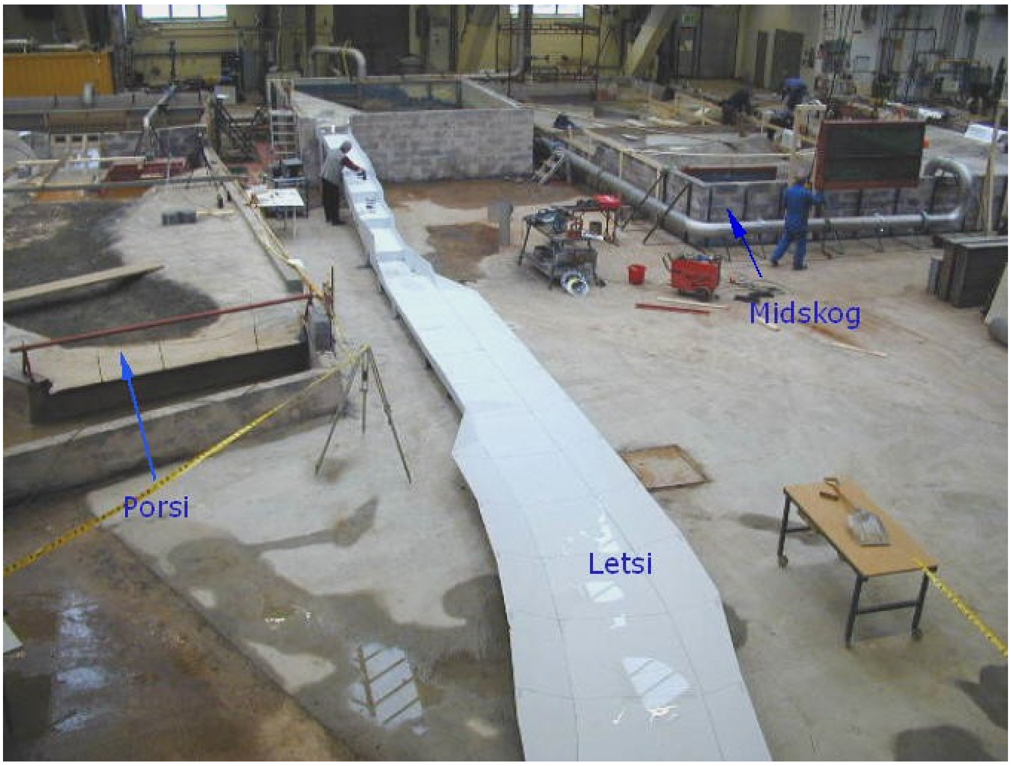


Figure 1.2a View of Vattenfall's Hydraulic laboratory, January 2002

Figure 1.2b View of Vattenfall's Hydraulic laboratory, November 2007

Hydraulic model tests, with the purpose of serving to evaluate engineering solutions of dam upgrading and rebuilding, date back to winter 1997/98, when the Bergeforsen dam on the lower reaches of Indalsälven, owned by Vattenfall, was first built. The model for Ajaure was constructed spring 1998, immediately after Bergeforsen. In the years that followed, models of about 20 dams have been built and tested here.

Due to the complexity of the waterway geometry, combined with complex flow phenomena with both sub- and super, hydraulic model studies were often requisite. The majority of dams and power stations were constructed after model tests. The purpose was to examine the function of proposed rebuilding measures for spillway floods up to a level somewhat higher than the new design flood, so that potential damages in the dam body and downstream that could jeopardize the dam safety were avoided in extreme flood situations.

It can be said that hydraulic model testing aimed at *verification and confirmation of a design*, *solving already known problems*, and *improvement and optimization of the design*. Perhaps the most important for certain problem categories was to *identify and avoid hidden undesired problems*, issues like unfavorable flow pattern, occurrence of vortex or sediment movement leading to deposition that a designer failed to foresee.

In Table 1.1, the dams that were examined in hydraulic models during 1998-2008 are listed. Most of the models were constructed in scale 1:30, 1:40, 1:50 or 1:60, with the exception of Långbjörn and Stornorrfors (1:100).

Documented in the report are often several technically feasible or potential rebuilding proposals examined for a dam om be upgraded. The final adopted engineering measure might be one of those tested or modified, in one way or another, from the tested proposals.

Table 1.1 Descriptions of conducted hydraulic model tests during 1998-2008

No	Name	Scale	Model building time	Model testing time	Test executor
1	Ajaure	1:50	1998	1998-2001	Nils Johansson, James Yang, Hans Persson, Lars Svensson
2	Bergeforsen	1:50	1997-98	1998-2001	James Yang, Nils Johansson, Mats Billstein, Hans Persson
3	Gallejaur	1:40	2003-04	2004	James Yang, Gösta Amnell, Ulf Aurosell, Peter Skärberg
4	Halvfari	1:40	2006	2006-07	Gösta Amnell, James Yang, Per Larsson, Dean McGowan
5	Harsprånget	1:60	2003	2003	Mats Billstein, Gösta Amnell
6	Höljes	1:50	2008	2008	James Yang, Jonas Persson, Peter Skärberg
7	Laxede	1:60	2006	2006-07	James Yang, Gösta Amnell, Peter Skärberg, Sara Bodén
8	Letsi	1:50	2001-02	2002	James Yang, Hans Persson, Gösta Amnell, Lennart Svensson, Jenny Jungstedt
9	Ligga	1:50	2005	2005-06	James Yang, Peter Skärberg, Gösta Amnell, Ulf Aurosell
10	Långbjörn	1:100	2006	2006	James Yang, Gösta Amnell, Peter Skärberg
11	Midskog	1:50	2001-02	2002	Mats Billstein
12	Porsi	1:50	2001	2002-03	James Yang, Hans Lindqvist, Lennart Svensson, Caroline Göthlin
13	Rusfors	1:30	2006	2006	Per Larsson, James Yang
14	Satisjaure	1:50	2002	2002-03	Mats Billstein
15	Stenkullafors	1:50	2002	2002-03	James Yang, Gösta Amnell
16	Storfinnforsen	1:30	2007	2007-08	James Yang, Gösta Amnell, Per Sunqvist
17	Stornorrfors	1:100	2003	2003-05	James Yang, Gösta Amnell

2 Ajaure

Ajaure, located on the upper reaches of Ume älv, is an embankment dam with a max. height of 45 m and an active reservoir storage of 200x10^6 m^3. Its layout is shown in Figure 2.1. The power station has one unit, utilizing a head of 45 – 58 m. The normal turbine flow is 150 m^3/s and the installed capacity is 85 MW.

The original spillway had two bottom outlets, each having a radial gate and an opening of 5.0 (width) by 10.4 m (height). The total discharge capacity at the full retention reservoir level (FRRL) +440.5 m was ~950 m^3/s. The water from the spillway is conveyed to the downstream river valley in a channel of some 150 m in length. The channel is not straight, but bends to the left. In the upper part of the channel, a partition wall exists in the middle, Figure 2.2.

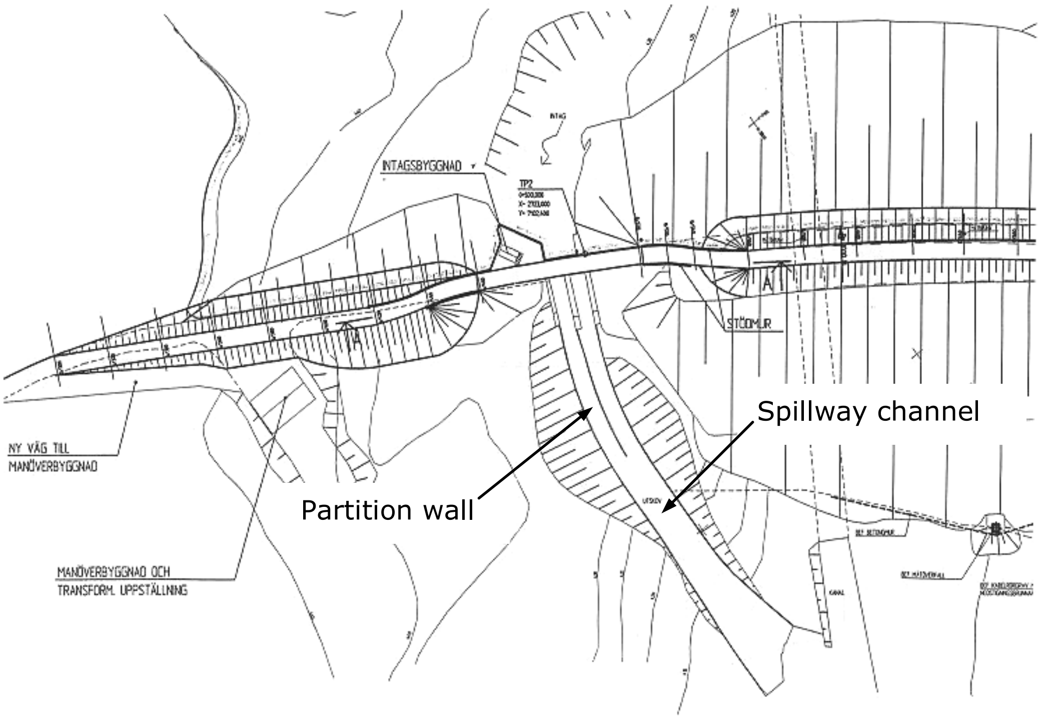


Figure 2.1 Ajaure dam layout. The spillway is placed between the left embankment dam and the intake to powerhouse.

With the new design-flood criteria, the spillway capacity needed to be increased from 1000 to 1300-1400 m^3/s. With the existing bottom spillway that had limited increase in the capacity with water level, the reservoir level would rise by up to 8 m above the FRRL and the dam crest would be overtopped. Extensive investigations were made to finalize how the dam would be refurbished, which depended to great extent on the discharge capacity of the spillway. If the spillway capacity was higher, the requisite increase in the dam height would be lower, implying lower costs for the rebuilding of the dam.

Figure 2.2 Spillway after rebuilding of left outlet to overflow spillway

2.1 Hydraulic issues

A hydraulic model was built for Ajaure with the bottom spillway and its discharge channel. The model, based on the Froudes law, had a scale of 1:50. Figure 2.3 shows the model seen from downstream. The reservoir 200 m upstream the dam was built with real river topography, so that the approaching flow to the spillway was correctly reproduced. The spillway was built in Plexiglas and the channel in 2-mm steel plate.

The model was first built with the existing bottom spillway; the purpose was to determine its discharge capacity and to understand the flow behavior in the channel before any modification was made

The issues of concern for Ajaure were as follows:

- To determine the existing discharge capacity of the bottom spillway and reservoir water level at the design flood
- To compare channel flow observations from 1997 flood release with the model tests
- To study flow behaviors in the channel at spillway discharges up to the design flood and to determine requisite channel wall heights with the existing spillway

- To modify the left bottom outlet to overflow type spillway and test the discharge capacity
- To determine requisite channel wall heights after the modification. Transition between the middle spillway pier and the partition wall is streamlined with a wedge to avoid unfavorable disturbance to the flow.
- To measure forces acting both sides of the partition wall, so that the load difference could be determined for stability analysis
- To compare channel water stages from the June 2000 flood release with the model test results after the left bottom outlet had been rebuilt in the prototype
- To estimate the effects of aeration at spillway piers and in the channel on channel wall heights
- To control if different combinations of left and right gate openings would require higher channel walls
- To work out spillway discharge capacity at different gate openings
- To rebuild both bottom outlets to overflow type spillway and to determine discharge capacity and channel water stages (requirement of even higher capacity in the future, a hypothetical case with the climate change in the background)

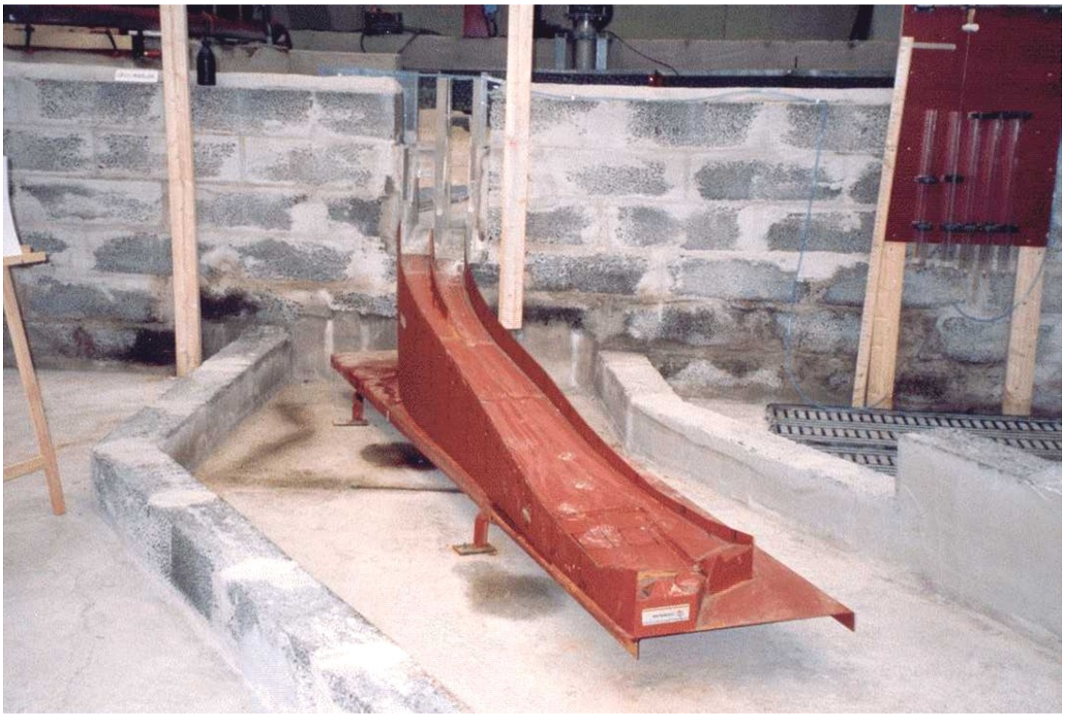
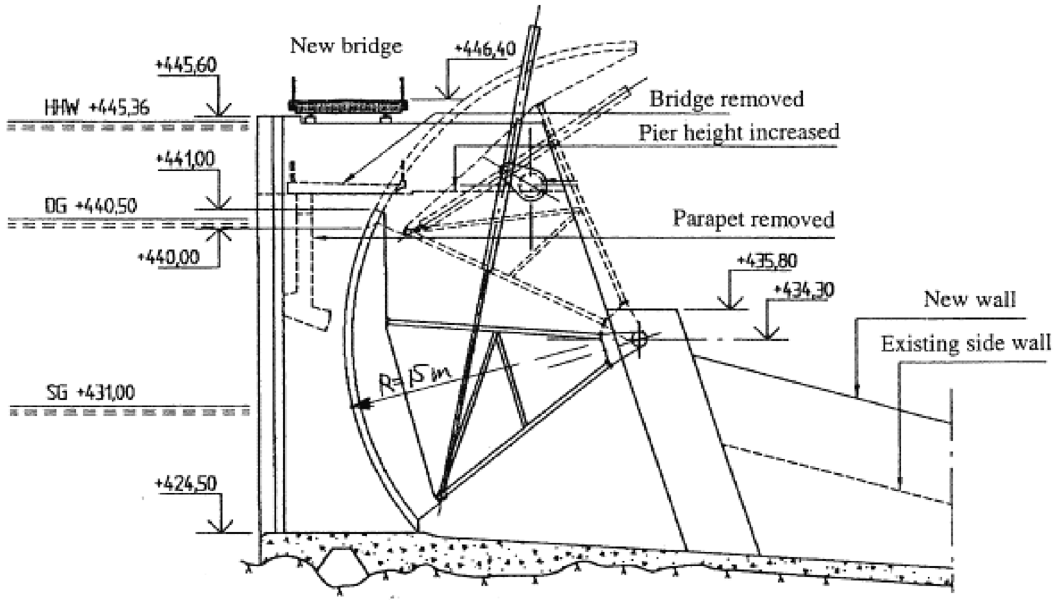


Figure 2.3 Ajaure model seen from downstream, scale 1:50

2.2 Modification of bottom outlet

It was suggested that one of the two bottom outlets, i.e. the left outlet, was modified to an open spillway (gated), Figure 2.4. In so doing, the reservoir water stage would increase much slower with increasing inflow discharge. The main reason why the left bottom outlet was chosen was that the radial gate in the right outlet had been, due to leakage, renovated a couple of years ago. The modification implied that the parapet wall in the left outlet was removed; the existing radial gate was replaced, and a larger, new one was installed. The sill elevation and width of the opening remained however unchanged.



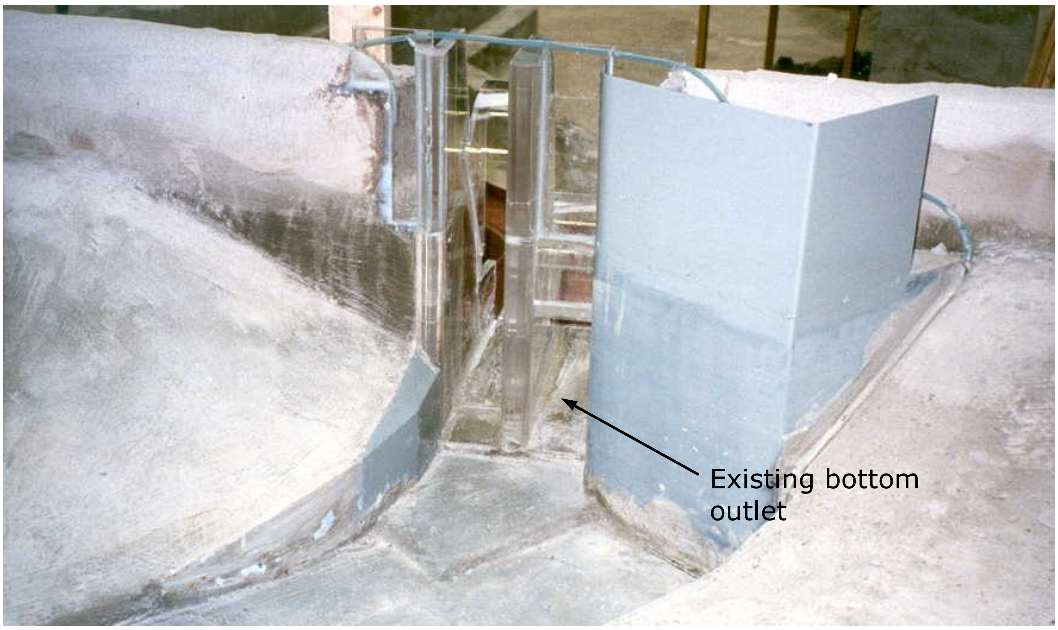


Figure 2.4 Modification of left bottom outlet to gated open spillway

Figure 2.5 shows the difference in the discharge, Q, through the modified overflow opening and right bottom outlet when they are opened separately. The lower edge of the parapet was at +434.9 m. If the reservoir level is higher than +437.0 m, the outlet becomes pressurized and the Q–Z relationship starts to differ. Obviously, the overflow opening contributes significantly to the reduction of the reservoir water level.

Figure 2.6 gives a comparison of the water level before and after the modification. The reduction is by e.g. 3.7 m at the design flood ~1350 m^3/s.

The figure illustrates also the result of a hypothetical situation (lower curve) where both bottom outlets are rebuilt into the overflow type, Figure 2.7. An additional decrease in the water level can be achieved, by e.g. about 2 m at ~1350 m^3/s. In Case B, the radial gates are opened high enough, so that they do not affect the discharge and the flow through the spillway has a free surface. In Case A, the maximum gate opening is limited by practical constraints and the spillway acts again as two bottom outlets at higher discharges than the discharge ~1400 m^3/s.

The parapet wall in the left bottom outlet was already removed and the outlet was converted to an overflow spillway and a new, higher radial gate with the same width was installed. Figure 2.1 shows the spillway after the modification.

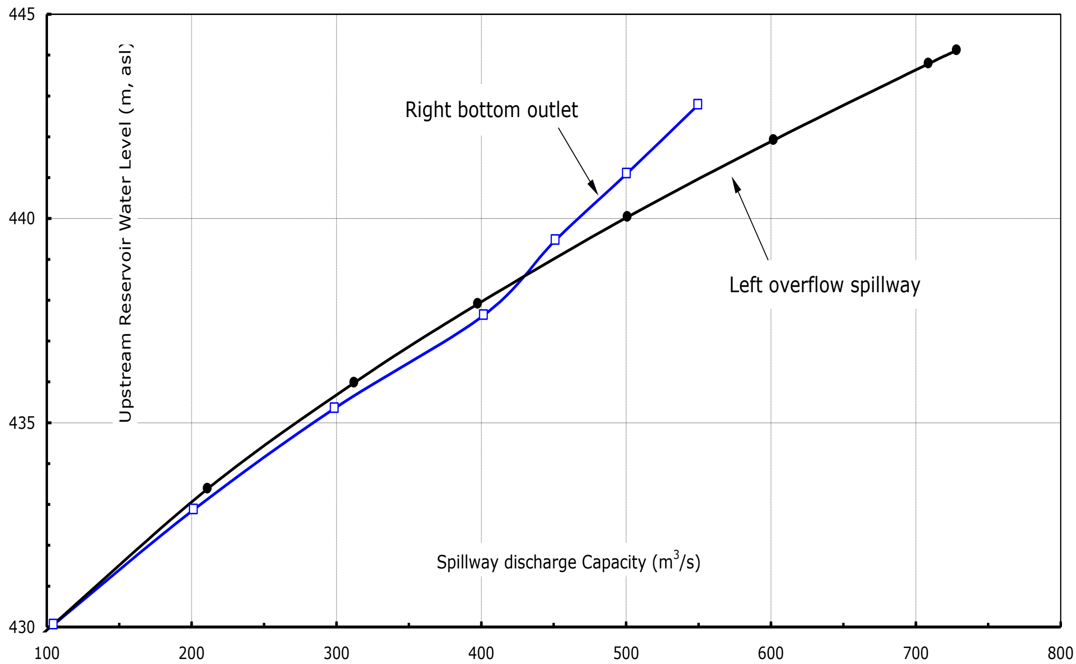


Figure 2.5 Q–Z relationship of left overflow spillway and right bottom outlet

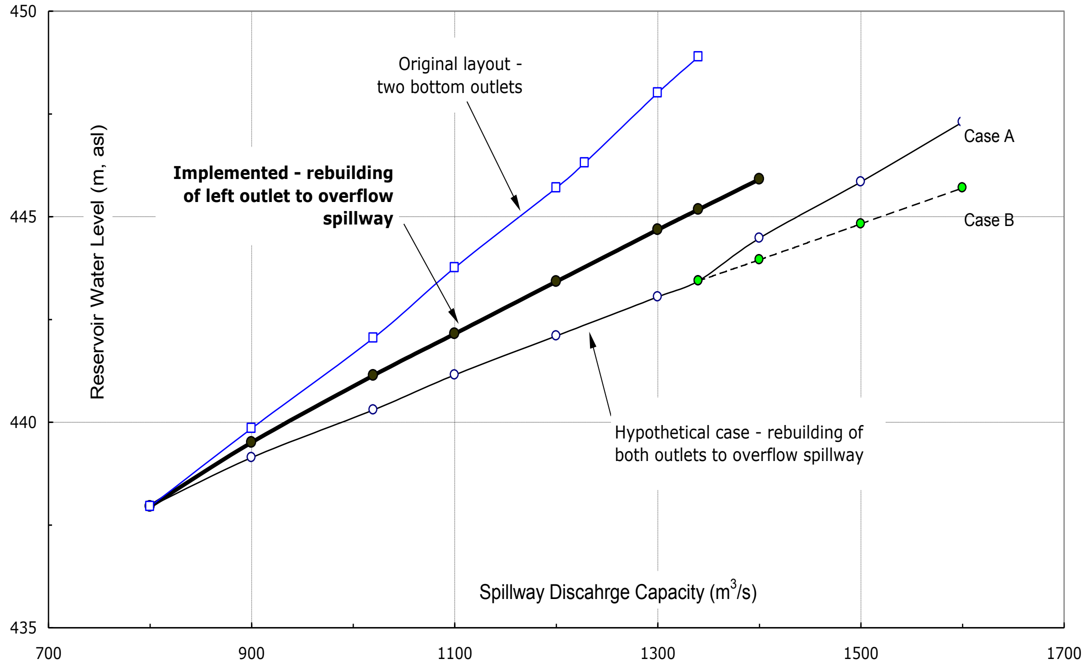


Figure 2.6 Rebuilding of left and both outlets to overflow spillway

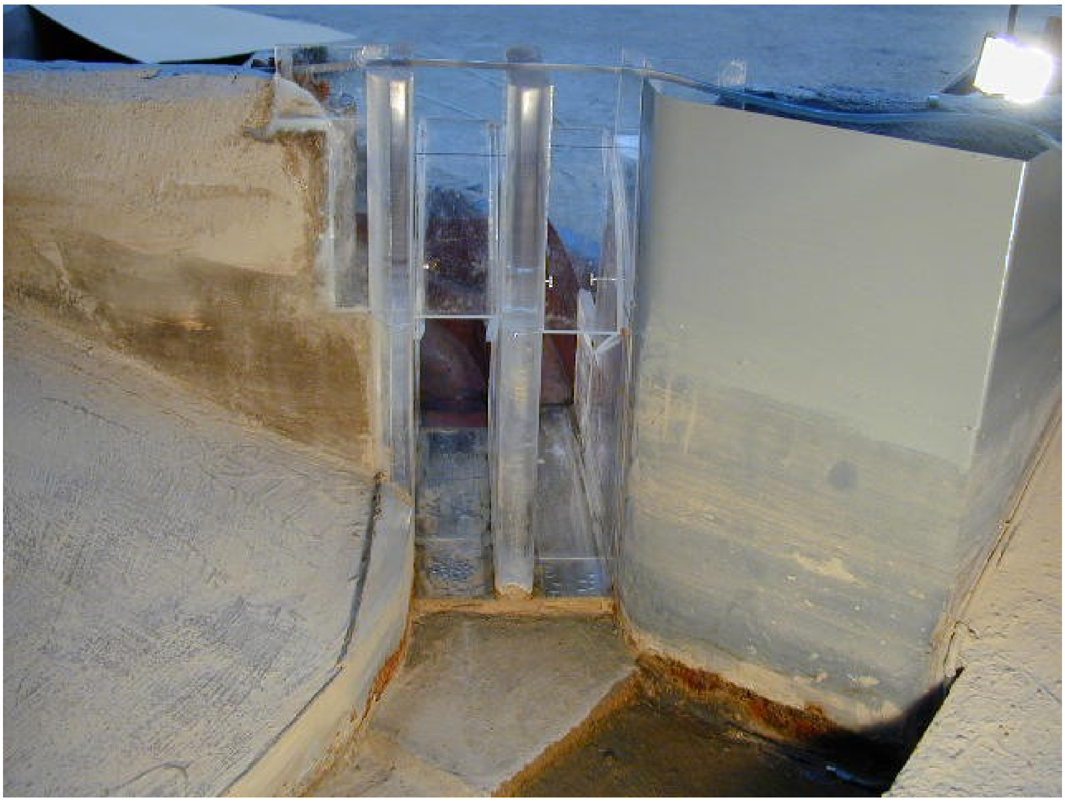


Figure 2.7 Hypothetical situation where both bottom outlets are rebuilt into the overflow type

2.3 Requisite channel height

The floodwater from the spillway is discharged to the downstream river valley in a channel, originally designed for a flood discharge of 900–1000 m^3/s. With the new design discharge, the channel needed to be re-dimensioned, so that the flood could be released without overtopping and damaging the dam body.

The channel is roughly 150 m long. It is typically 12 m wide and becomes wider downstream. The channel is described in terms of cross-sections running from 0/190 to 0/340 (from up- to downstream). Due to turbulence and aeration, the channel flow was characterized by strong unsteadiness. The measured water level referred therefore to the max. level taking into account the unsteadiness of the spillway flow.

In the upper part of the channel, the pier between the two spillway openings was follows by a low wall that separates the flow from them. The transition was however abrupt. By simply streamlining the transition, the flow condition was greatly improved. The water surface profile along the channel became more uniform and the requisite sidewall height was reduced.

Figure 2.8 and 2.9 show the requisite height of the left and right channel sidewalls at the design flood discharge ~1350 m^3/s. For the left sidewall, the sidewall needed to be increased substantially between section 0/250 and 0/300. The maximum increase was 4.80 m and occurs at section 0/280. For the right side, the whole wall had to be heightened. The maximum additional increase was 5.20 m at section 0/220.

By examining the flow pattern, one could easily find that the requisite height for the left and right side walls was closely related. Due to the fact that the channel bended to the left and the separating wall ended at section 0/238, the water from the right outlet hit first the right side wall between sections 0/210–0/240, the water was then reflected to the left wall between sections 0/260–0/290.

The requisite increase given here corresponded to the max. water-surface profile in the channel. For practical design purposes, one should take into account the effect of aeration, which was not correctly reproduced in the model. Usually, more safety marginal was required further downstream.

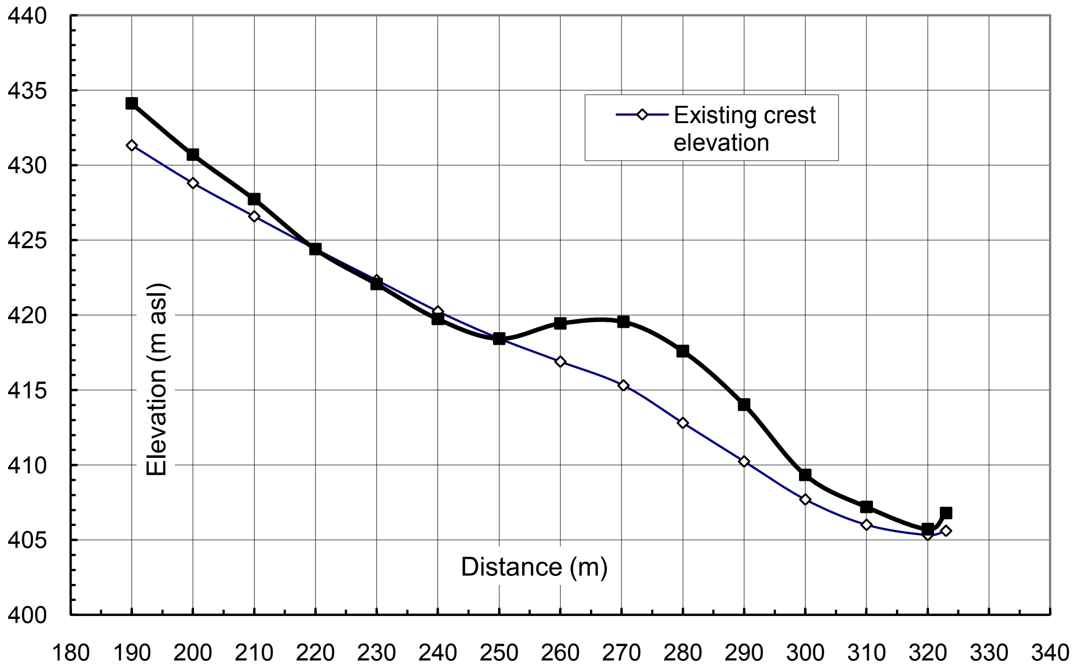


Figure 2.8 Requisite height of left sidewall at spillway discharge 1350 m^3/s

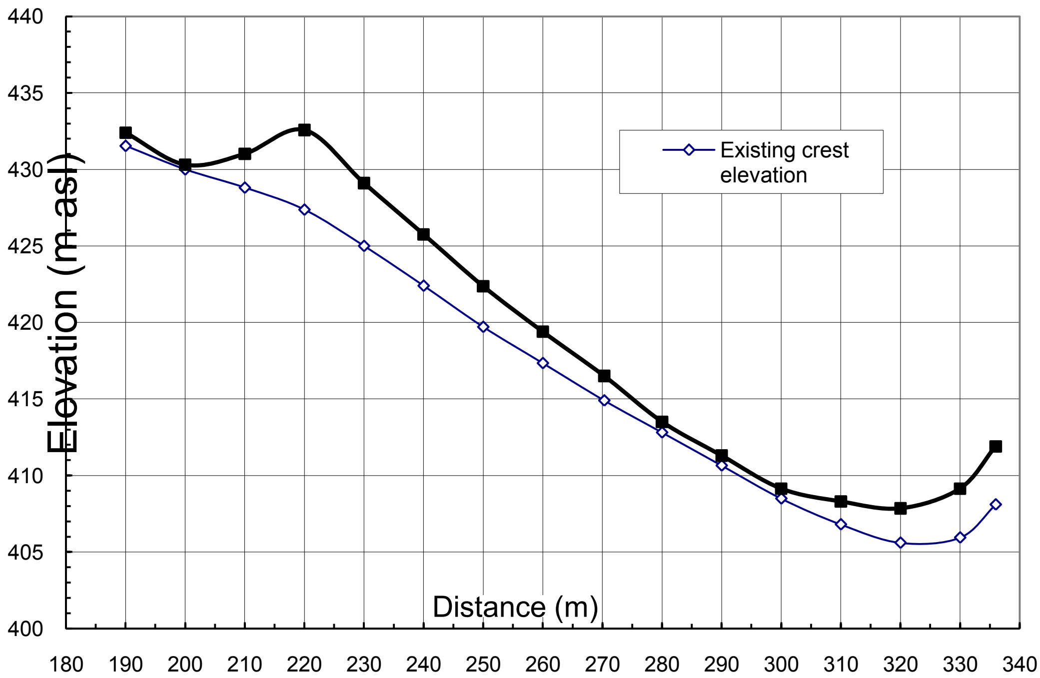


Figure 2.9 Requisite height of right side wall at spillway flow 1350 m^3/s

The following publications deal with the model tests for Ajaure.

Yang, J. (1998), Ajaure dam – physical model studies of bottom outlet and discharge channel (*Ajaure kraftstation – fysiska modellförsök för bottenutskov och avloppskanal*), Report No. US 98:5.

Yang, J. (1998), Ajaure dam – hydraulic model studies of spillway discharge capacity (*Ajaure kraftstation – modellförsök för bestämning av avbördningsförmåga*), Report No. US 98:29.

Yang, J. (1999), Ajaure dam – modification of bottom outlet and model tests of discharge capacity (*Ajaure kraftstation – ombyggnad av bottenutskov och modellförsök med avbördningsförmåga),* Report No. US 99:4.

Yang, J. (1999), Ajaure power station – design of spillway channel after rebuilding of bottom outlet (*Ajaure kraftstation – dimensionering av avloppsränna efter ombyggnad av bottenutskov*), Report No. US 99:6.

Yang, J. (2000), Refurbishment of spillway at Ajaure – design of spillway channel and tests of spillway capacity (*Ombyggnad av Ajaures utskov – dimensionering av utskovskanal & avbördning vid olika lucköppningar*), Report No. US 00:06.

Yang, J. (2001), Ajaure spillway channel – design bases at design flood (*Ajaures utskovskanal – dimensioneringsunderlag vid dimensionerande flöde*), Report No. U 01:03.

Yang, J. (2001), Ajaure hydropower scheme – Design bases for rebuilding to two overflow spillways (*Ajaure kraftstation - Projekteringsunderlag för ombyggnad till två ytutskov*), Report No. U 01:33.

Yang, J, Johansson, N, Stenmark, M & Gunnarsson, H (1999), "Upgrading of Spillway Channel for Ajaure Power Plant", Uprating & Refurbishing Hydro Powerplants, May 1999, Berlin.

Yang, J, Dahlbäck, N & Johansson, N (2001), "The Ajaure Dam – Spillway Refurbishment for Increased Design Flood", XXIX IAHR Congress, Sept. 2001, Beijing.

Yang, J, Johansson, N & Cederström, M (2002), "Towards Safer Dams – Refurbishment Examples in Vattenfall's Dam-Safety Program", HydroVision 2002, July/Aug. 2002, Portland, OR.

Yang, J & Cederström, M (2007), "Modification of spillways for higher discharge Capacity", Journal of Hydraulic Research, Vol. 45, No. 5, 2007.

3 Bergeforsen

Bergeforsen, built in 1955-59, is located on the lower Indalsälven. The complex consists of left and right earth-fill dam, spillway section, timber flume intake (plugged), powerhouse and tailrace channel, see Figure 3.1 and 3.2.

The earth-fill dam, built with impervious moraine core, has a crest length of 410 m and a maximal height of 29 m. The power plant is equipped with four units with Kaplan turbines, the total installed capacity being 155 MW. Three overflow spillways, with a total width of 45 m, are designed to discharge 2300 m^3/s at the FRRL (+23.0 m).

Based on the new guidelines, the design flood for the dam has to been raised from 2300 to ~3500 m^3/s, an increase in magnitude by almost 50%. To finalize the refurbishment program, extensive studies have been carried out during the past few years. Included in the studies are hydraulic and structural investigations, economical analyses and environmental considerations.

The hydraulic model testing for the power station was previously made around 1954–55, the purpose of which was to study the layout of the dam, including the design of the spillway and the timber flume. At that time, there was no strict procedure to follow for documentation. What can be found today is barely limited results of several pages from the model tests, which is inadequate as far as the planned dam refurbishment is concerned.

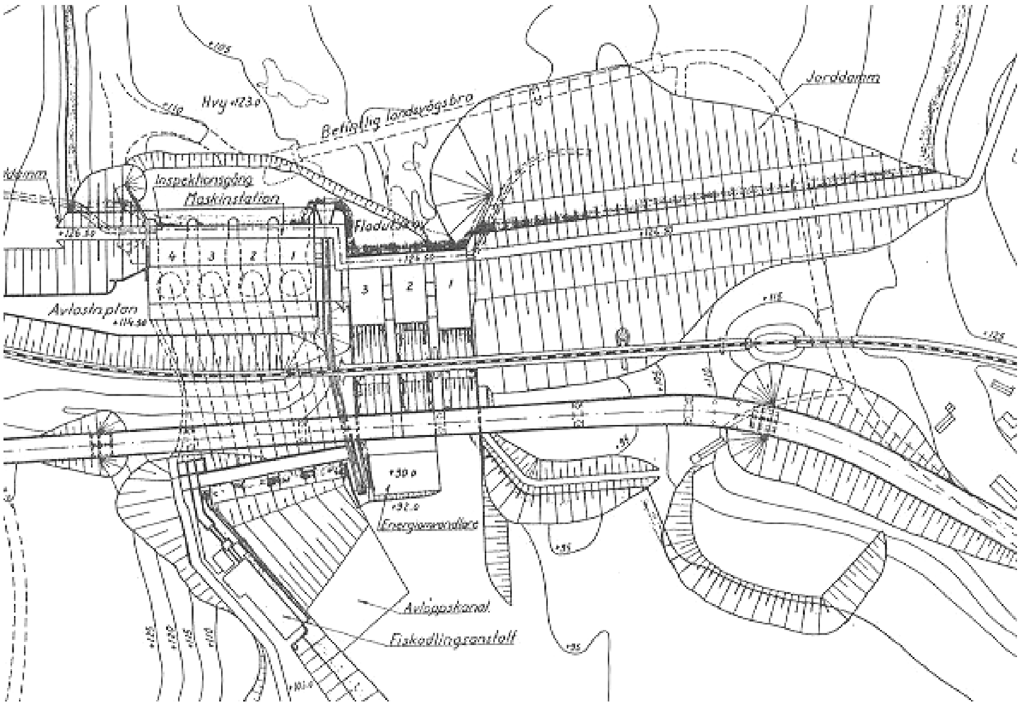


Figure 3.1 Bergeforsen dam layout

Figure 3.2 Bergeforsen dam and spillway

In connection with the re-building of the dam, a number of hydraulic issues need to be clarified. Some of them constitute the basis for the ongoing structural investigations of the dam. The studies are mainly performed through hydraulic model tests. A complete model of the dam complex is thus built, Figure 3.3.

Figure 3.3 Model of Bergeforsen dam, seen from upstream

3.1 Model setup

The hydraulic model was built in scale 1:50 and was 12 m wide and 25 m long, representing the river valley 350 m upstream and 875 m downstream of the dam. The model was considered long enough to correctly reproduce the approach flow conditions upstream and to examine the erosion and flooding problems downstream. The river valley was built with correct topography from echo sounding.

The spillway was constructed with Plexiglas and the intake to the power station with sheet metal. The existing spillway was built in such a way that it could easily be replaced with a new design. The flow to each of the four turbines was controlled with valve, so that the exact flow rate can be tuned in. The railway and highway downstream the dam, as well as the fish-breeding station, were constructed with correct geometry, as the risk for erosion and flooding they were subjected to at high floods should be taken into consideration in the rebuilding of the dam.

The dam is located some 10 km from the Gulf of Bothnia, where the observed water-stage variation during the period 1898–1992 is 217 cm. The water level and flow velocity downstream of the dam are affected by the variation. In order to study downstream erosion and flooding risk and dredging effect, the water level at the downstream boundary of the model should be correctly set at different sea levels and flow discharges. For this reason, 2D numerical simulations were made for the river reach between the dam and the Gulf.

The characteristics of the existing layout were first re-produced in the model, which formed the basis for different rebuilding alternatives of the dam. Included in the tests were investigations of the maximum discharge capacity, capacity at different gate openings, water-surface profiles through the spillway, free board under the roadway bridge, spillway discharge scheme, and risk for erosion in terms of water level and flow velocity downstream the dam at high river floods.

3.2 Spillway capacity

Structural studies showed that to modify only one opening instead of two or three was the most cost-effective way. Hydraulic model tests indicated that to lower the crest of the middle one would give relatively symmetrical flow patterns downstream.

Two major alternatives were tested in the model. One was to lower the sill of the middle spillway opening by about 4.0 m, the width remains the same (layout A). The other (layout B) was to lower the sill by 4.75 m and the width was reduced to 13.80 m, as the spillway pillar had to be reinforced to withstand the extra force from the segment gate, Figure 3.4 and 3.5. For either alternative, a new gate was needed.

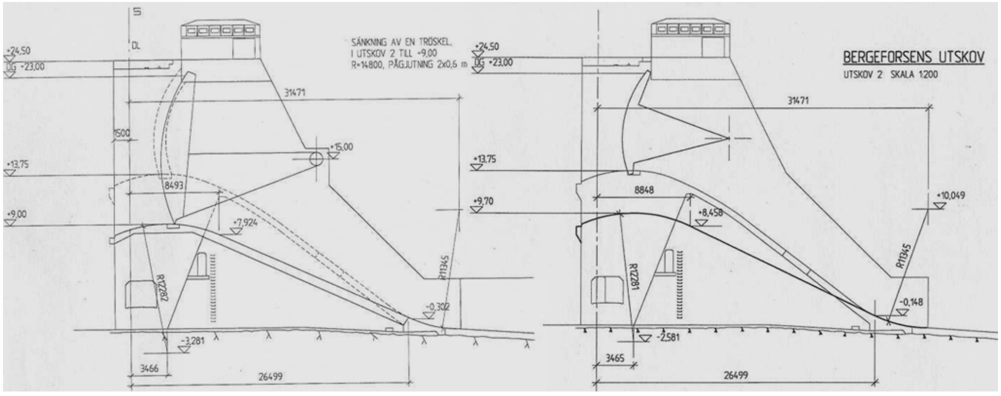


Figure 3.4 Layout B - sill lowering to +9.0 m, opening width reduced by 1.2 m (left); layout A - sill lowering to +9.75 m, existing width 15 m unchanged (right)

Figure 3.5 Sill lowering in the middle opening in model tests

The result for the middle spillway opening, before and after the sill lowering, is shown in Figure 3.6. The discharge coefficient C is defined by $Q = CBH^{1.5}$ where B = opening width and H = design head excluding the approaching velocity head.

At the FRRL, the existing middle opening released ~800 m^3/s. Layout A and B resulted in almost the same water level at a given spillway discharge; only a minor difference exists. The sill lowering increased the spillway capacity by ~600 m^3/s, giving a total flow of ~1400 m^3/s at the FRRL. The sill lowering of each layout, in combination with temporary storage of the flood water above FRRL, allowed one to discharge the new design flood ~3500 m^3/s at the reservoir level +24.0 m.

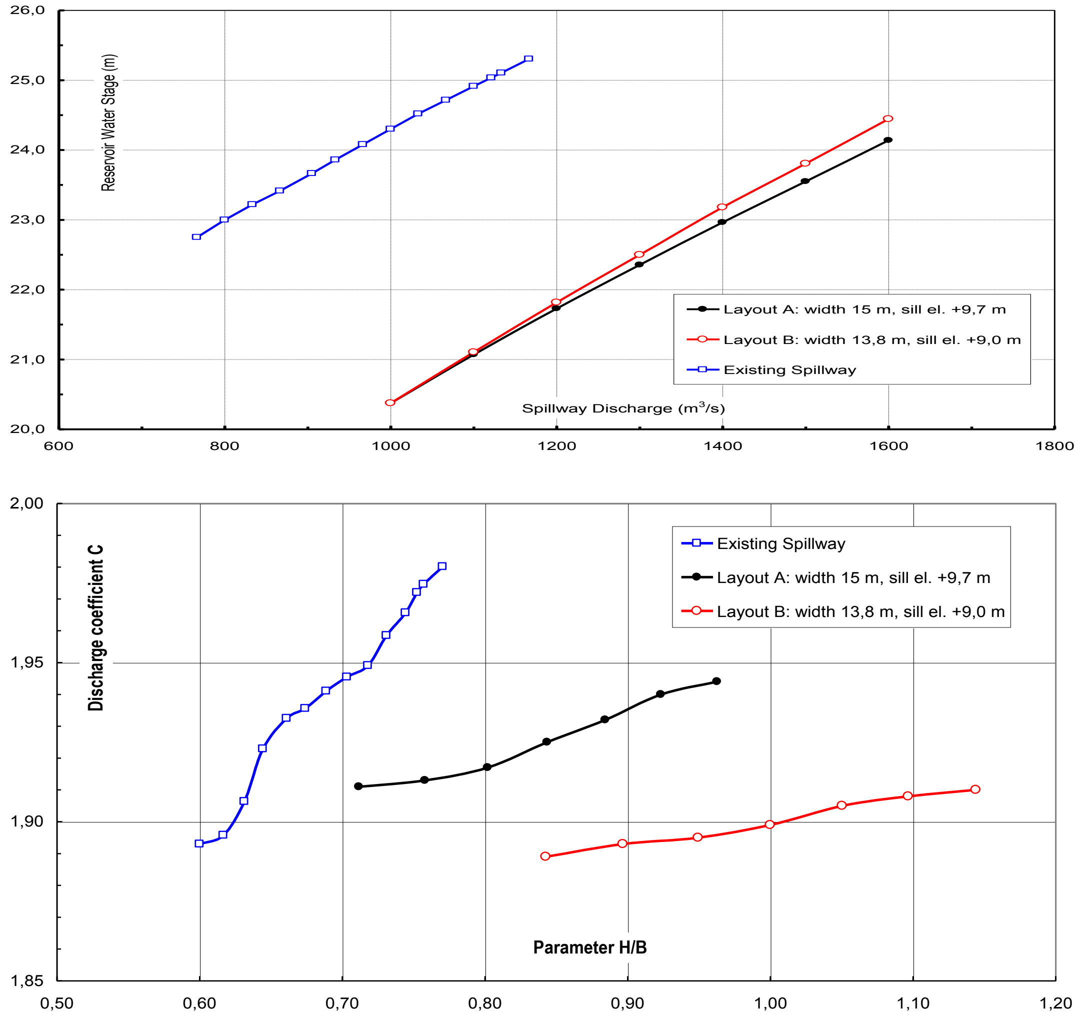


Figure 3.6 Comparison of Q-Z (above) and discharge coefficient (below) before and after modification of middle spillway opening

The middle opening has a standard Waterways Experiment Station (WES) crest shape, with a design head of 9.25 m and a discharge coefficient falling within 1.90–2.00 at water levels above the FRRL, which is typical. The difference in elevation between the sill and the river bottom is ~14 m. Theoretically, if the sill elevation of a spillway is lowered, its design head is increased and the downstream crest face should have a little bit flatter slope as specified by WES. However, this ideal situation is usually not satisfied. Limited by practical constraints and costs in an existing rebuilding project, a much flatter slope is often given as a compromise, thus leading to higher water pressure on the crest than in the existing design and accordingly lower C-value.

Another aspect is that any sill lowering reduces the distance from the spillway sill to the river bottom. This gives rise to less favorable approaching velocity profile than the existing upstream of the spillway and has a negative effect on the discharge.

In layouts A and B, the discharge coefficient falls within 1.90–1.95 at higher water levels than the FRRL. Both the flatter spillway crest and less favorable approaching velocity contribute to the decrease. In layout B, the opening width is less than that in layout A and the effect of the side contraction plays also a role.

3.3 Downstream impacts

One important aspect in the tests concerned the downstream area of the dam. The fish breeding station is located on the right riverbank. Immediately downstream the power plant, the left bank consists of relatively loose material. At high floods, the risk of flooding as well as erosion existed.

Figure 3.7 showed the velocity distribution at the discharge ~3350 m^3/s. Due to the oblique dam location in relation to the river valley, back flow was created and it took up roughly half width of the river. The back-flow velocity was as high as 2–3 m/s, depending on the discharge. Measures must be taken to protect the left bank immediately downstream the dam from erosion. It could be seen that the velocity along the riverbank is very high, up to 7- 8 m/s. Due to flow unsteadiness, irregular wave motions hit the bank and the breeding station. The spillway discharge scheme was therefore investigated to obtain the most favorable flow pattern at high floods, so that the risk for erosion was minimized.

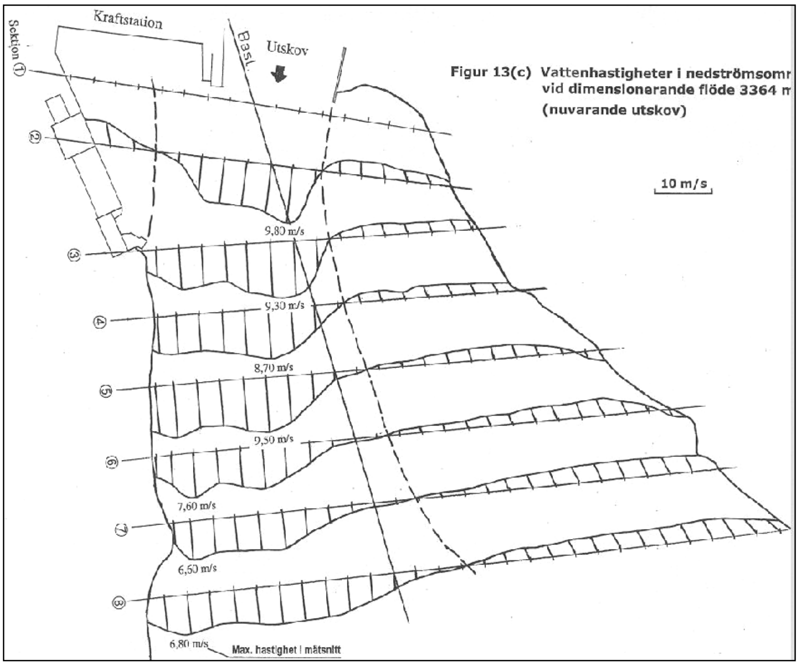


Figure 3.7 Flow velocity downstream at the discharge 3350 m^3/s

Another aspect studied in the model was the effect of dredging downstream the dam. Close to the right bank, there existed one area with much higher terrain, which contributed to higher water level at high floods and aggregated the situation. Even at normal turbine operation conditions, it caused higher downstream water level, which in turn led to reduced power production. There were plans to dredge the terrain so as to improve the flow pattern. How to dredge was tested in the model. Dredging should also take into consideration environmental impacts.

3.4 Comments

The basis for the conducted model tests was that a higher water stage than the FRRL was allowed in handling the extreme floods. Recent studies have shown that higher reservoir level might cause other problems for the dam. It is most likely that the dam has to be rebuild one more time, with an addition of a new spillway.

Documents related to Bergeforsen model investigations are given below.

Yang, J. (1998), Flow patterns downstream Bergeforsen – calculations with 2D model SMS (*Bergeforsen nedströmsvattenstånd – beräkning med 2-D modell SMS*), Report No. US 98:2.

Yang, J. (2000), Hydraulic investigations for Bergeforsen dam on Indalsälven (*Hydrauliska studier för Bergeforsen kraftstation, Indalsälven*), Report No. US 00:01.

Yang, J. (2000), Bergeforsen – spillway operation strategy and dredging in downstream area (*Bergeforsens kraftstation – tappningsplan för utskov och rensning i nedströmsområde*), Report No. US 00:34.

Billstein, M. & Yang, J. (2001), Bergeforsen – additional model tests with spillway capacity and floating debris (*Bergeforsen kraftstation – kompletterande modellförsök med avbördning och drivgods)*, Report No. U 01:82.

Yang, J, Gustafsson, A, Johansson, N, Johansson, G & Mikaelsson, J (2000), "Bergeforsen Dam – Refurbishment for Increased Design Flood", HydroVision 2000, Aug 2000, Charlotte, NC.

Yang, J, Cederström, M, Johansson, N & Hellstadius, K (2004), "Spillway Modification for Increased Discharge Capacity in Some Swedish Dams", HydroVision 2004, Aug 2004, Montréal, Québec.

4 Gallejaur

Gallejaur, completed in 1965, is located on Skellefteälven, about 100 km from the river mouth, Figure 4.1. The scheme consists of a power plant excavated in rock, a total of ten embankment dams of varying sizes and a spillway with a discharge canal. The crest length of the dams varies from 150 to 1600 m and their max. height from 1 to 50 m. The power plant with two units has an installed capacity of 220 MW, operating at a gross head of 80 m and a turbine flow of 300 m^3/s.

Gallejaur is a high-hazard dam (class I); its new design flood is about 850 m^3/s. Figure 4.2 shows the spillway dam. The spillway is composed of two 12-m openings with radial gates. The FRRL is at +310 m, corresponding to which the spillway discharge capacity is 720 m^3/s.

Figure 4.1 Gallejaur dam with gated spillway openings

Three ski-jump baffle blocks are adopted at differentiated levels in each opening, covering about 45% of the apron width. The baffles are also equipped with channels to provide aeration to the jets. The aeration guarantees atmospheric pressure beneath the nappes, thus allowing the jets to develop freely and obtaining effective energy dissipation.

The canal bed is located about 15 m below the FRRL. The canal, 45 – 55 m wide, is provided with slope protection of blasted rock riprap and stretches some 1000 m downstream. At a distance of 300 m from the dam, there exists an artificial threshold, Figure 4.2, the purpose of which is to maintain a reasonably high water level in the canal for efficient energy dissipation.

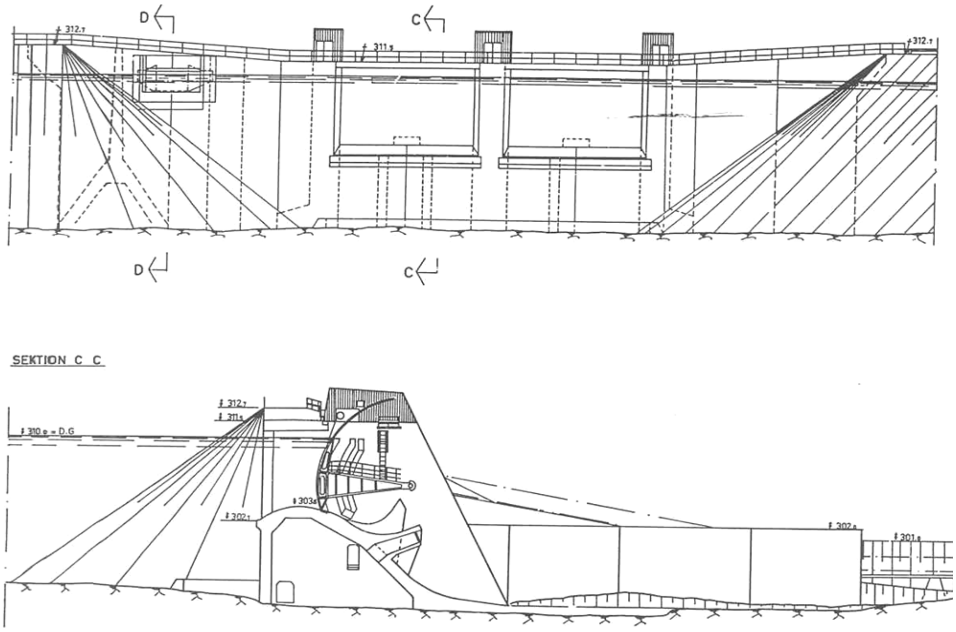


Figure 4.2 Layout of the Gallejaur spillway

4.1 Hydraulic concerns

The existing dam-safety guidelines stipulates that a class-I dam, as is Gallejaur, must be able to discharge the corresponding design flood without serious damages, either in the dam body or in surrounding structures, that can jeopardize its safety. The dam-safety issues of the dams are as follows.

Spillway discharge capacity

There was some uncertainty as regards the spillway capacity, which affected, in turn, the determination of reservoir storage height above the FRRL at the design flood. The model tests conducted during 1960-62 gave relatively low discharge coefficient, which was, also due to its poor documentation, questioned.

Ski-jump baffles and aeration

When the spillway was used in winter, water flowed via the aerating channels into below the spillway. To avoid this problem of water and ice, the channels were sealed with metal sheet, an issue that was handled without objective estimate. The channels were requisite to aerate the jets to avoid vacuum effects and jet oscillations. It was not clear that under which discharge the anticipated function was not affected.

Energy dissipation

The original idea of adopting the ski-jump baffle blocks was to replace a stilling basin. However, available records of the spillway operation during the past decades pointed to improperly designed energy dissipation. With the now higher design flood, there was a need to re-examine the energy dissipation.

Damages in erosion protection

Damages were often found in the rock riprap after spillway flood release. Temporary reparations were made at occasion. The damages might be due to under-dimensioned stone sizes in combination with ineffective energy dissipation. To prevent future damages and thereby reduce operation costs, one should investigate if there was a need to re-construct the erosion protection. Besides, the canal walls must be heightened to accommodate the higher design flood.

4.2 Model test results

The issues raised in the dam-safety evaluations were difficult to answer without further hydraulic investigations. In consideration of several factors, model studies were chosen to address the questions. A hydraulic model, in scale 1:40, was built, Figure 4.3. It included a 200 m river reach upstream, the spillway and a 350 m canal downstream. The model ended downstream of the concrete threshold serving as a critical section. The ski-jump baffles and aerating channels in the spillway were constructed with correct geometry.

Figure 4.3 Model of Gallejaur spillway and discharge canal

Besides conventional measurements, the under-pressure below the jets and wave motions in the spillway canal were also recorded. In certain cases, numerical simulations, if needed, were also carried out, so that the results can be compared with each other.

Existing Flow Conditions

As a basis for examining rebuilding measures, the existing flow conditions of the spillway and in the canal were first identified in the model. It aimed also at finding explanations for the damages that occurred in the erosion protection in the spillway canal.

Spillway discharge capacity

The test findings of the spillway capacity obtained coincide with the tests made 1960-62, only minor differences exist. The discharge coefficient C, defined in $Q = CbH^{1,5}$, falls within the range 1.75–1.85. The spillway does have somewhat lower C-value than the normal. The reason is mainly due to the effect of the middle ski-jump baffle. This upper baffle is placed very close to the sill and affects the free flow on the spillway crest at higher discharges. Tests also showed that, whether the aerating channels are sealed or not, the spillway capacity is not affected.

Aeration of flow - Figure 4.4 shows the under-pressure measured under the jets over the middle (upper) baffle and the side (lower) baffles. Neglecting the model scale effect, the flow should be obviously aerated at discharges higher than about 50 m^3/s for the middle baffle and 100 m^3/s for the side ones. To seal the aerating channels is not advisable above the limits.

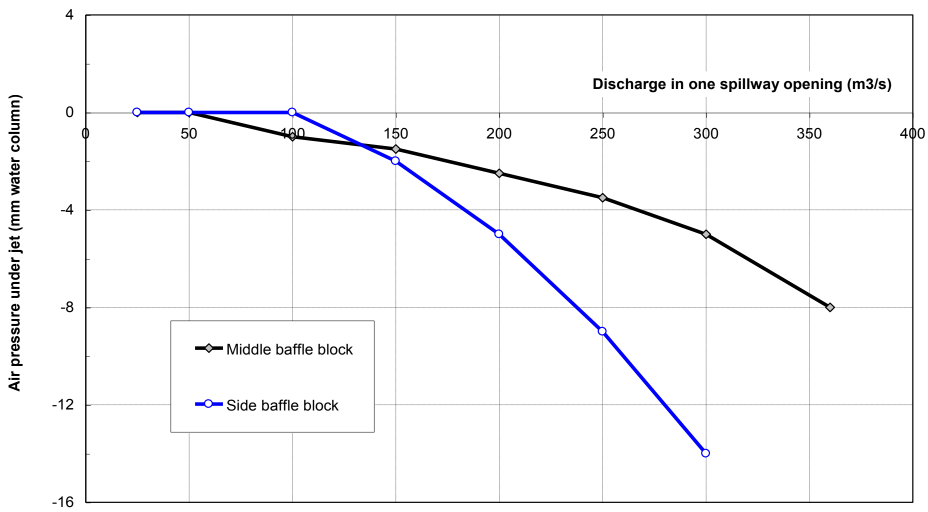


Figure 4.4 Measured under-pressure beneath nappe in the model

For comparison of the jet behaviour, all the three aerating channels in the left spillway opening were sealed while the channels in the right one were kept open for aeration. It was observed that the aerated channels spread out the flow all over the opening width. The nappe developed freely. As a result, a stable, thin water curtain was formed. For the sealed opening, the pressure difference across the nappe at higher discharges resulted in a smaller curtain and the nappe reaches in the flow direction not as farther as in the aerated opening. Besides, the jet was not stable – it oscillated, though with low frequency, in the flow direction. This instability gave rise to less effective energy dissipation and stronger wave motions in the canal.

Flow conditions downstream

The document from the 1960-62 model tests did not provide any information as regards which discharge the energy dissipation was tested and designed for. It seemed that the use of the ski-jump baffles instead of an ordinary stilling basin did not achieve the desired result at higher floods. It was indicated in the tests that the energy dissipation was insufficient and the water current was accompanied by strong waves in the whole 300 m canal.

Available records showed that the spillway canal was only designed for a flow discharge of approximately 500 m^3/s – the new design flood was 850 m^3/s. Previous flood releases in the prototype showed that the canal was overtopped at several location around 500 m^3/s. Obviously, the determination of the sidewall crest elevation took into account only the water level, not the wave motions (run-up) that usually required separate free board.

Figure 4.5 illustrates the velocity distribution in the canal at a location 80 m from the dam. The resulting velocity with only one spillway gate in operation, irrespective of left or right, was much higher than that with both gates opened at the same time. Besides, due to the fact that a lower discharge corresponds to a lower water level in the canal, the waves acted on lower position of the erosion protection when only one spillway was open.

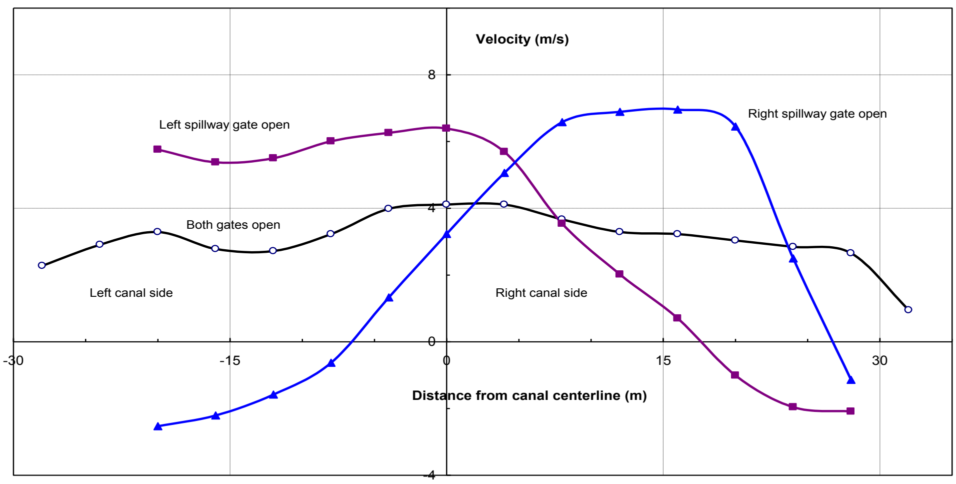


Figure 4.5 Flow velocity distribution in the canal, only one gate open 360 m^3/s and both gates open 840 m^3/s

At many occasions, damages in the erosion protection were found after spillway operation. With the help of commonly used empirical formulas, the required stone size (D50) was calculated. It appeared that the design of the rock riprap in Gallejaur dam was only based on the flow velocity.

Different rebuilding possibilities were examined in the model. Not all the alternatives were adopted in the dam.

Use of nappe splitter

The aerating channels that were sealed prevented the energy dissipation from functioning properly. One idea was to use a so-called nappe splitter or divider to open the nappe, so that it was aerated through the opening created, as is seen in Figure 4.6. The splitter was vertically mounted on the top of the ski-jump baffle. It was given a semi-elliptical shape in plane, defined by

$$\left(\frac{x}{A}\right)^{\hat{}} + \left(\frac{y}{B}\right)^{\hat{}} =$$

where A and B are major and minor axes of the ellipse. The splitter height should be larger than the water depth at the end of the baffle block.

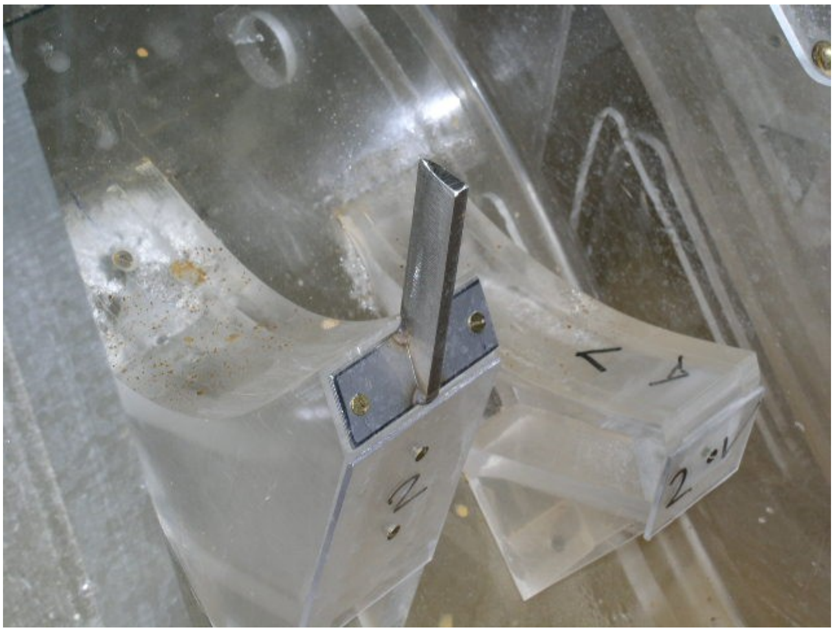


Figure 4.6 Use of a splitter on the baffle to aerate the nappe

Through tests, it was shown that the splitter was an effective to avoid under-pressure beneath the nappe. Its design should be, however, optimised in relation to the flow in question. One tested shape corresponded to a major axis of A = 60 cm in the flow direction and a minor axis of B = 10 cm (prototype size). The splitter thickness was thus 2B = 20 cm. With this shape, the already divided nappes could meet and re-open at intervals, which meant that the splitter thickness was not large enough and should be increased.

One disadvantage of using the nappe splitter was that it might clog floating debris usually released through the spillway during high floods. Structurally, it should be constructed in such a way that it withstood the impact of flow, ice and floating debris.

Flow behaviour in new stilling basin

To improve the flow conditions in the canal, excavation of a stilling basin was proposed. The basin tested in the model was given different layouts, corresponding to a length of 25 – 80 m and a depth of 6.0 – 8.0 m below the exiting canal bottom. Depending on the canal width, the basin width varied between 25 - 40 m, Figure 4.7.

Figure 4.7 Use of stilling basin to improve energy dissipation

Downstream the sloping spillway chute, the desired basin bottom was a curved profile to be excavated below the existing concrete surface, illustrated by the dotted line in Figure 4.8.

However, structural investigations showed that, due to the requirements of spillway stability and other restrictions imposed on construction, to excavate to that profile was not permitted. As a compromise, the existing horizontal chute was maintained and the basin was instead given a stepped profile. Due to this, the upstream end of the stilling basin was located somewhat downstream, at a distance of 40 m from the dam axis.

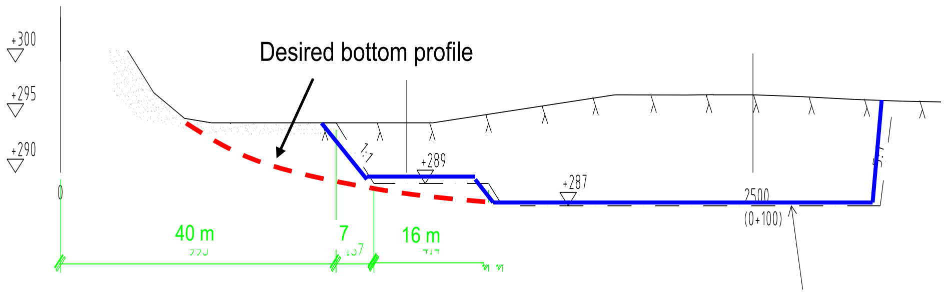


Figure 4.8 Excavation of stilling basin – longitudinal profile

Tests indicated that the jets over the three baffle blocks plunged unexpectedly on the horizontal chute, i.e. outside the basin, where the water depth was very limited. Besides, conditioned by the horizontal chute, very strong surface currents prevailed in the basin area. As a result, strong waves were generated, propagating further downstream in the canal. The flow velocity along the basin bottom was very low, implying that the basin depth did not play much role in the energy dissipation. Despite a larger basin length, the flow pattern was not affected.

Based on the general flow pattern and measurements of velocity and waves, it was difficult to state that the stilling basin improved the energy dissipation and flow conditions in the canal. The reason was twofold. The flow was directed towards the surface water, which did not follow the design principle of a typical stilling basin where the water should be directed towards the bottom of the basin. That the flow from the three baffles plunged outside the basin accounted also for the unsatisfactory flow condition.

Raising canal sidewalls

To obtain reliable results, the canal flow was compared with 1D numerical simulations. As far as the mean water level was concerned, the two approaches provided very similar results, with a discrepancy of about ±15 cm. The sidewalls had to be raised to accommodate the new design flood 850 m^3/s. Due to the strong wave motions, the increase in the heights must include the effect of the wave run-up.

Figure 4.9 shows, in relation to the mean water level, the result of wave run-up on the canal slopes at the discharge 850 m^3/s. Larger run-ups dominated farther upstream, with a maximum amounting to ~1.5 m. The test results provided a basis for determination of the final sidewall crest elevation and some safety margins should also be added. Pre-fabricated parapet walls can be used on the top to reduce the requisite wall height and their purpose was mainly to prevent overtopping of waves.

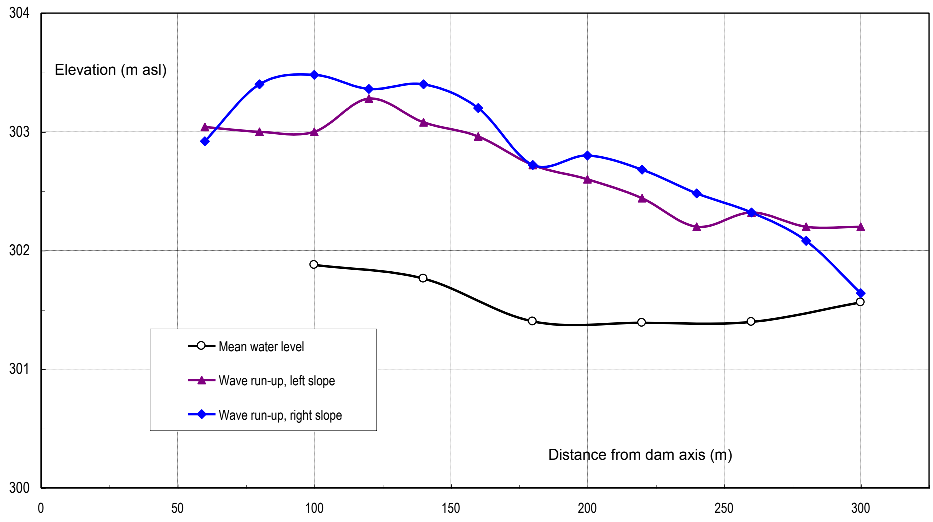


Figure 4.9 Impact of wave motions on canal sidewall height

Removal of ski-jump baffles

One way to tackle the problem with the aerating channels was to remove the ski-jump baffles. The practice was probably difficult to carry through and also expensive. However, as a potential option, it was examined in the model from the hydraulic point of view. With the baffles demolished, a stilling basin must be built to dissipate the energy.

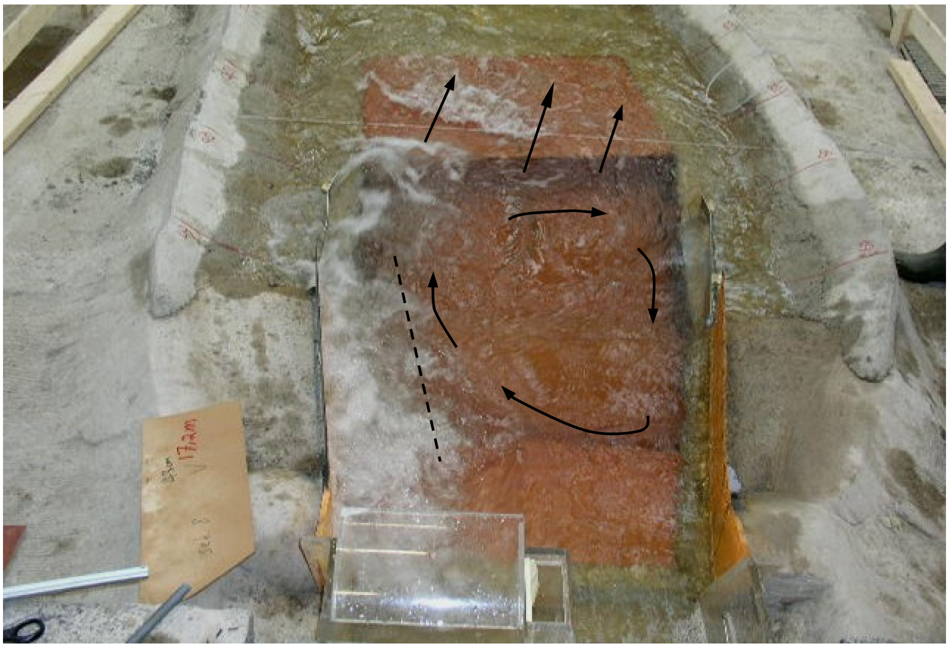


Figure 4.10 Removal of ski-jump baffles - highly concentrated flow with large circulating zone generated in canal

Conditioned by the geometry of the canal and the stilling basin, a highly concentrated flow, accounting for only 20–25% of the canal width, built up downstream the spillway if only one gate, either left or right, was in use, Figure 4.10. The flow was characterised by a large back-flow area and considerable water-level difference. Tests demonstrated that, if the middle spillway pier was extended downstream in form of a partition wall, the flow pattern could be avoided and relatively even flow was achieved in the canal.

Despite the extra volume added to the stilling basin, the flow velocity was still about 1 m/s higher than in the existing layout with the baffles. With the help of wave measurements, the wave motions were found to be stronger and the wave run-ups higher. To construct a larger stilling basin was therefore not an effective way to dissipate the energy, at least not in this case.

Modification of erosion protection

In the model, a section of the canal sidewall immediately downstream the spillway chute was removed. It was replaced by blasted stones whose size D_{50} was based upon those often used formulas at a given canal discharge. The stone slope was built with the same angle as in the prototype. Repeated tests showed that the slope collapsed easily, Figure 4.11. To guarantee its stability, a large safety factor was a must. To use rock riprap as erosion protection in close vicinity of a stilling basin was probably not a good engineering practice.

Figure 4.11 Spillway canal subjected to with high velocity and strong waves - collapse of riprap-slope protection whose size was only based upon flow velocity

Most empirical formulae for determination of stone sizes in an erosion protection in a canal take only into consideration the velocity effect, as is the case in Gallejaur. Very limited information is found as to how the contribution of the wave motions can be considered. One can mention that, for the upstream slope protection of an embankment dam, rock of sufficient size is required to resist wind-generated waves in the reservoir. It is unclear if these formulas of stone size determination developed for reservoir waves can be borrowed to assisting in estimation of the effect of canal waves caused by spillway release. Neither is it clear how the effects of the flow velocity and wave actions on stone size can be combined.

Even if one could estimate the requisite stone size, it wouldn't be realistic to replace the riprap in an existing canal because large engineering costs were usually involved. The rebuilding measure suggested for Gallejaur was to have the voids filled with concrete grout, so that the stones were joined together to form a layer of rigid, monolithic armour, Figure 4.12. Grouted rock can resist larger hydraulic forces than the riprap itself. Proper drainage should be arranged to avoid the uplift pressure behind it when the water level in the canal drops. The loss of the supporting bank material should be also avoided, as the grouted rock was particularly susceptible to failure from undermining.

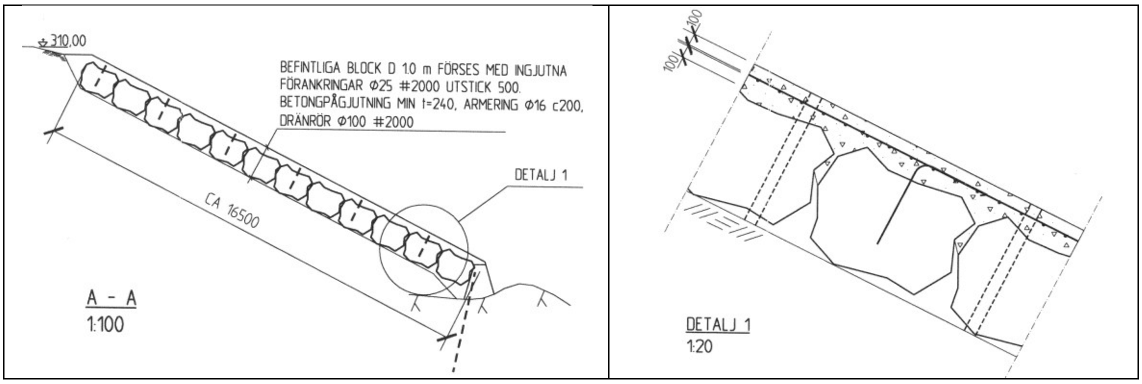


Figure 4.12 Illustration of concrete grouting of stones in exiting erosion protection (stone size 1.0 m, min. concrete thickness above stones t = 240 mm; reinforcement bar ϕ25, #2000, L=1500; drainage hole ϕ100, #2000)

Gallejaur model studies are found in the following documents.

Yang, J & Amnell, G. (2004), Dam Safety at Gallejaur dam – hydraulic model investigations (*Gallejaur dammsäkerhet – hydrauliska utredningar*), Report No. U 04:04.

Amnell, G & Yang, J (2004), Spillway at Gallejaur dam – discharge capacity at different gate openings *(Utskov i Gallejaur – avbördningsuppgifter vid olika lucköppningar)*, Report No. U 04:102.

Yang, J & Cederström, M (2006), "Gallejaur dam safety – hydraulic concerns related to spillway flood discharge", HydroVision 2006, July/Aug 2006, Portland, OR.

5 Halvfari

Halvfari was commissioned in 1978. It is composed of an embankment dam, a power station and a spillway structure, Figure 5.1. The max. dam height is 43 m and the crest length is ~1000 m. The active reservoir storage corresponds to 25 Mm^3 at the FRRL, +430.0 m. The power station is equipped with a 24-MW generating unit, operating at a gross head of 25 m and a turbine flow of 120 m^3/s.

The spillway structure is located to the right of the power station. The spillway is composed of a siphon spillway and two overflow openings equipped with upward radial gates. The two openings have the same sill elevation, +420.0 m, and the same width, 6 m. The left opening has a standard Ogee crest, while the right one is of a free-drop type. Excluding the siphon capacity, the flood discharge capacity of the spillway is ~650 m^3/s at the FRRL.

The spillway water is discharged into a plunge pool excavated in rock. Sidewalls exist on both sides to prevent overtopping at high spillway discharges. Downstream of the plunge pool, the right riverbank consists of easily erodable material.

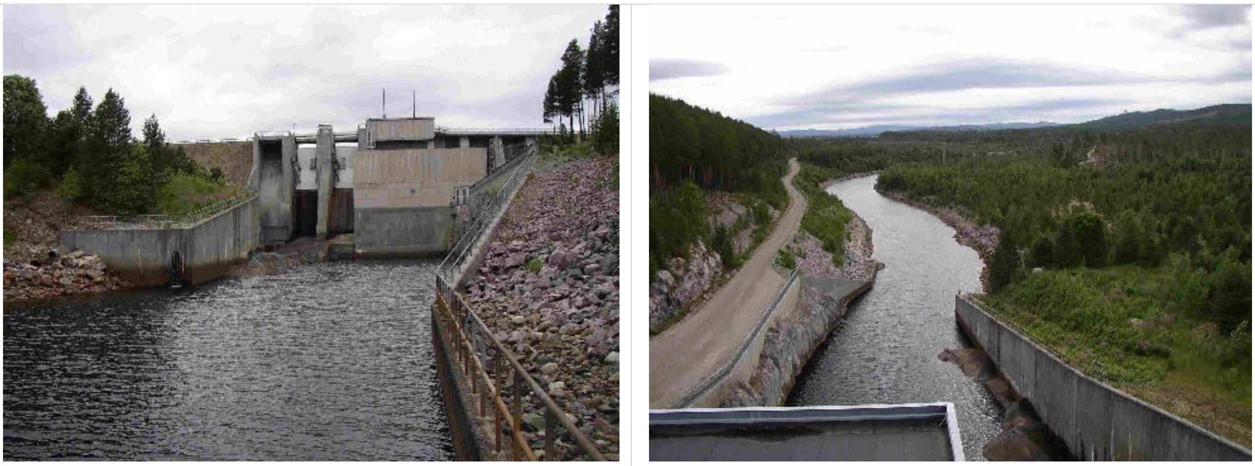


Figure 5.1 Halvfari dam - spillway section and channel downstream

Halvfari is a high-hazard dam (class I). Its updated design flood is as high as 1200-1300 m^3/s, much higher than the existing discharge capacity about 650 m^3/s. The main hydraulic concerns of the dam and safety issues are closely related to this large increase in the design-flood magnitude.

The siphon spillway, located to the right of the overflow spillway, can discharge around 120 m^3/s. However, due to the safety concern arising from vibration and cavitation, its discharge capacity is not included in the overall capacity of the dam in high flood events.

5.1 Hydraulic concerns

The fact that the spillway has insufficient capacity leads to several rebuilding proposals. One way to increase the discharge capacity is to construct a new spillway in the right embankment dam. To add a separate spillway unburdens the exiting energy dissipation. However, analysis shows that it is not a cost-effective solution.

The spillway between the Ogee and siphon spillway is a free-drop type of opening. The passage below it is at present sealed with a vertical concrete plug. Another rebuilding proposal is to remove this plug and modify the passage into a bottom outlet, Figure 5.2. This means that the plug is dismantled and the horizontal crest of the overflow opening is extended downstream and bent somewhat downwards. At the end of the extension, a bulkhead gate is installed for closure of the outlet. The outlet has the same width as the free-drop spillway at its upper part, 5.0 m, and reduces to 4.5 m towards the bulkhead gate. The outlet height is 8.5 m and reduced to 7.0 m at the bulkhead gate. As a technically feasible solution, this proposal is chosen for further investigation.

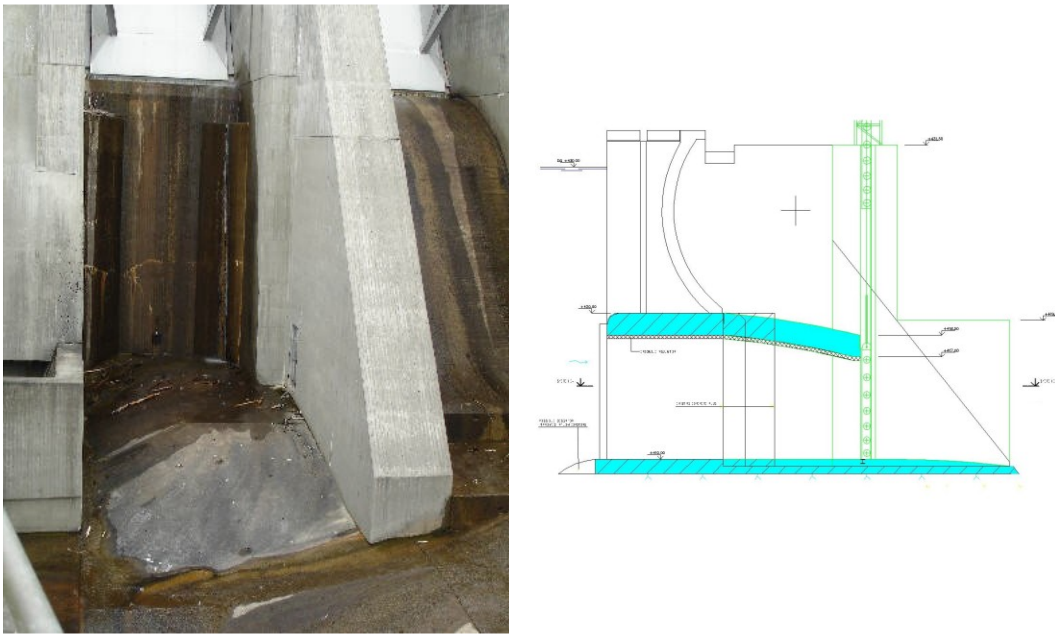


Figure 5.2 Rebuilding of sealed passage to bottom outlet

The plunge pool is located immediately downstream of the powerhouse. This means that the spillway water plunges obliquely from the right side into the plunge pool of limited volume. The proposed bottom outlet aggravates the energy dissipation in such a way that the plunging jet becomes more powerful if all the spillways are open for flood release. The river course from the powerhouse and some distance downstream of the pool is bounded by bedrock on the left. However, the right riverbank consists of loose material sensitive to erosion. With the large increase in the design flood, concerns arise of the risk of erosion in the right bank due to high flow velocity and strong wave motions.

The questions of the bottom outlet, energy dissipation, erosion potential downstream and the need of engineering measures and structural modifications would be difficult to answer without hydraulic model tests. CFD is not a suitable tool at the moment. Notwithstanding the rapid developments of computer modeling, the effective solution of this type of hydraulic problem involving heuristic engineering reasoning in the face of constraints, model testing and actual prototype implementation, is perhaps best handled by intelligent use of physical models.

5.2 Model tests

A hydraulic model of the dam was constructed, Figure 5.3 and 5.4. A reach of some 400 m upstream and 500 m downstream of the dam is included in the model. The chosen scale is 1:40, resulting in a model length of ~20 m and a maximum width of ~10 m.

The issues examined in the model comprises flow pattern in the reservoir, spillway discharge capacity, structural improvements for more effective energy dissipation and erosion development in the right riverbank during high floods.

Figure 5.3 Fixed-bed model for Halvfari dam

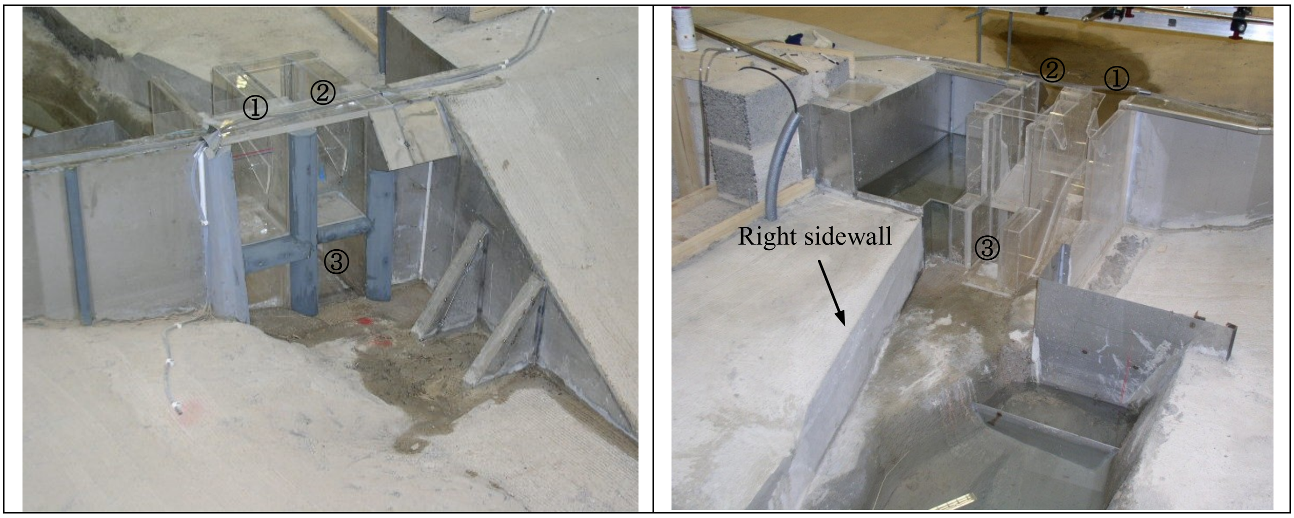


Figure 5.4 Spillway structure in the model, seen from up- and downstream

5.2.1 Discharge capacity

Before and after the addition of the bottom outlet, the spillway discharge capacity of the dam when all the gates are in fully open position is illustrated in Figure 5.5. With the outlet, the total discharge increases to ~1100 m^3/s at the FRRL, as compared to the exiting ~650 m^3/s. The design flood ~1300 m^3/s can be discharged at a ~1.5 m higher reservoir level than the FRRL.

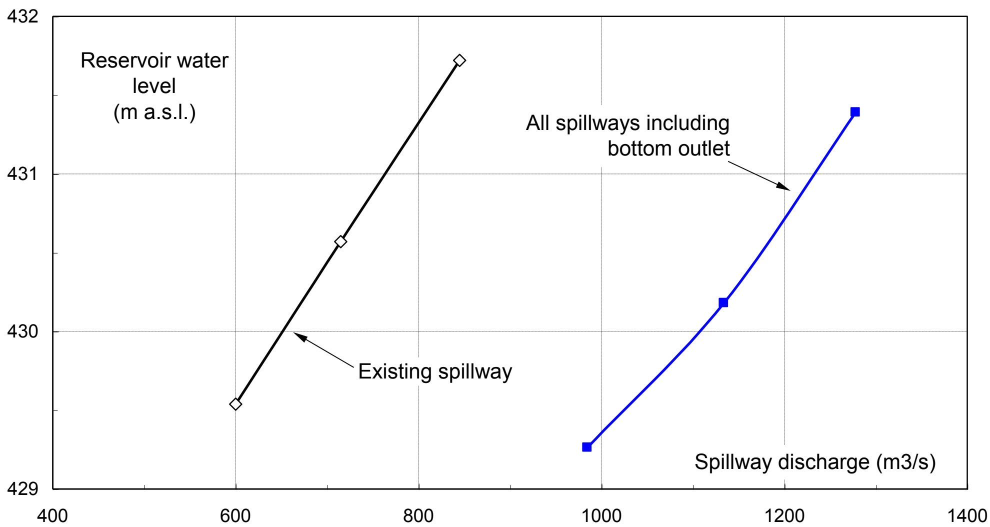


Figure 5.5 Discharge capacity before and after addition of the bottom outlet

It is interesting to observe that the outflows from the right spillway and the bottom outlet affect each other. Figure 5.6 shows their *separate* and *simultaneous* discharge capacity when the left spillway is closed. If they are opened separately (one at a time), we denote the discharge from the bottom outlet and the right spillway as Q_B and Q_R, respectively, their sum being $Q_B + Q_R$. If they are opened simultaneously, we denote the discharge capacity as Q_{BR}. Tests have shown that Q_{BR} is 80 – 100 m^3/s lower than $Q_B + Q_R$ within the reservoir water level interval +427 – 431 m. The reason for the lower simultaneous capacity is mainly due to the reduction in the outlet outflow. The flow from the right spillway affects unfavorably the water pressure acting on the outlet flow downstream of the bulkhead gate, which retards, to some extent, the flow from the outlet and decreases its capacity.

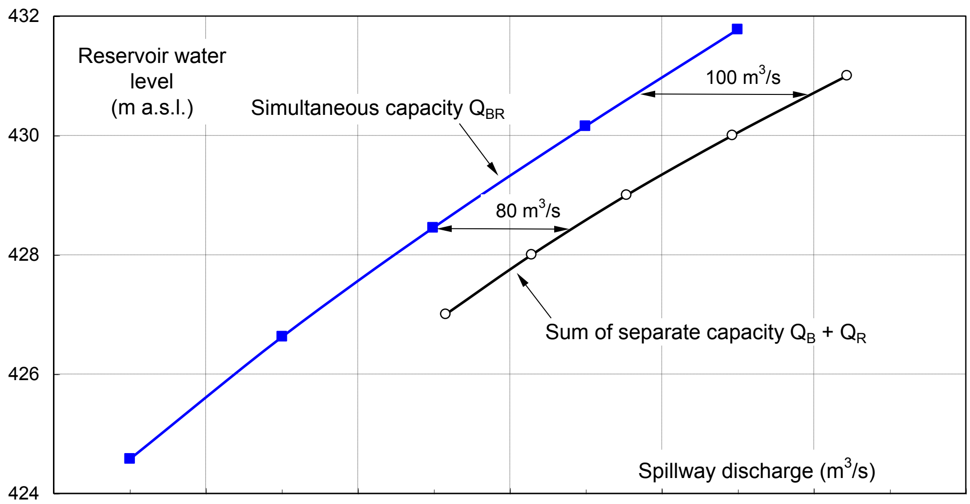


Figure 5.6 Separate and simultaneous capacity of right free-drop spillway & bottom outlet

When discharging simultaneously at the FRRL, the bottom outlet and right spillway discharge some 500 m^3/s more than the left spillway, a great difference that gives rise to problem for both the right sidewall and plunge pool. Figure 5.7 shows the flow situation at ~500 m^3/s from the bottom outlet and ~1300 m^3/s when all the spillways are open.

5.2.2 Existing energy dissipation

The right sidewall, running at an angle with the spillway piers, is overtopped if the bottom outlet is fully open. Any flow contribution from the right overflow spillway worsens the situation. The flow from the left spillway meets the flow from the left and bottom spillway, pressing against the right sidewall, before being reflected askew into the plunge pool.

The jet plunges into the downstream part of the pool and the pool water volume is not effectively used. This causes plane-circulating flow pattern to build up in the pool, with considerable water level difference between its up- and downstream parts. The flow with strong waves constantly washes up on the left riverbank at the end of the plunge pool, before being redirected back to the river. Due the insufficient energy dissipation, high flow velocity and strong wave motions accompany the flow in the watercourse downstream.

Structural modifications are an absolute necessity to improve the energy dissipation and mitigate the turbulent flow situation in the pool.

Figure 5.7 Existing energy dissipation, ~500 m^3/s only from bottom outlet (left) and ~1300 m^3/s from all three openings (right)

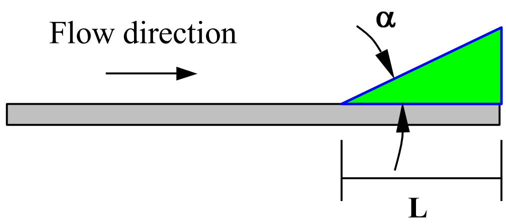
5.2.3 Improving energy dissipation

The existing spillway piers end too early upstream, the water from both sides interrupts each other. A large portion of the spillway water runs directly onto the right sidewall. By trial and error, it has been found that if the middle and right spillway piers are extended, the flow situations immediately downstream of the spillway gates are improved, which provides prerequisite of further improvements (Figure 5.8).

Figure 5.8 Improvement through extension of spillway piers, at the flow discharge ~1300 m^3/s

It is suitable is to extend the piers to the front of the powerhouse. The extension measures ~10 m for the middle pier and ~13 m for the right one. By doing so, the water is directed further downstream, before entering into the pool. The current from the bottom spillway is no longer affected by the flow from the left spillway and flows straight ahead. The extensions cause the water level in the pool to rise about 2.0 – 2.2 m immediately downstream of the powerhouse. This means that more water is retained in the pool. The right side of the middle extension should also be streamlined to eliminate abrupt changes of water surface in the flow direction.

From the energy dissipating point of view, the plunge pool has a very limited volume and water-surface area. To achieve effective energy dissipation, a concentrated flow jet into it should be avoided if possible. With the extended spillway piers, the flow conditions and energy dissipation in the plunge pool can be further improved by adding vertical deflectors at the end of the piers, Figure 5.9. Tests show that deflectors are needed in both spillway openings. The reasonable angle, measured in the flow direction, is $\alpha_L = 30°$ for the left deflector and $\alpha_R = 25°$ for the right one. The suitable length of both deflectors is $L_L = L_R = 2$ m.



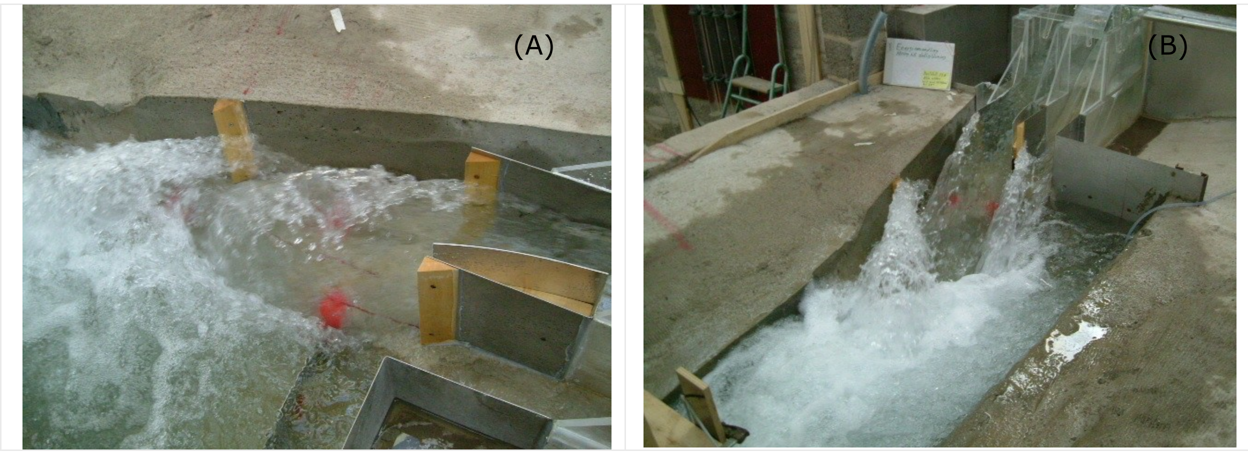


Figure 5.9 Finalized modification by installation of vertical deflectors at the end of extended piers and on the right sidewall, (A) discharge in bottom outlet and (B) discharge in all openings

The right deflector directs effectively part of the concentrated outflow from the bottom and right spillway into the pool. To deflect the remaining part of the water to form a plunging jet, it is found necessary to add a deflector on the right sidewall. Its shaping is dependent on the angle of the right deflector. With $\alpha_R = 25°$, a suitable angle for this deflector is $\alpha_w = 30°$. The deflector is placed vertical against the sidewall, at a perpendicular distance of L_w = ~57 m to the dam axis.

With those deflectors, the concentrated jet is avoided; the spillway flow is redirected in three separate plunging jets, reasonably distributed over the pool area. The pool water level becomes also somewhat higher. The jets plunge roughly in the middle of the pool, consequently resulting in less impact for the river course immediately downstream. The proposed structural modification functions well for spillway discharges up to the design flood.

5.2.4 Erosion potential downstream

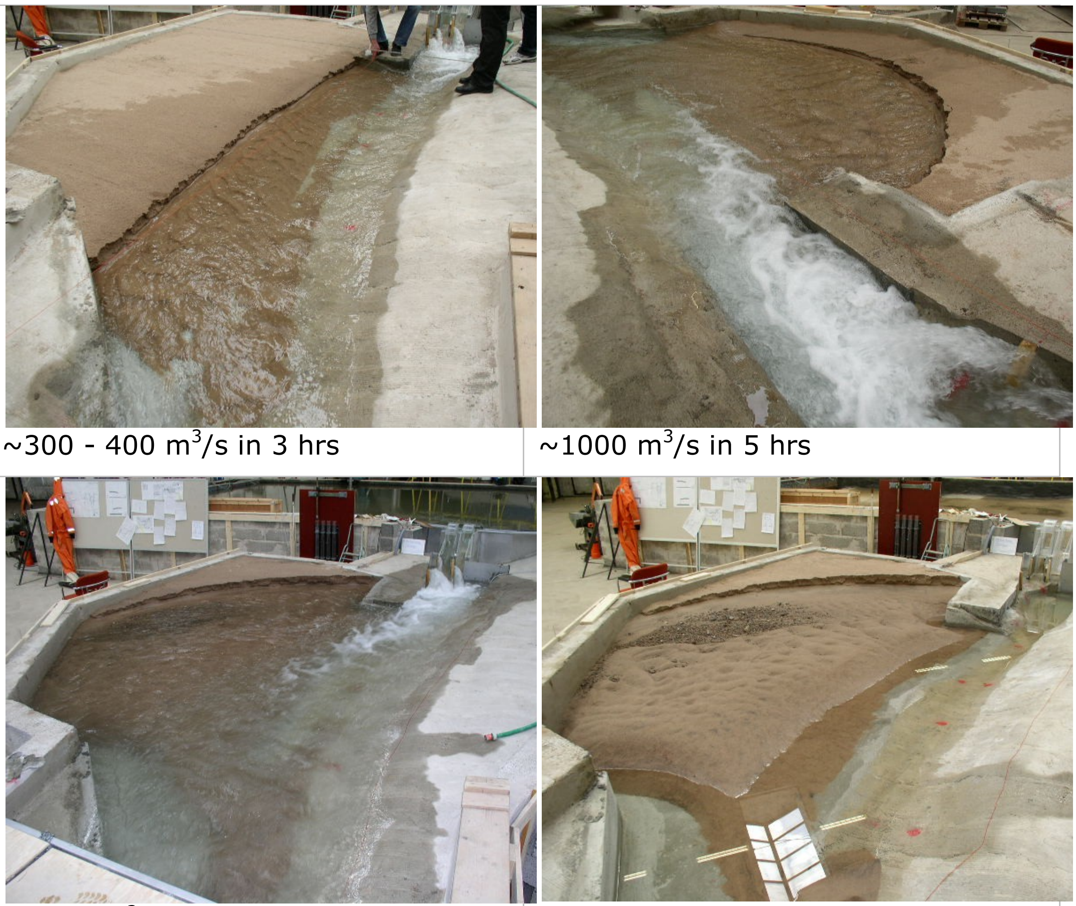
In spite of the structural modifications and improved energy dissipating function, the risk for erosion in the right riverbank consisting of loose material remains a major safety concern of the dam. It is estimated that the riverbank can withstand a flow velocity below 3.0 m/s without initiation of erosion. Tests of erosion are made in the model, the purpose of which is to identify the erosion potential and justify the need for engineering counter-measures.

The fixed-bed model downstream is rebuilt and the right riverbank is covered with a natural sand to the solid rock elevation, representing a width of ~100 m and a length of ~200 m in the flow direction downstream of the plunge pool. The sand is non-cohesive and has a density of ρ = 2250 kg/m^3 and a median diameter d_{50} = 0.55 mm.

Tests are made, starting with a low spillway discharge of ~300 - 400 m^3/s, successively increased to 1000 m^3/s and finally to 1300 m^3/s, Figure 5.10. The erosion tests last roughly 12 hrs (net time). At the low discharges, erosion occurred at a lower elevation in the right bank and developed somewhat uniformly alongside the river. With an increasing flow discharge, wave motions become probably dominant and erosion develops much quicker as expected. Accelerated erosion is visible immediately downstream of the cross wall. The erosion is obviously due to the combined effect of high flow velocity and strong wave motions.

About 10 min. after the spillway discharge 1000 m^3/s is released, a large circulation zone, driven by the main stream, is formed along the first 50 – 60 m downstream of the plunge pool. The circulation zone extends finally to the whole model length in the downstream area. With the widened flow passage, the main stream changes its direction and points diagonally to the right bank. The mean flow velocity in the river course measures 5.5 – 7.0 m/s. The mean velocity in the circulation zone amounts to 2.0 – 2.5 m/s, with a maximum varying between 3.0 – 3.5 m/s.

The erosion tests could be run somewhat longer. However, one can see that a large circulation zone, characterized by higher flow velocity, is formed in the right riverbank due to high spillway discharges. The erosion could develop further in the riverbank and towards the dam toe, which obviously necessitates engineering measures to prevent this. The performed tests have illustrated the need for counter-measures to be taken, so that the safety of the dam is not jeopardized at high spillway discharges.



~300 - 400 m^3/s in 3 hrs

~1000 m^3/s in 5 hrs

~1300 m^3/s in 4 hrs

Eroded right riverbank

Figure 5.10 Tests to examine the risk of erosion in the right riverbank due to spillway discharge

Model tests made for the dam are documented in the following publications.

Amnell, G, Yang, J, Larsson, P & McGowan, D (2007), Halvfari Dam, Ljusnan - Hydraulic Model studies for increased Spillway Capacity, Report No. U 07:21.

Yang, J, Larsson, P, Dath, J & Löwén, K-L (2007), "Halvfari Dam – hydraulic concerns and rebuilding for higher safety standard", Intl. Symposium on Modern Technology of Dams, Oct. 2007, Chengdu.

6 Harsprånget

The dam at Harsprånget was completed in 1952. Three units of 110 MW each were taken into operation in 1951-52. The power plant was extended during 1974–83, with two new units of 165 and 450 MW. The station, with an installed capacity of 945 MW, operates at a head of 107 m and a total turbine flow of 1040 m^3/s, Figure 6.1.

The dam is of the rock-fill type with an impervious core of concrete. The length of the crest is 1320 m, the width of the crest is 4 - 20 m, and the maximum height above bedrock is 50 m. The dam has three overflow openings, regulated by tainter gates. The opening width is 20 m each and the water depth from the spillway crest to the FRRL (+312.7 m) is 7.6 m. According to the laboratory test results from 1980, the max. discharge capacity at the FRRL corresponds to 2300 m^3/s, Figure 6.2.

Figure 6.1 Harsprånget dam

Figure 6.2a Spillway channel, seen from downstream (photo: Leif Kuhlin)

Figure 6.2b Spillway channel, seen from upstream

The model for Harsprånget was built in a scale of 1:60, covering the river 350 m upstream and 850 m downstream of the dam, Figure 6.3a. The spillway section, half of the main dam from the spillway and the whole river valley downstream were included in the model.

In order to in an approximate way reproduce the rock-surface condition, the model was in its downstream area provided with macadam, 40–60 mm, and some gravel, 70-120 mm, Figure 6.3b.

Figure 6.3a Model of Harsprånget, 1:60, seen from downstream

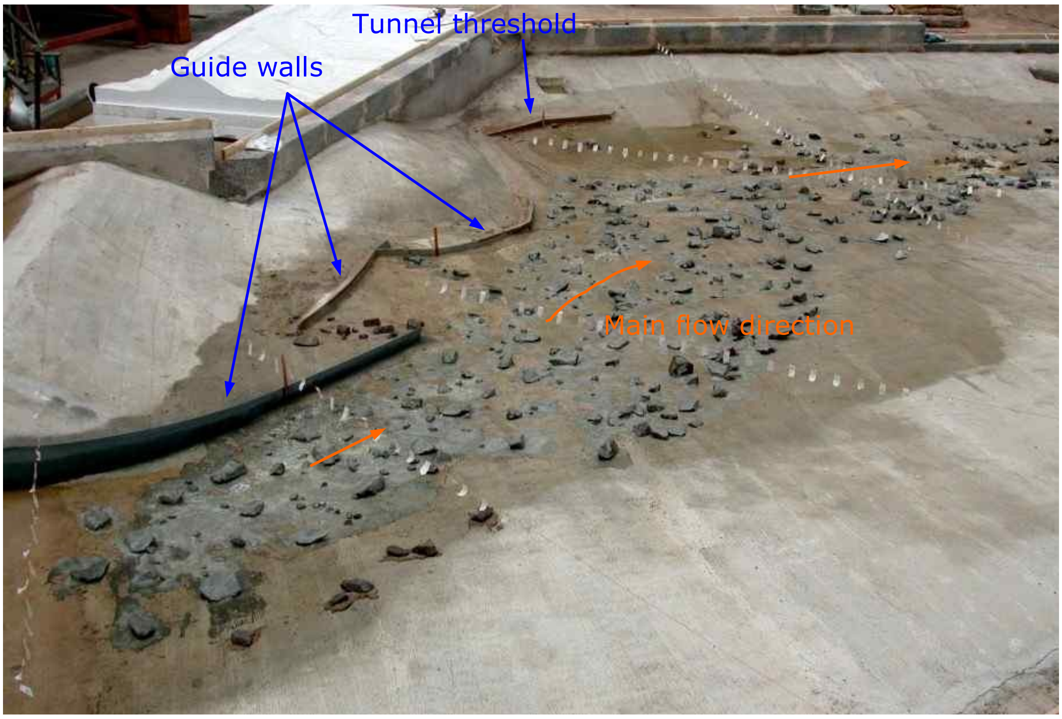


Figure 6.3b The model surface was roughened downstream to imitate reality

The following aspects are issues of concerns at the dam.

- flow situation in the existing design
- spillway discharge capacity
- water levels at the dam toe
- water velocity and flow directions downstream
- wave amplitudes at the dam toe
- re-shaping spillway channel and increasing the dam safety

The design flood of the dam is roughly 2000 m^3/s, which means that the existing spillway capacity is enough to discharge the flood. However, the weakness of the existing layout was obvious with respect to overtopping of guide walls, extensive erosion of the dam toe and high water level in the downstream area. The wave amplitudes were almost 3 m high and were considered to cause erosion downstream. The overall dam safety was not fulfilled for the dam. Consequently, an upgrading to meet the discharge criteria was necessary.

The natural rock surface of the spillway channel had a slope falling to the left, which forced the water into a narrow discharge channel and towards the guide walls. This was the major reason for the above-mentioned problems. Thus, a solution ought be a re-shaping of the spillway channel distributing the water over a larger cross-sectional area and diverting it away from the guide walls.

The spillway channel, following basically the exiting curvature, was re-shaped by excavation; several layouts were tested. On a trial-and-error basis, a total of 80 000 m^3 rock was removed over a length of 200 m from the spillways and downstream. The water was thus diverted away from the guide walls and the thresholds to the tunnels. The channel surface was made as smooth as possible; the water was thus distributed over a wider area, which also reduced the loads acting on the rock. The excavated rock mass would be used for improvement of the erosion protection at both upstream and downstream side of the dam. The new channel shaping is shown in Figure 6.4.

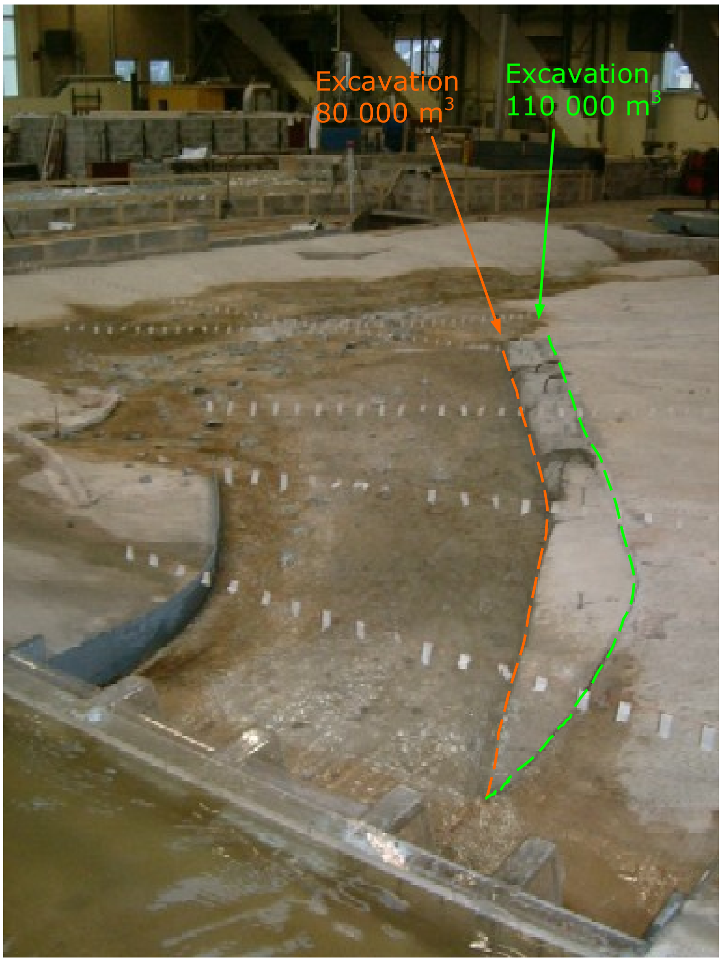


Figure 6.4 Rock excavation downstream of the spillway worked out through model tests

The modified channel eliminated the guide-wall overtopping and erosion in the dam toe and high water levels in the downstream area. The max. wave heights/pressure amplitudes were reduced to ~1.5 m with an average of ~0.5 m. These amplitudes were not considered to cause any severe rock erosion problems.

The construction the new spillway channel would provide the owner with sufficient amount of rock for the new erosion protection on the upstream side of the embankment dams. Adopting any other alternative without excavation meant that some 30 000 m^3 rock must be purchased for the purpose, as well as additional rock volume required for erosion protection of the dam toe.

Model tests conducted for the dam are described in the following reports.

Amnell, G & Billstein, B (2003), Hydraulic model tests 2003 of Harsprånget dam – securing spillway discharge function (*Modellförsök 2003 Harsprånget kraftstation – säkerställande av avbördningsförmåga*), Report No. U 03:87.

Yang, J & Amnell, G (2005), Harsprånget dam - spillway capacity at full and partial gate openings (*Harsprånget - utskovskapacitet vid fri avbördning och olika lucköppningar*), Report No. U 05:02.

7 Höljes

Höljes, constructed during 1959-62, is one of the largest embankment dams in the country, with a max. height of 81 m. The dam is classified in the highest category with regard to dam break consequences, Figure 7.1.

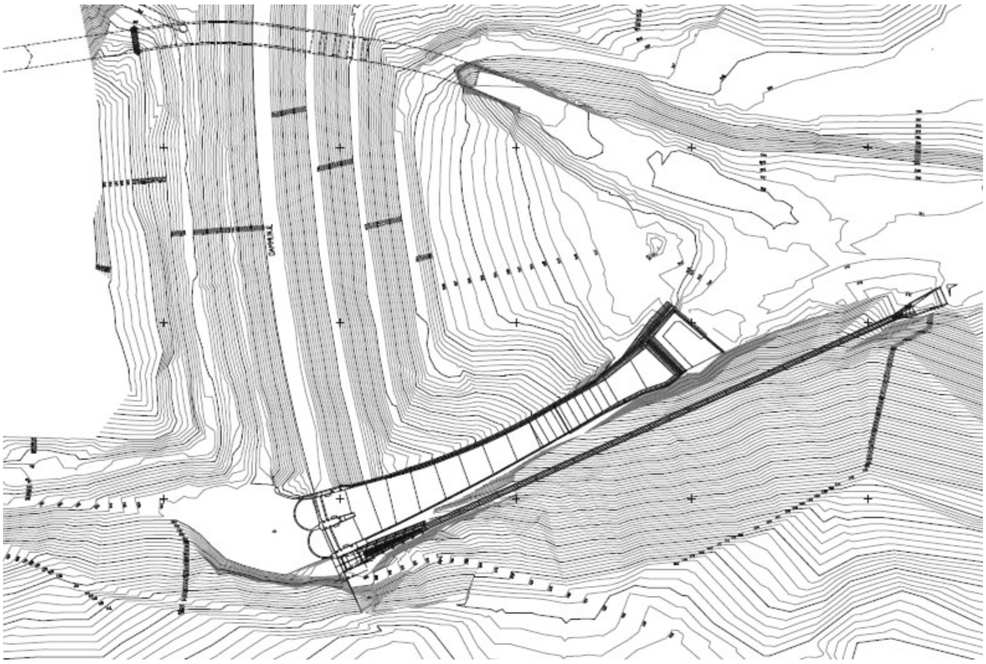


Figure 7.1 Höljes dam - existing layout

The dam consists of the following parts.

- a 400 m long and 81 m high embankment dam (earth and rock fill)
- intake to the power station in the rock at the left shore
- an underground power station
- a bottom outlet at the left shore
- two primary surface spillways and a log flume at the right shore

The power station has the following data:

Design flow	170 m^3/s
Gross head	88 m
Installed capacity	132 MW
Full supply level, FRRL	+304.0 m
Dam crest	+309.0 m
Crest of impermeable core	+305.5 m

In 2006, the dam owner appointed a group of independent international experts to review the dam safety at Höljes. In the Advisory Board Report, a number of potential failure modes were described. In particular, a number of issues with regard to the design flood and the discharge capacity were raised. It was argued that the design flood may have a significantly shorter return period than the stipulated ~10 000 year for high consequence dams, and that a much higher flood had to be accounted for. Moreover, it was argued that the practical discharge capacity might be much lower than the theoretical, which was due to limitations of the stilling basin downstream of the spillways and due to a possible inundation of the tunnel from the bottom outlet.

In response to the remarks by the Advisory Board, the dam owner initiated a major project, starting with a renewed evaluation of all aspects of the dam safety, and aiming at taking the necessary measures for the dam to comply with international standards. Within the project, extensive hydraulic model studies have been made to evaluate the present function of the spillways and to optimize the redesign proposals. Full-scale tests of the spillways will also be carried out before and after the measures.

7.1 Safety concerns

The safety concerns at Höljes are closely related to the function of the existing stilling basin and the erosion risk in the dam toe that is also dependent upon the design flood magnitude.

7.1.1 Design flood

The Advisory Board compared the class I flood at Höljes to the probable maximum flood (PMF) higher up in the river, on the Norwegian side. The comparison gave that a value over 3000 m^3/s may have to be accounted for at Höljes. SMHI has commented that the Swedish method and the Norwegian method for estimating the design flood are not directly comparable. The Norwegian method focuses on a situation with sudden heavy rainfall when the reservoirs are full, whereas the Swedish method focuses on a critical combination of rainfall and snowmelt during the filling of the reservoirs in springtime. When the Swedish guidelines were drafted, it was noted that the Norwegian method normally would give higher values. But it was considered less realistic for the Swedish conditions.

The conclusion from the safety evaluation made during 2007-08 is that it is not enough to provide discharge capacity for the present design flood, but that measures need to be taken to guarantee safe passage of the highest peak inflow flood calculated according to the Swedish guidelines (2000-2100 m^3/s, depending on how the areal data base is judged).

7.1.2 Existing spillway and stilling basin

Höljes has three spillways: one bottom outlet, one primary surface spillway (with two tainter gates) and one log flume. The dimensions and discharge capacities of the spillways are given in Table 7.1. The discharge capacity of the spillway has been verified through ongoing hydraulic model studies. The discharge capacities of the bottom outlet and the log flume were established in 1958-59 hydraulic model studies.

Table 7.1 Spillways at Höljes

Spillway	Gate type	Width [m]	Height [m]	Sill level [m]	Discharge capacity at FRRL [m^3/s]	Discharge coefficient μ
Bottom outlet	Radial	2.5	3,9	+263.4	185	0.89
Primary spillway	Radial	2 x 14	-	+295.3	1290	0.61
Log flume	Sector	8.0	-	+299.2	120	0.48

About 80 % of the discharge through the log spillway is guided over from the log flume to the primary spillway through holes in the bottom of the flume. The total flow that reaches the stilling basin beneath the primary spillway is hence ~1400 m^3/s if all gates are fully open when the reservoir is full. During the hydraulic model studies 1958, the stilling basin for the primary spillway was designed to give sufficient energy dissipation up to a flow of ~1150 m^3/s, which according to that model was the total discharge capacity of the two radial gates. It was not considered necessary to take simultaneous discharge through the radial gates and the log flume into account, since an "exceptional flood" at that time was estimated to be in the order of 1000 m^3/s. In the final design of the stilling basin, it was however made both shorter and narrower, giving a capacity of less than 900 m^3/s, Figure 7.2.

Figure 7.2 Spillway flood release 900 m^3/s, September 17, 2008

The basin is divided into two parts – a deeper and narrower upstream part and a shallow and wider downstream part, Figure 7.3. The separating wall is lower on the right side, thus concentrating the flow towards the right-hand wall. The energy dissipation is primarily by impact since the depth and length of the basin are insufficient for proper hydraulic action and since the incoming flow is nearly vertical.

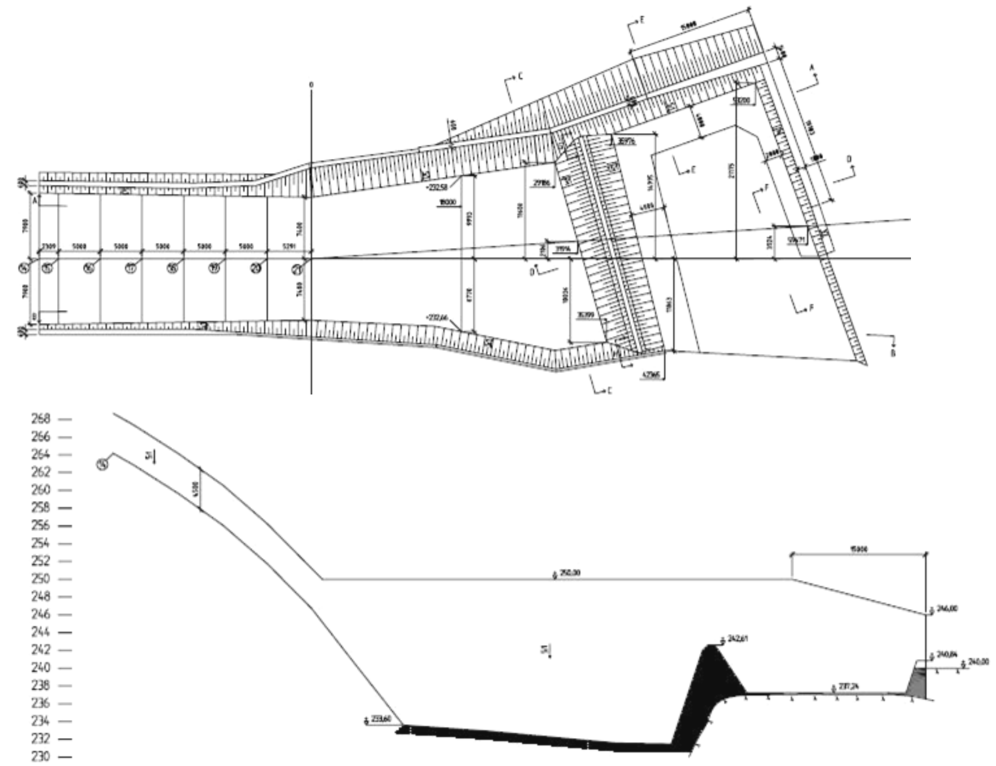


Figure 7.3 Layout of existing energy dissipator

7.1.3 Practical discharge capacity

The bottom outlet discharge is conveyed via the old diversion tunnel, in which a hydraulic jump occurs. The Advisory Board discussed the risk that the diversion tunnel outlet could get inundated at high discharges. The free aeration for the hydraulic jump would then be cutoff, which could lead to large-scale air pulsations and dangerous shock pressures. In the evaluation made during 2007-08, it has been shown that the theoretical downstream water level at 2000 m^3/s is about 1 m below the ceiling at the tunnel outlet.

However, it can be argued that erosion downstream of the stilling basin could lead to the creation of a berm, which could raise the water level further. The Board suggested that this might have happened before. The river bed is most likely not stable for flows in the order of 2000 m^3/s. Below the upper layer of stones and rocks in the river bed, there is 10-15 m overburden material that is believed to be easily erodable. There is hence potential for deep erosion pits. The safety evaluation concluded that the bottom outlet shouldn't be used when the total discharge is in the order of 2000 m^3/s.

For discharge of 2000 m^3/s through the primary spillway and the log flume, without the use of the bottom outlet, the ongoing model studies indicate that a reservoir level 1.8 m above full supply level could be sufficient. This is 0.3 m above the crest of the impermeable core and 3.2 m below the dam crest. With only moderate measures (raising of the impermeable core and raising of guiding concrete walls at the spillway) this reservoir level would be acceptable for a sufficiently long time without any risk for dam failure. However, presently the discharge that can be sustained in practice in the primary spillway is limited for several reasons:

- The capacity of the stilling basin is less than half of the capacity needed. Sustained discharge in excess of ~900 m^3/s is likely to cause erosion in the river bed and at the shores. The present instructions that the primary spillway may be used only up to 700 m^3/s. Above this limit, first the bottom outlet and then the log flume should be used to their full capacity. Should discharge above 1000 m^3/s be needed, continuous observation of the downstream area is prescribed.
- The structural integrity of the basin, in particular the left-hand wall, is questionable. The wall is dependent on unstressed rock anchors for its stability. The condition of the anchors is unknown. Should the wall collapse at high discharge, there is potential for erosion towards the dam toe.
- The rock wall at the right side of the spillway chute is heavily fractured. The rock bolts are old and many of them have likely lost their function. Should large pieces of rock fall into the chute, there is a great risk for damage on the concrete in the chute and in the stilling basin. If this happens at high discharge there is a risk that the water takes alternate paths towards the embankment dam.
- The amount of debris that could reach the spillways during a flood has not been estimated. The amount at normal conditions is very small at Höljes, but in principle there is potential for debris problems if the reservoir is raised above full supply level since large parts of the reservoir is surrounded by forest.

7.2 Proposed rebuilding alternatives

During 2008, different measures were evaluated for increasing the discharge capacity and safely discharging the design flood. Three alternatives were analyzed through hydraulic model tests:

- Alternative A - raising the impermeable core of the dam and enlarging the stilling basin for the primary spillway, to increase the capacity of the existing spillway.
- Alternative B1 - constructing a new, separate tunnel spillway to the right of the dam and the existing spillway.
- Alternative B2 - replacing the log flume with a second primary spillway and widening the chute.

7.2.1 Alternative A

Alternative A includes raising the impermeable core of the dam and concrete structures around the spillway, so that a sufficiently high reservoir level for discharge of 2000 m^3/s can be accepted for a limited amount of time. In this alternative, no new spillway is constructed and the capacity of the bottom outlet is not included. The layout is shown in Figure 7.4.

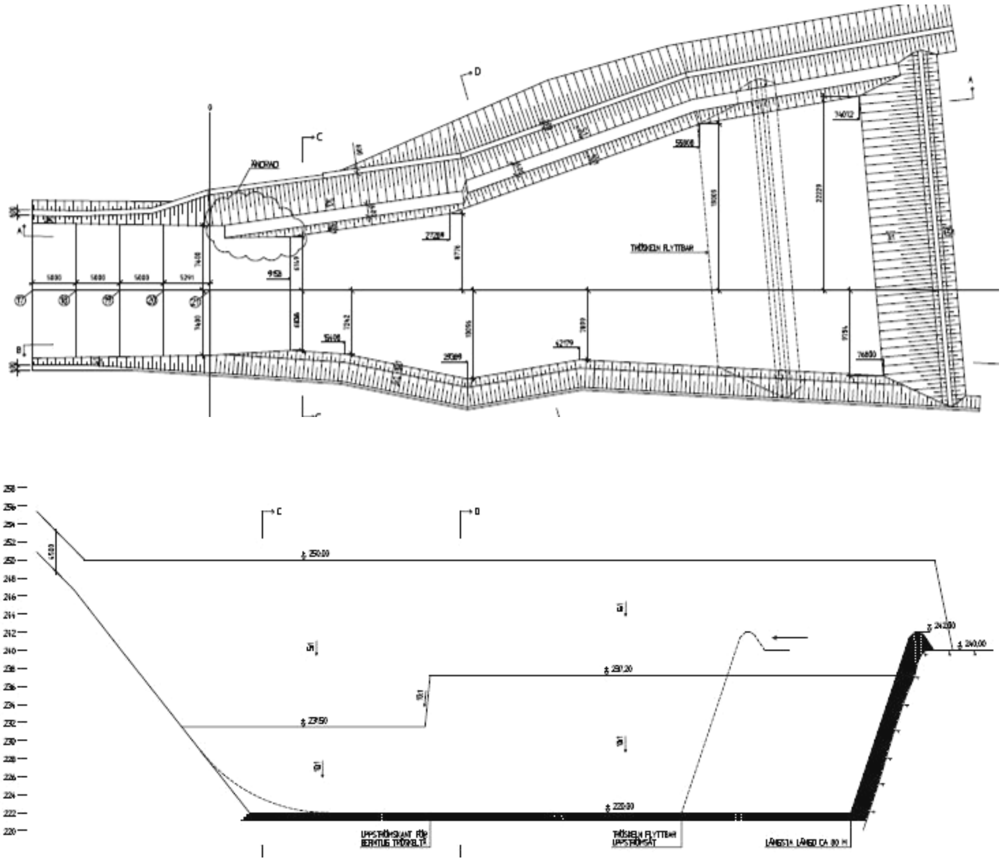


Figure 7.4 Alternative A – enlargement of the stilling basin

More extensive modifications are needed on the downstream side.

- The left guiding wall of the spillway chute has to be raised by 1-3 m.
- The stilling basin has to be extended by about 40 m and made 5-10 m deeper. Baffle blocks should also be installed on the bottom of the basin.
- The left wall of the stilling basin has to be raised by 3-5 m and extended downstream.
- Erosion protection is needed on the river bed downstream of the energy stilling basin and on the left shore.

In spite of these measures, this alternative is expected to be by far the least expensive. The disadvantages are:

- During construction, only the bottom outlet (discharge capacity 185 m^3/s) and the power station (design flow 170 m^3/s) can be used. If the inflow is greater than this over some period of time or if the power station for some reason needs to be shut down, the construction works have to be interrupted and the construction area evacuated.
- All the discharge is conveyed in the same spillway chute (except for the small part continuing in the log flume). A major slide in the rock wall to the right of the chute could be critical.
- The stilling basin is founded on rock on the right side of the river. The sill of the stilling basin is located on the same level as the riverbed, but 10-15 m above the bedrock in the center of the river. There is hence potential for deep erosion pits downstream of the stilling basin, if for some reason the energy dissipating function would be diminished.
- All the discharge is through three gated spillways with all their capacity used, while at the same time the reservoir is at a very high level. Mechanical problems or problems with debris could then be critical.

Against the latter point, it can be argued that the rate of increase in the reservoir level is small at Höljes, only about 0.4 m/h at an inflow of 2000 m^3/s without spillway discharge. There is also the possibility to use the bottom outlet at least up to a total discharge of about 1500 m^3/s.

7.2.2 Alternative B1

The third alternative involves a new spillway with a tunnel to the right of the log flume, Figure 7.5. The log flume is untouched and can be used for discharge. The new spillway is designed for a discharge of ~600 m^3/s (total capacity 2000 m^3/s at the FRRL, with log flume and existing primary spillway, but without bottom outlet). A new stilling basin is constructed at the exit from the tunnel about 200 m downstream of the existing stilling basin. The existing stilling basin is re-sized to allow a discharge of ~1400 m^3/s.

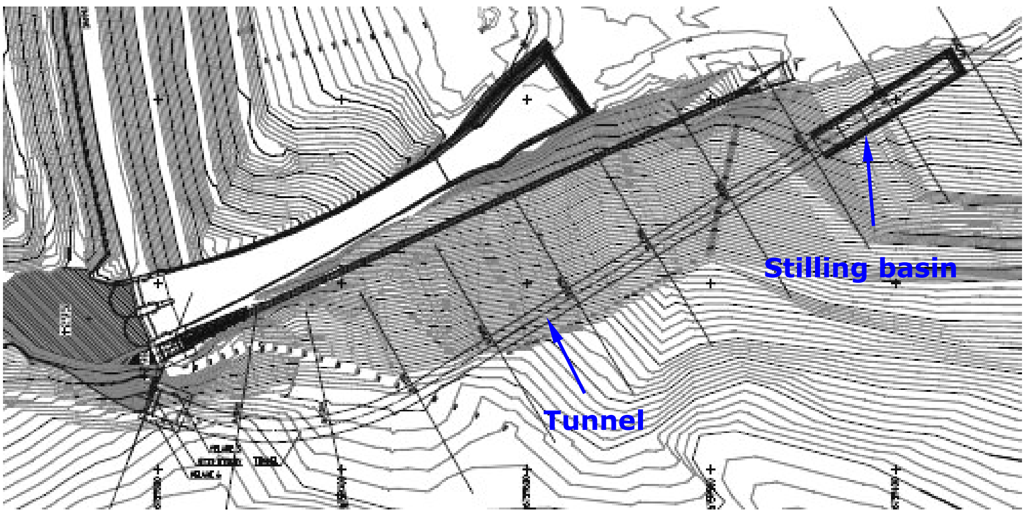


Figure 7.5a Alternative B1 – new tunnel spillway

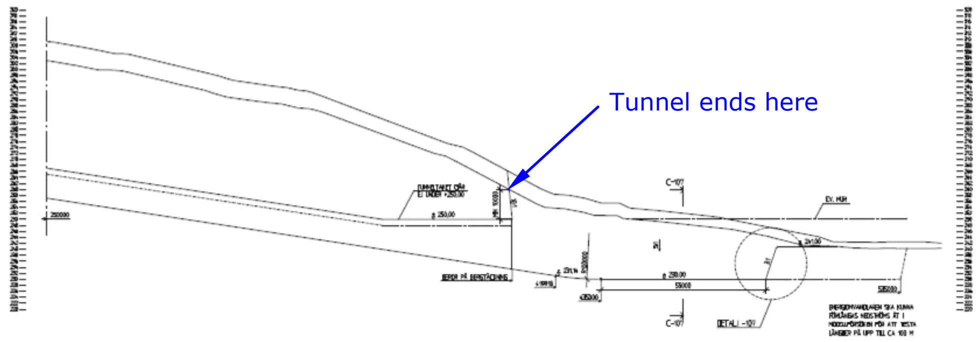


Figure 7.5b   Alternative B1 – new tunnel spillway

This alternative has the obvious advantage that a new, separate spillway is provided. The discharge is then no longer dependent on one single chute and stilling basin. This is important both during the construction phase and afterwards. Another important advantage is that parts of the discharge enters the river further downstream, far away from the dam and in a less erosion sensitive section of the river. The obvious disadvantage is the cost. Even with this new spillway, measures are needed for the existing spillway that include re-shaping the stilling basin and stabilization of the rock wall to the right of the spillway chute) On top of this are the costs for the new spillway, the tunnel and the new stilling basin.

7.2.3 Alternative B2

This alternative involves replacing the log flume with a new 17 m wide gated spillway and with the same sill level as the present spillway. The discharge capacity is estimated to 710 m^3/s, which gives a total capacity of 2000 m^3/s at the FRRL, without the use of the bottom outlet. The existing spillway chute and the stilling basin are widened to the right, in proportion to the increased discharge, Figure 7.6. The basin also needs to be extended downstream and deepened in the downstream part. There is probably no need to raise the left wall of the spillway chute, since the unit discharge is unchanged. The left wall of the stilling basin may have to be raised slightly.

This alternative is expected to be more expensive than alternative A but less expensive than alternative B1. The disadvantages are the same as for alternative A, but to a lesser degree since there is one more gate and a lower discharge per m width.

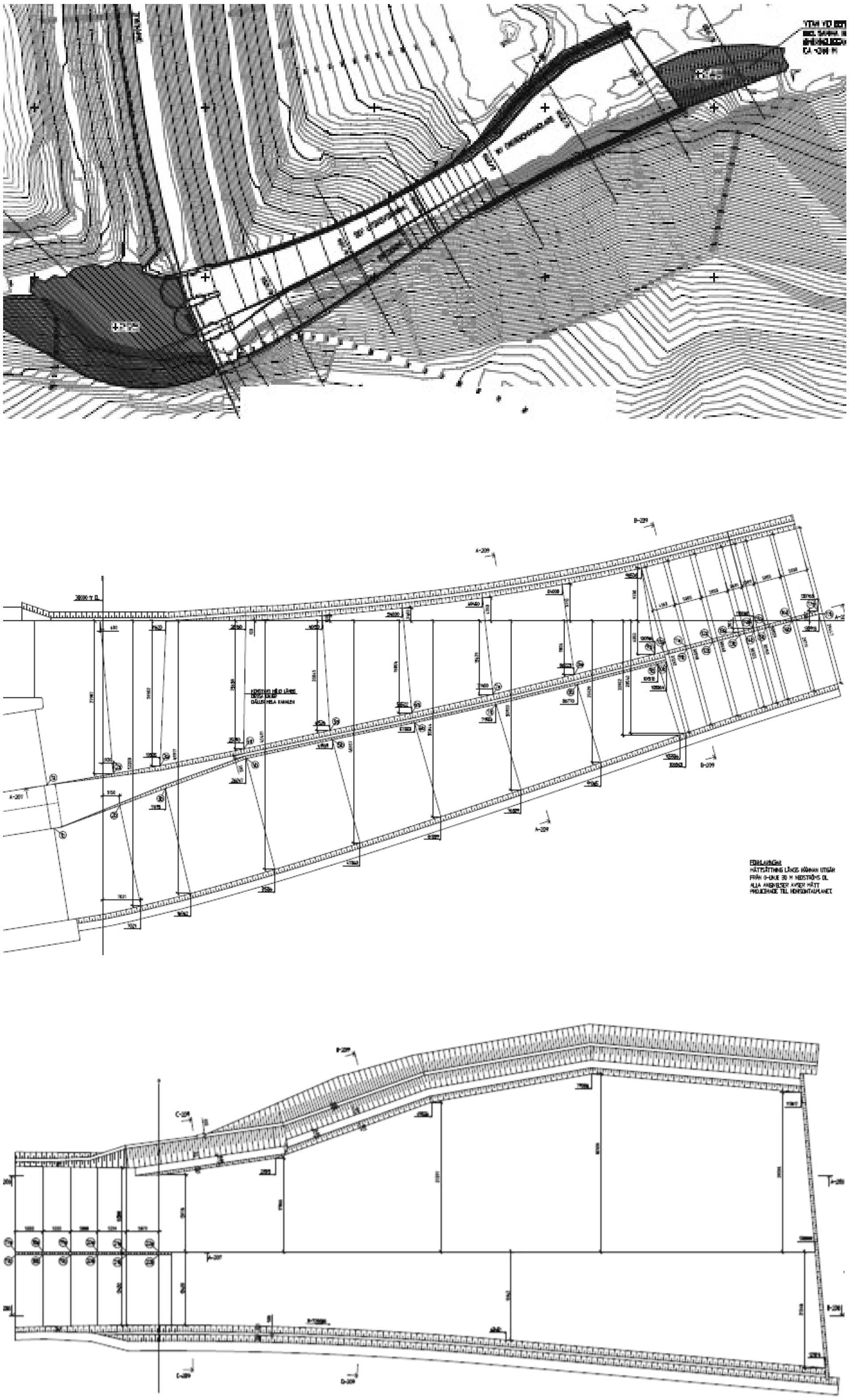


Figure 7.6 Alternative B2 – widening of the exiting waterway

7.3 Model tests

A base model, representing the existing dam design including the flood discharge structures, was first constructed, Figure 7.7. It was built in scale 1:50 and was ~25 m long, ~10 m wide and 2 m high. The riverbed was shaped in concrete; the spillway chute and stilling basin were built with 3-mm sheet metal. The use of sheet metal enabled quick modifications of the model, so that a number of configurations could be evaluated.

The design principle was that the design flood ~2000 m^3/s should be safely discharged at or below the FRRL. A safety margin would be then created for possible higher design floods in the future. The following hydraulic aspects were evaluated for each alternative:

- Spillway discharge capacity
- Geometry of the spillway chute or tunnel and its optimization
- Energy dissipation and optimal configuration of stilling basin
- Flow pattern in the river downstream
- Erosion risks in the river valley and towards the dam toe

Figure 7.7 Base model for Höljes, with existing spillway layout

7.3.1 Existing layout

The tests indicated that, when fully open, the Höljes spillway, including the log flume but not the bottom outlet, could discharge ~1300 m^3/s at the FRRL (+304,0 m), and a reservoir level of 2.2 m above the FRRL would be needed for discharging the design flood. This was 0.7 m above the impermeable core crest. Without any extra spillway, the release of the design flood would take place with extra storage above the FRRL.

The water levels in the existing spillway channel were relatively stable, without noticeable fluctuations up to the discharge 1600 m^3/s. At 2000 m^3/s, the channel configuration gave rise to uneven, locally high water surfaces. Taking account of the effect of air entrainment, the left channel wall must be raised to avoid overtopping and potential erosion behind it.

The function of the existing stilling basin was satisfactory up to a discharge of ~1000 m^3/s, some 100 m^3/s more than originally designed for. Starting from the discharge 1000 m^3/s, the function gradually deteriorated; above 1600 m^3/s, the function was basically lost, Figure 7.8.

The outflow into the river from the basin was characterized by large water level differences and high-intensity turbulence. The flow velocity along the right river bank downstream of the stilling basin was as high as 10-12 m/s and strong wave motions occurred along the erosion sensitive left bank.

Figure 7.8 Flood discharge in existing energy dissipator, 1600 m^3/s

7.3.2 Alternative A

No modifications were made in the spillway channel, except that the left sidewall height needed to be increased for discharges above 1600 m^3/s.

The existing stilling basin has a length of ~60 m and a bottom elevation +232 m upstream and +237 m downstream. To increase the efficiency, the downstream part was first given the same elevation as the upstream, the basin was extended to a length of 80 m. By slight modifications of the geometry, it was possible to get satisfactory dissipation up to ~1400 m^3/s.

The basin was then extended by another 20 m and was made 10 m deeper (from the elevation +232 m). The separating wall in the basin was removed. This gave satisfactory energy dissipation up to 1600 m^3/s and somewhat acceptable energy dissipation up to 2000 m^3/s. However, a strong bottom current in the basin disrupted the flow pattern, resulting in strong waves and large water level differences downstream.

To achieve sufficient energy dissipation at the design flood, the stilling basin must be extended to a total length of 120 m and deepened with 10 m.

Tests were made with four rows of baffle blocks on the basin bottom. The blocks proved to be efficient and good results were obtained for discharges up to 2000 m^3/s, Figure 7.9. By using the blocks, the extension could be 20 m shorter and 2.5 m shallower, i.e. the basin was 100 m and 7.5 m deeper.

Figure 7.9 Alternative A - stilling basin with a length of 100 m, a 7,5 m lower bottom and baffle blocks, 2000 m^3/s

If the transition between the steep spillway channel and the stilling basin was rounded with a radius so as to reduce the jet impacts on the bottom, the stilling basin needed probably to be lengthened to 120 m and deepened with 10 m. This was however just a qualified guess, as no tests were made for this situation.

7.3.3 Alternative B1

A new, separate gated spillway, discharging into a tunnel, was suggested. The spillway opening had a width of 14 m and the same threshold elevation as the existing spillway, +295.3 m. The finished model is shown in Figure 7.10. The tunnel was roughened with 8–12 mm balls to imitate the prototype construction requirements.

Tests showed that its discharge capacity was ~630 m^3/s at the FRRL if other openings were in closed position; the total discharge capacity of the dam increased thus to 2000 m^3/s. The proposed 70 m long stilling basin functioned satisfactorily up to 630 m^3/s, Figure 7.11.

The existing stilling basin needed to be re-shaped to safely withstand a discharge of 1400 m^3/s. Tests demonstrated that the basin must be prolonged to some 100 m and deepened with 7.5 m. If baffle blocks were used, the basin needed to be extended to 80 m, with a 7.5 m lower bottom.

Figure 7.10 Alternative B1 – tunnel spillway to the right of log flume

Figure 7.11 Alternative B1 – hydraulic jump & energy dissipation at tunnel outlet

If the steep spillway channel passed to the stilling basin with a radius, the basin needed to be 105 m long, 10 m deeper and equipped with baffle blocks. Again, this is a qualified guess; no tests were made.

7.3.4 Alternative B2

In this proposal, a new spillway opening, placed to the right the existing ones, was recommended. The log flume was removed to give place to it. Both the existing spillway channel and the energy dissipator were widened to the right by some 100%. With the addition of this spillway, the design flood could be discharged at the FRRL.

In the first proposed layout, the whole width of the widened channel was given the same elevation in cross-section. Tests pointed to the presence of cross waves and wave reflections in the channel, which was un-desirable, Figure 7.12.

Based on previous engineering experiences, a channel-bottom elevation difference in cross section, roughly along the right side-wall of the existing channel, was suggested. Besides, a partition wall was added along the upper part to reflect more water to the left. There were also needs to extend and optimize both the existing and new spillway piers in order to obtain satisfactory flow patterns in the channel up to the design flood, Figure 7.13.

The extension of the existing pier was based on the shape of the spillway at Alqueva dam outside Lisbon.

Different ways of re-shaping the stilling basin were tested in the model. No matter how the basin was deepened, its left sidewall must be positioned in such a way that planar flow circulations were avoided. This meant that the originally proposed wall location had to be adjusted and moved somewhat to the right. To achieve effective energy dissipation, the basin needed to be prolonged to at least 105 m, deepened with 10 m and with baffle blocks on the bottom. To avoid the penetration of the strongly pulsating water flow into the joint between the channel and the basin, a radius must be used.

Figure 7.12 Alternative B2 – Non-uniform flow with cross-wave if the channel bottom given the same cross-sectional elevation.

At the design flood, the approaching flow conditions at the spillway and the energy dissipation for the final chosen layout for alternative B2 are shown in Figure 7.14.

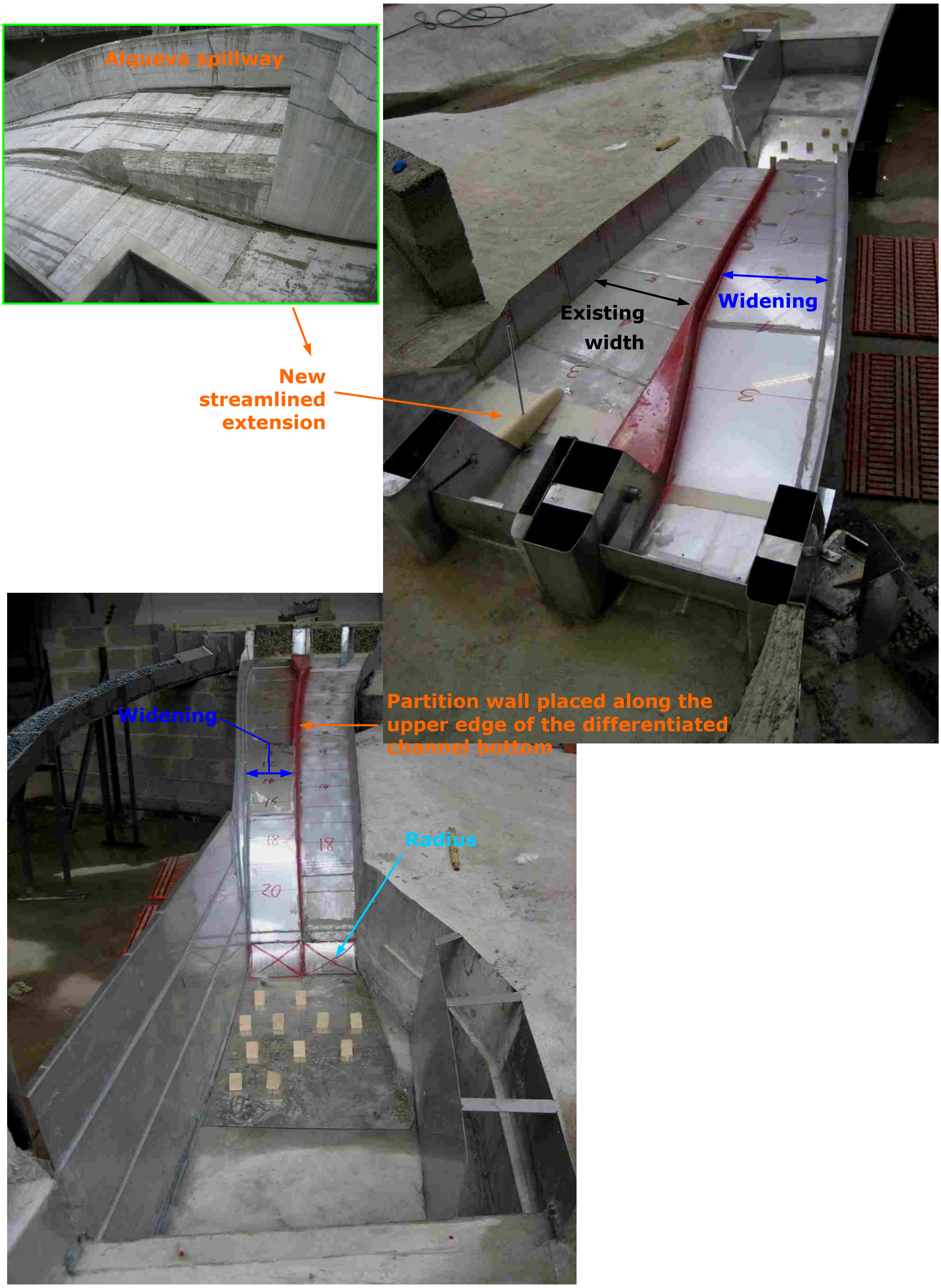


Figure 7.13 Alternative B2 – new spillway to the right of the existing with widened spillway channel

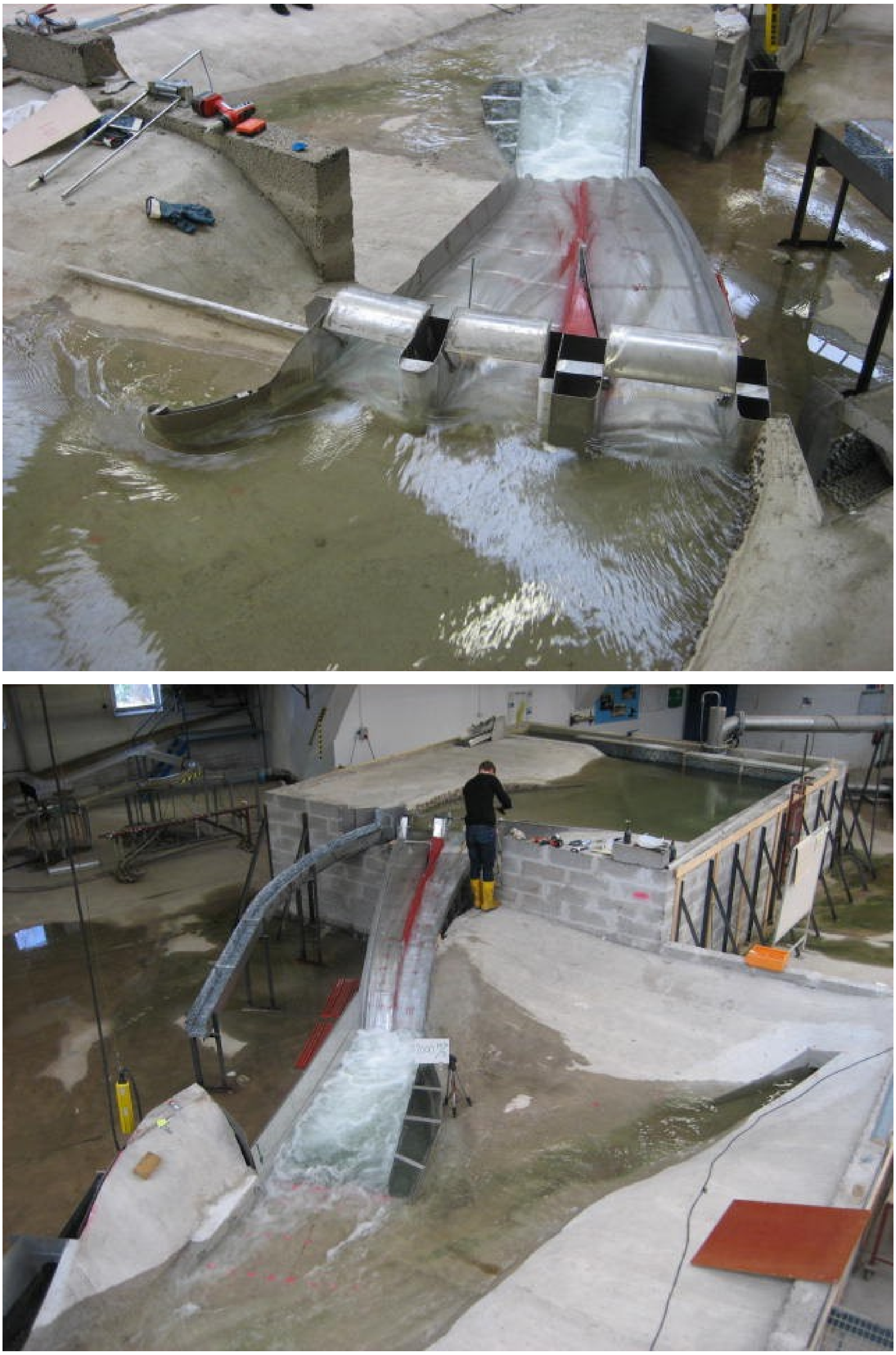


Figure 7.14 Alternative B2 – Flow in the final worked-out layout

7.4 Comments

Model tests made for the Höljes dam rebuilding project were extensive. A large number of test combinations were involved for both the existing layout and each proposed alternative. What is described in the chapter is only some test summaries – interested readers should refer to the project report for more detailed descriptions of the conducted tests.

The following publications describe the model tests conducted for Höljes.

Sundqvist, P & Yang, J (2007), CFD calculations of flow behaviors in spillway channel of the Höljes dam *(CFD beräkning av strömningsförhållanden i utskovskanal i Höljes damm),* Report No. U 07:14.

Yang, J & Persson, J (2009), Höljes dam safety – hydraulic model investigations of dam rebuilding measures (*Höljes dammsäkerhet - modellförsök med säkerhetshöjande åtgärder*), Report No. U 08:112.

Stenström, P, Yang, J, Bond, H, Sjödin, A & Steiner, R (2009), "Increasing the discharge capacity at the Höljes dam in Klarälven, Sweden", 25th ICOLD Congress, May 2009, Brasilia.

8 Laxede

Laxede is classified as a high-hazard dam according to the current dam-safety guidelines, Figure 8.1. The dam consists of power station, spillway and connecting embankments with a central impervious core of moraine that extend upstream on both left and right sides. The max. dam height is roughly 24 m. The left embankment dam is 460 m long; the right one is 580 m long and is connected on to a sealing blanket stretching to the highway bridge located some 1000 m upstream of the dam.

The spillway section is located to the right of the power station and consists of three gated overflow openings of 15 m wide each. The total discharge capacity corresponds to ~2800 m^3/s at the FRRL. The updated design flood amounts to some 3200 m^3/s.

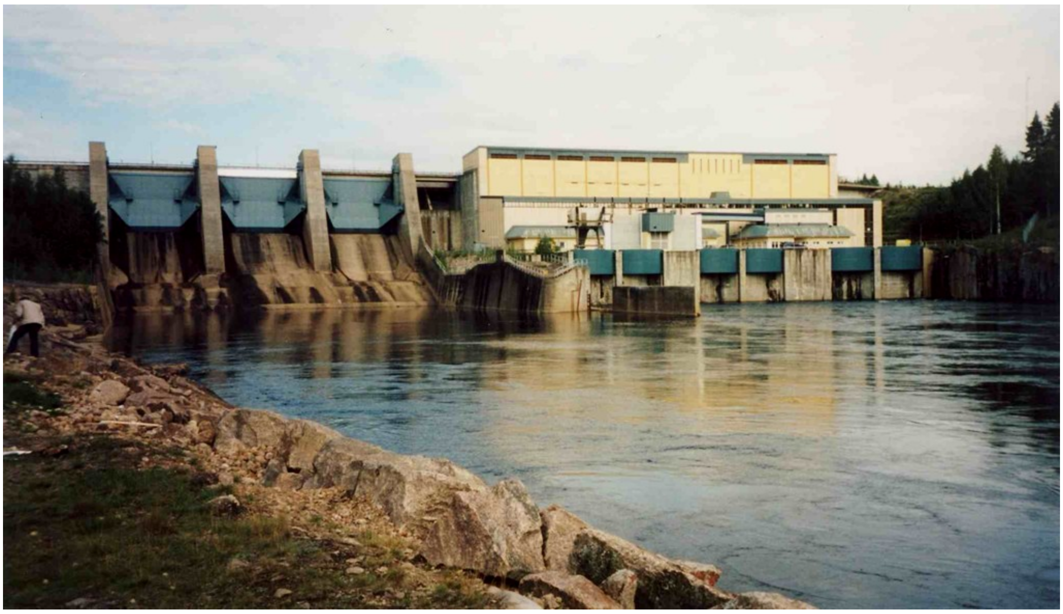


Figure 8.1 Laxede dam, with spillway located to the right of the power plant

The hydraulic model of Laxede, built winter 2006, was used to evaluate the hydraulic safety of the spillway discharge up to the revised design flood and to verify proposed dam-safety measures. The modeled area is illustrated in Figure 8.2. The model, shown in Figure 8.3, was constructed in a scale of 1:60 and covered the whole river width and a river length of 1200 m upstream and 400 m downstream, implying that the model was about 25 m long and 11 m wide. The power station and the highway bridge were also included in the model. Water was fed separately to the spillway and the power station.

Figure 8.2 Modeled reach covering 1200 m upstream and 400 downstream

Figure 8.3 Model of Laxede, built in scale 1:60, 25 long and 11 m wide

8.1 Examined dam-safety issues

From the dam-safety point of view, the following issues were examine in the Laxede model.

- Spillway capacity, free discharge
- Spillway capacity at varied gate openings
- Freeboard under the highway bridge
- Freeboard in spillway openings
- Flow pattern & water-surface profile in the reservoir
- Effects of rounding off of log flume intake on spillway discharge
- Risk of erosion at the highway bridge
- Effect of turbine flow on energy dissipation
- Energy dissipation at normal downstream water stage
- Sensitivity analysis of energy dissipation - limiting water stage

8.2 Measures against floating debris

The Laxede model, with some modifications, provided a suitable platform for examination of floating debris in a laboratory environment. The spillway in Laxede has three 15-m gated openings. In terms of opening width and head, its size is representative among the Swedish dams, Table 8.1.

Table 8.1 Typical gated spillway dimensions in North Sweden

Dam	No of openings	Opening width (m)	Head (m)
Laxede	3	15	10.4
Porjus	2	15	11.0
Harsprånget	3	20	16.6
Letsi	2	15	9.7
Ligga	3	20	7.6
Långbjörn	3	15	7.0
Porsi	3	15	10.4
Stornorrfors	4	15, 21	9.6
Bergeforsen	3	15	9.3

The floating debris used consisted of ~130 spruce and pine trees, with a median of 40 cm and a standard deviation of 8 cm. The median was thus ~1.6 times the spillway width. It was chosen in such a way that it roughly corresponded to typical tree heights of 20–30 m along the rivers. The density of most trees fell within 850–950 kg/m^3. The trees were randomly released at different locations upstream in the model, often one by one.

With a limited number of trees in a river, to improve the debris passing capacity is a probably meaningful thing to do. It happens, however, seldom that solitary trees approach the spillway one after another during a high flood. When there is a huge amount, especially if two or more trees approach the spillway at the same time, debris blocking of the spillway becomes often a fact and cannot be avoided. The possibility of removing debris from a blocked spillway is almost non-existent. To provide excess discharge capacity often implies large engineering costs. Spillway debris handling in the existing dams often implies that the spillway capacity shall not be reduced significantly with the presence of debris and the flood is released safely without any human interference.

The possibility of using debris booms in Laxede is first investigated. Figure 8.4 shows the flow pattern in the Laxede reservoir at the deign flood (results from the program Surfacewater Modeling System (SMS), based on shallow-water equations). Different locations of placing a debris boom were tested, including anchorage from the riverbanks and from a buoy in the middle of the main stream. The narrow bridge opening caused a flow velocity amounting to 2 – 4 m/s throughout the reservoir. Debris temporally stopped by the boom would be dragged down by the water and pass the boom from underneath. Due to the concentrated high flow velocity in the reservoir, the use of shear booms proved to be unsuitable.

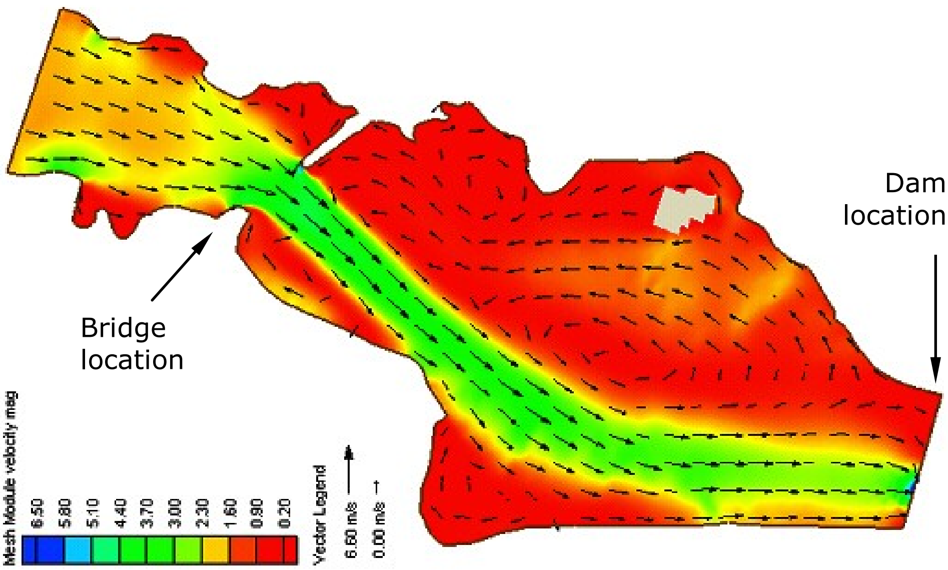


Figure 8.4 Flow pattern in Laxede at spillway discharge Q = 3200 m^3/s

8.2.1 Devising debris visors

Two types of debris visors or racks were devised to stop the debris in from of the spillway – one was a semicircular visor with sloping beams supported on a platform and the other was straight resting on the spillway bridge, Figure 8.5.

Figure 8.5 Debris visors placed at the spillway

The semicircular visor is composed of sloping beams resting on a semicircular platform that has a diameter roughly as large as the length of the spillway section. The platform is placed on the same elevation as the spillway bridge. In the prototype size, the platform has an inner radius of R = ~30 m and a width of a = 4 m. The distance between the neighboring beams is b = ~10 m at the platform. The beam width is c = 0.6 m. On the left side, the two beams close to the spillway are given a steeper slope of α = 50 – 65°, as they would otherwise intersect the flow passage of the power plant intake. On the right side, the two beams close to the spillway are given a slope of α = 25° as they support themselves on the embankment slope. The slope of the remaining beams varies in the interval of α = 20 – 30°.

As for the straight visor, it comprises a number of sloping beams placed directly on the spillway bridge and covers the whole spillway section. The distance between two neighboring beams is roughly half of the spillway opening width. All the beams are given the same sloping angle, typically falling within α = 20 – 30°.

In both cases, the sloping beams run all the way to the river bottom. In practice, this is not necessary as they would be very long. They can instead extend a couple of meters below the water surface, and then support themselves on vertical beams.

8.2.2 Semicircular debris visor

With the semicircular visor, the situation with the trees that accumulate upstream of the visor is pictured in Figure 8.6; the comparison of the discharge capacity without and with the debris is plotted in Figure 8.7.

Figure 8.6 Capture of floating debris with semicircular visor

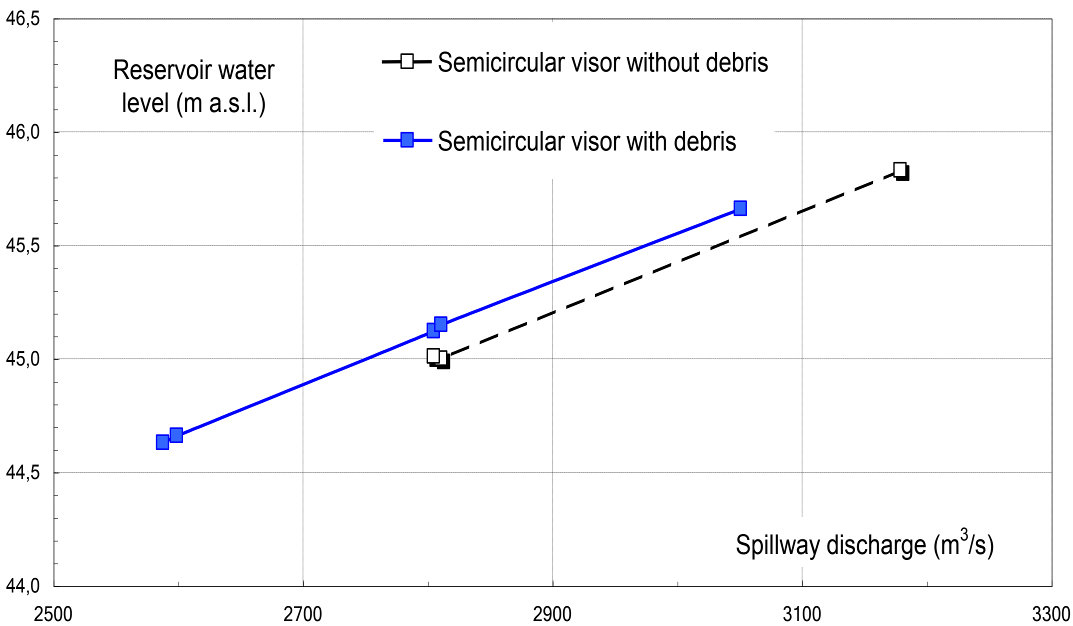


Figure 8.7 Semicircular visor - spillway discharge with floating debris

Repeated measurements in the model demonstrate that the visor itself affects somewhat the discharge capacity. It is however difficult to determine exactly its influence. Its effect seems to be inconsiderable and falls within the margin of error of the flow measurement.

Depending upon how the trees approach the spillway, a solitary tree can pass the visor. The tree that passes the visor passes often the spillway, as the visor can align the tree onto the horizontal flow direction. If a few trees get wedged on the visor, a jam is built up in a short time and the trees that follow will be all stopped. The visor keeps the semicircular area in front of the spillway free from the debris. That is the reason why the discharge capacity is affected marginally by the debris.

With the semicircular visor and the 130 trees, the discharge capacity is approximately 2750 m^3/s at the FRRL, which corresponds to a reduction in the capacity by 55 – 65 m^3/s (about 2%). As expected, it is easier for lighter trees to glide on the sloping beams. Heavier trees have the tendency to be dragged down and become submerged. The density of the trees plays therefore an important role in its behavior upstream of the visor.

8.2.3 Straight visor placed on spillway

Figure 8.8 illustrates the accumulation of the debris at the straight visor on the spillway bridge, while Figure 8.9 illuminates the reduction in the spillway discharge capacity due to the debris.

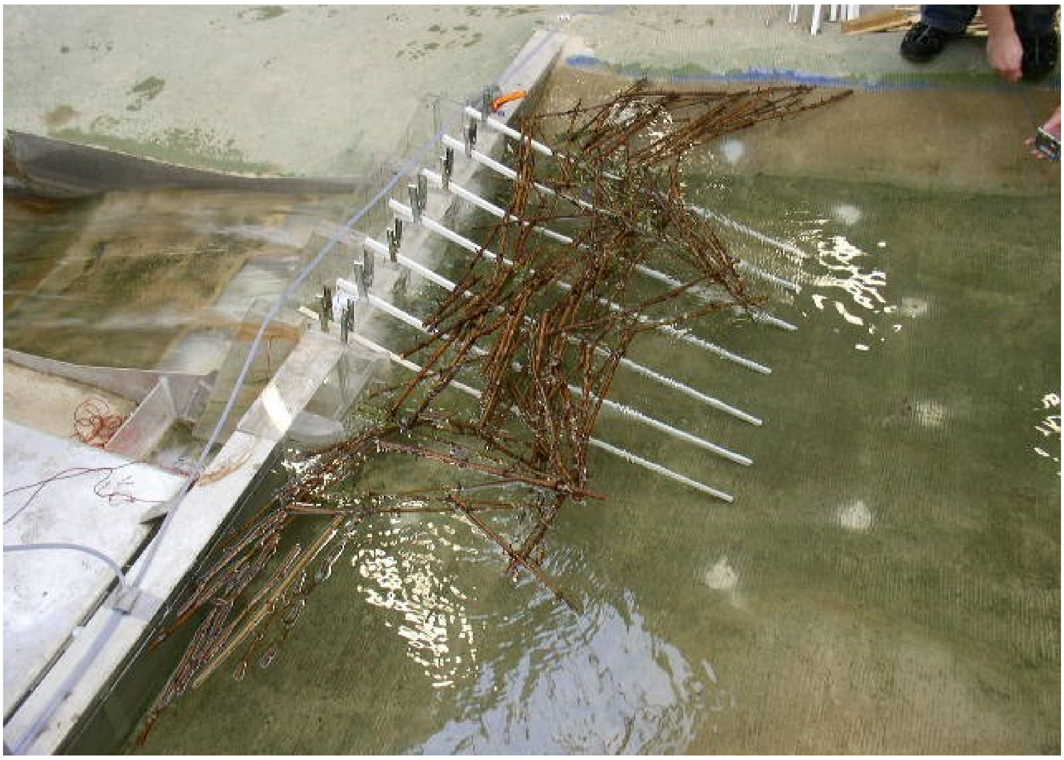


Figure 8.8 Capture of floating debris with straight debris visor

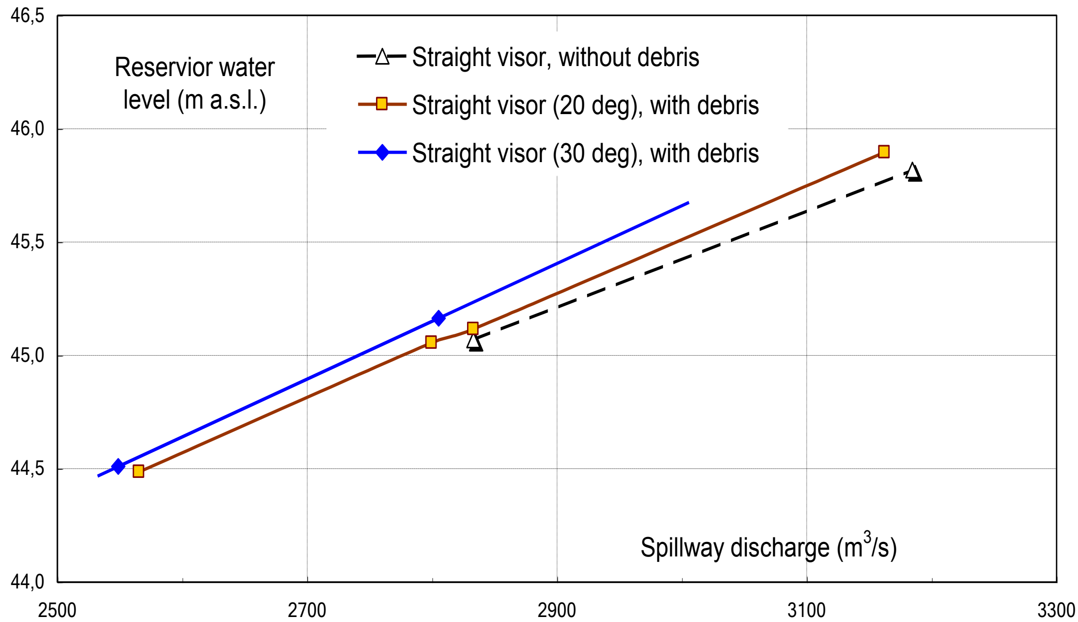


Figure 8.9 Straight visor - spillway discharge with floating debris

The visor blocks up effectively the trees and keeps the spillway openings free from the debris. In other aspects, the straight visor functions as satisfactorily as the semicircular visor. The sloping angle of the beams affects to some extent the discharge capacity. For α = 20°, the reduction in the capacity corresponds to ~35 m^3/s (1,2%) at the FRRL; for α = 30°, the reduction is ~70 m^3/s (2.4%). In despite of this, the effect of the debris on the capacity is insignificant.

From the practical point of view, there exists potential to optimize, within certain interval, the sloping angle of the beams, especially when the debris density is taken into account.

The model tests have demonstrated the design concept of using the visors to stop the floating debris upstream of the spillway, maintain relatively free spillway flow and a marginal reduction in the discharge capacity. From the practical point of view, the resulting forces from the visors must be taken into consideration so as to guarantee the overall structural stability of the spillway. The tests are made in the Laxede hydraulic model. However, many general conclusions can be drawn.

When model studies are to be made for a specific dam, one should have reliable field data of the trees in the catchment. The tree lengths are, in relation to the spillway dimension in question, a governing parameter. In a model, the debris flow should be simulated as close to reality as possible. Due to the random nature of the floating debris movement in quickly moving water, the uncertainty of such parameters as debris shape, length, density etc. deserves attention and should be quantified.

The following publications deal with the model tests for the dam.

Yang, J, Amnell, G, Skärberg, P, & Bergsten, M (2007), Rehabilitation of Laxede dam for higher safety, hydraulic model studies *(Laxede ombyggnad för ökad dammsäkerhet, hydrauliska modellförsök)*, Report No. U 07:15.

Amnell, G & Yang, J (2007), Laxede spillway – testing of spillway capacity through model tests *(Laxede utskov – bestämning av avbördningsförmåga genom modellförsök),* Report No. U 07:34.

Yang, J (2008), Hydraulic model tests of floating debris in Laxede hydraulic model *(Modellförsök med drivgods i Laxede modell),* Report No. U 08:24.

Yang, J, Johansson, N & Cederström, M (2009), "Handling reservoir floating debris for safe spillway discharge of extreme floods – laboratory investigations", 25th ICOLD Congress, May 2009, Brasilia.

9 Letsi

Letsi dam was constructed during 1967-70. The power plant, excavated in rock and located in the left riverbank, consists of three generating units, equipped with Francis turbines, with a total turbine discharge of 390 m^3/s. Two of them were commissioned in 1967 and the third in 1970. The total rated effect is 450 MW at ~130 m head.

The dam is of rock-fill type, having a max. height of 85 m and a crest length of ~550 m, Figure 9.1. It is grounded on solid rock and has a conventionally formed vertical impervious core of moraine, surrounded by filter and rock fill. The dam axis is given a slightly convex form in the upstream direction.

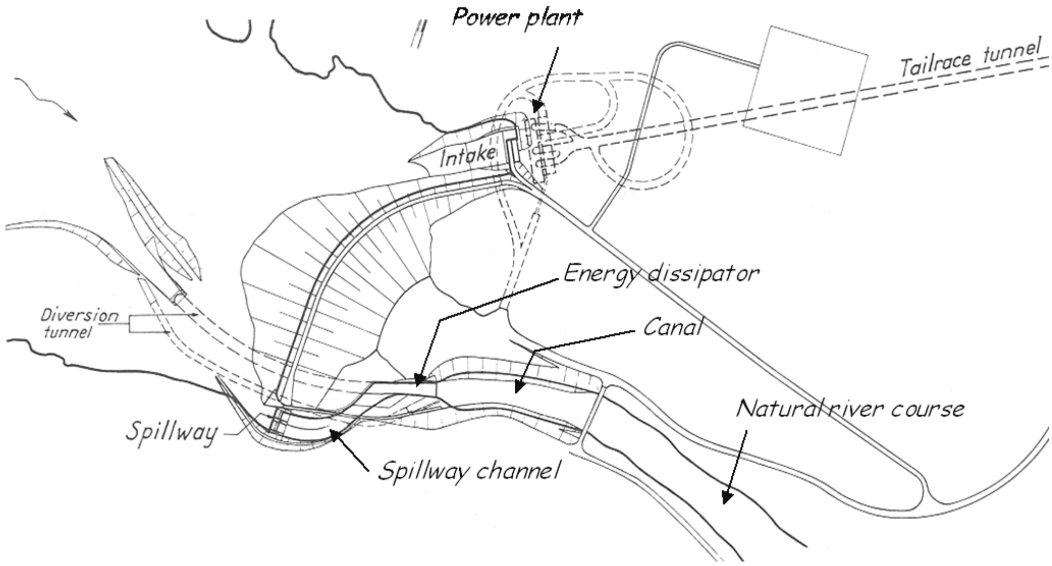


Figure 9.1 Layout of Letsi before rebuilding

The spillway, with concrete channel and energy dissipator, is placed on the right side of the rock-fill dam and adjacent to the rock foundation. It consists of two 15 m openings with tainter gates. The total discharge capacity corresponds to ~1500 m^3/s at the FRRL.

The spillway channel is bent to the left in plan and is about 120 m in length. The channel bottom is partially in rock and partially in concrete, Figure 9.2. From the channel, the spillway water is discharged into an energy dissipator that is some 40 m lower in elevation. The dissipator runs almost perpendicular with the dam axis. It is ~100 m long and 20 m wide. The water is then conveyed to the natural river through an artificial canal paved with stone.

Figure 9.2 Spillway channel before rebuilding

9.1 Safety evaluation – hydraulic aspects

Based on the flood criteria, the discharge capacity of the spillway in Letsi has to be increased by ~25% from the existing level 1500 m^3/s.

Dam-safety evaluations were made for Letsi in 1996. The purpose was to achieve an increased safety level by identifying and solving safety-related problems of the dam that were normally not covered in a traditional inspection. The investigation included several parts, i.e. review of documents and records from archives; experiences from maintenance and operation; organization, preparedness and training; overall dam inspection; and safety evaluation and recommendations.

There had been some doubts regarding the actual discharge capacity and for how long time the spillway would endure the design flood. These doubts were confirmed by the evaluation, which included a test discharge. In 1993 and 1995 high floods occurred in the river and the spillway was opened to discharge surplus water. In 1993, the flood release lasted about a month, with a maximal discharge of 700–800 m^3/s in four days. In 1995, the spillway was open for three weeks, with a maximal discharge of 700–800 m^3/s in two days. The spillway, its channel and energy dissipator worked satisfactory during the floods. Some damages were found in the energy dissipator and erosion protection downstream.

Test releases were previously made at different occasions. The latest one was carried out summer 1996 in connection with the safety evaluation of the dam. The maximum spillway discharge reached 1200 m^3/s and was limited to a very short period due to the incipient erosion in the river downstream. The problems that were observed at this time included:

- The left spillway side-wall was too low and tended to be overtopped and there was not much safety marginal for the right wall.
- Aerated water from the spillway channel fell outside the energy dissipator and on the rock-fill of the downstream dam slope, resulting in erosion.
- When the discharge increased to 1200 m^3/s, relatively strong waves started to occur in the canal downstream, which meant that the energy dissipator worked less effective with increasing flood discharge.
- Erosion occurred in the erosion protection of both riverbanks downstream.

The function of the energy dissipator at spillway floods larger than 1200 m^3/s was not verified. Without effective energy dissipation, there would be extensive erosion in the canal. Refurbishment was therefore imperative as far as the spillway channel, energy dissipator and the canal downstream was concerned.

9.2 Refurbishment proposals

In 1998, preliminary studies were made as to how the waterway downstream of the spillway would be refurbished to safely discharge the design flood. Four rebuilding alternatives were initially outlined, two of which, designated as Layout A and B, were in 2001 chosen by the dam owner for further investigation. Geo-technical studies were made for both layouts, which formed a partial basis for the design.

9.2.1 Layout A – new waterway

Layout A, illustrated in figure 9.3, was a new, straight spillway channel, drawn to the right of the existing one. The difference in elevation between the spillway and the bottom of the energy dissipator was about 60 m. The layout is summarized as follows:

- To build a new sidewall on the left side to close the existing channel.
- To excavate the channel upstream of the stilling basin in rock with a width of 30 m.
- To construct a new energy dissipator partially in rock and partially in concrete, without any baffle blocks.
- To provide the canal downstream of the dissipator with erosion protection to withstand the high flow velocity. Its bottom width is 40 m.
- To build a new bridge over the canal, with new road connection.
- To abandon the existing waterway and refill it with excavated materials, and to remove the bridge across it.

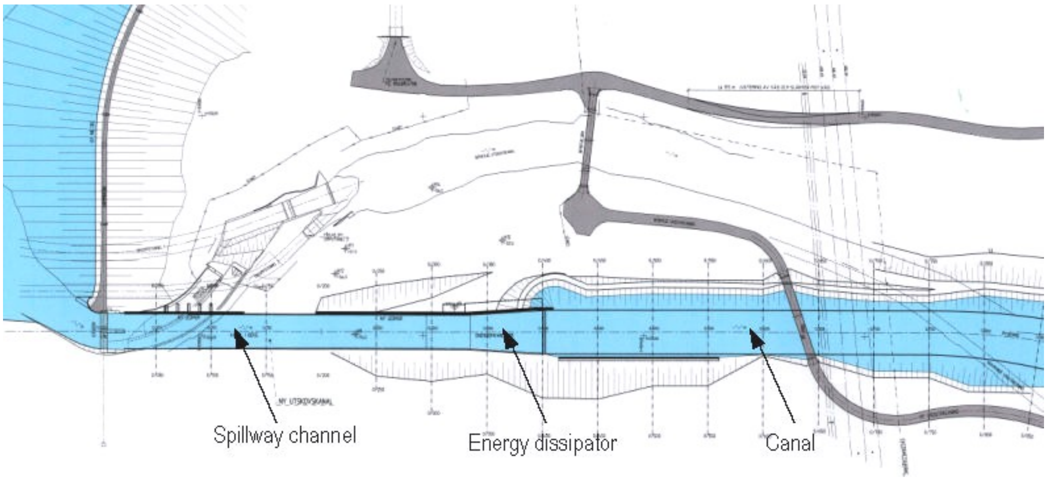


Figure 9.3 Layout A – new spillway channel with energy dissipator

The advantages are that the existing waterway can be used as spillway during the construction period. Besides, the space between the existing and new waterway can be used for temporary storage of excavated materials from Layout A. Three longitudinal bottom profiles are originally proposed for the channel between the spillway and the energy dissipator, Figure 9.4. The distance between the spillway and the energy dissipator is about 350 m.

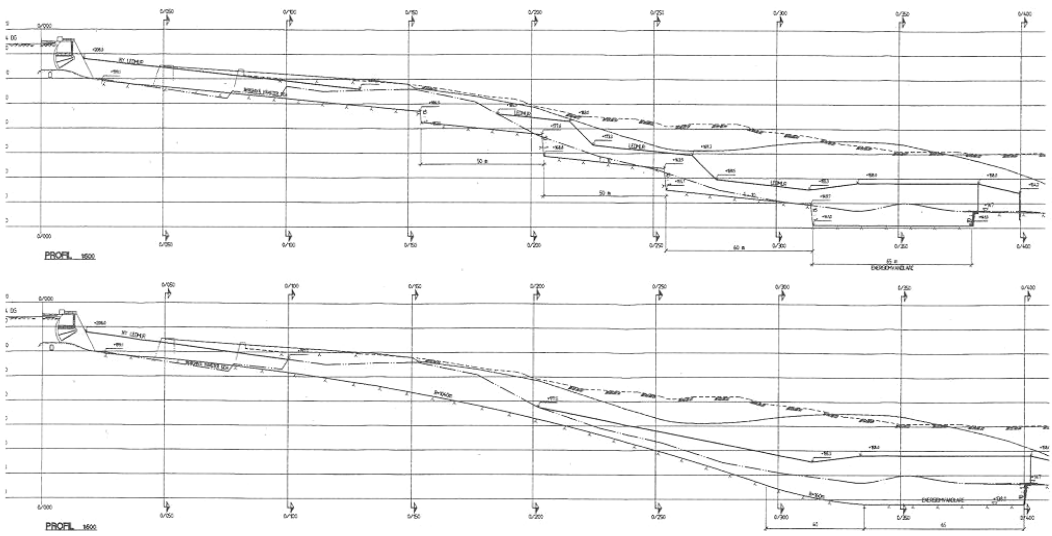


Figure 9.4 New waterway – Layout A1 (above) and A3 (below)

Layout A1 has a stepped shape with several steps at an interval of about 50 m. The first step is about 150 m from the spillway. The difference in bottom elevation at each step is about 4–8 m. The idea is that the steps help to dissipate energy and to aerate the high-velocity flow.

Layout A2 a straight line is drawn from the spillway to the dissipator, giving the spillway channel a constant bottom slope of approximately 17%.

Layout A3 the major part of the channel is given a convex form, followed by a short concave transition to the energy dissipator. Compared with Layout A2, the channel becomes less costly as less excavation is required.

9.2.2 Layout B – modification of existing waterway

Layout B, shown in Figure 9.5, refers to rebuilding of the existing spillway waterway, which consists of the following aspects.

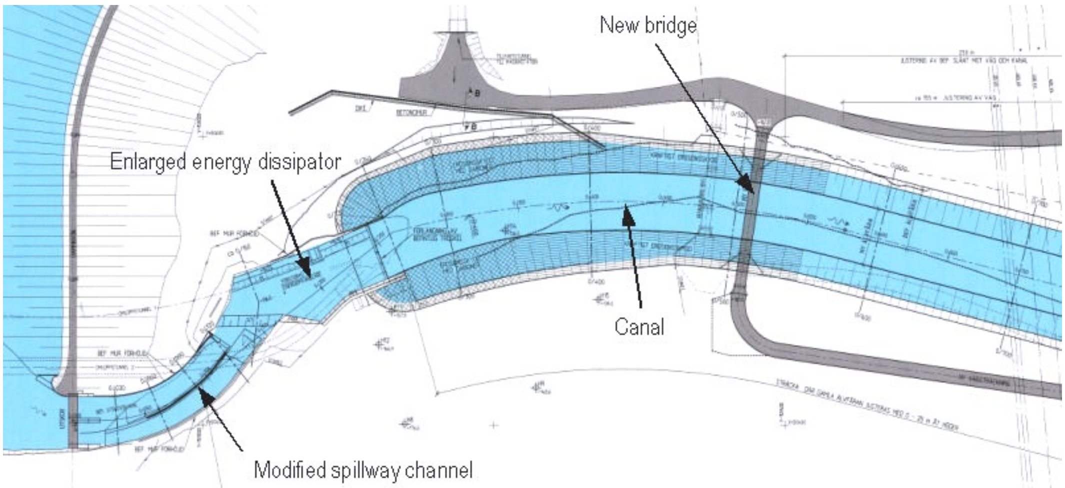


Figure 9.5 Layout B – rebuilding of the existing waterway

- The existing spillway channel, bending to the left in plan, can be widened and deepened (Layout B1).
- The energy dissipator is enlarged in plan and its bottom is given a lower elevation.
- Repair of concrete side-walls in the existing dissipator that are damaged.
- Modifications of the existing canal downstream the dissipator by relocating it up to 25 m to the right and providing with a more gentle side slope.
- Reinforced erosion protection especially on the left side of the canal up- and downstream the existing bridge.
- The left sidewall of the energy dissipator is increased to account for the high water stage and wave height at the design flood.

There are two proposals of the longitudinal profile of the channel upstream the energy dissipator, Figure 9.6. Layout B1 is excavated in the existing channel bottom. The channel is made wider and has several steps, however at much shorter interval and with smaller difference in bottom elevation than in Layout A1.

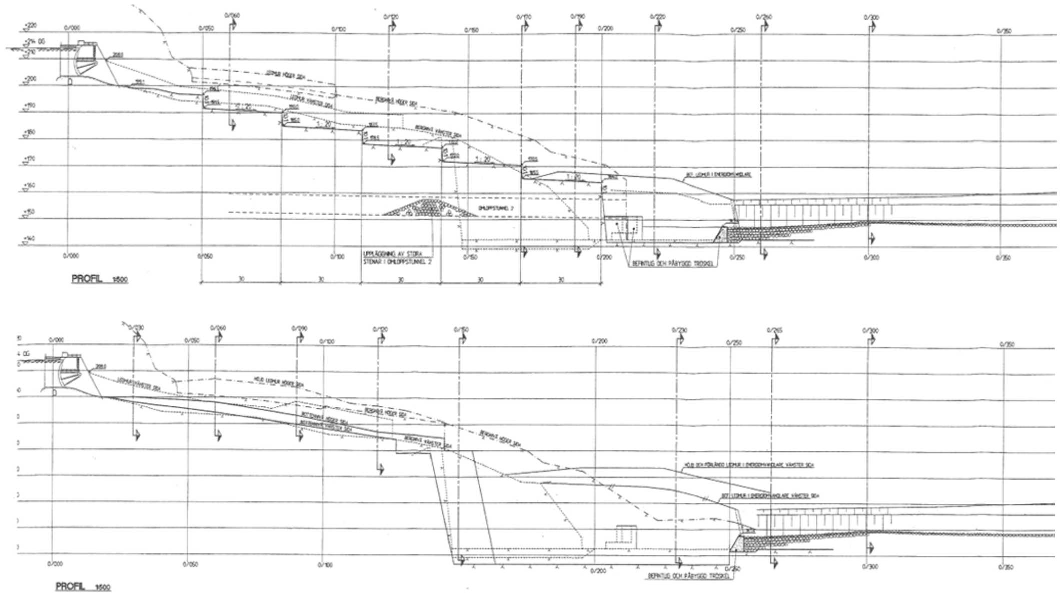


Figure 9.6 Modified existing channel – Layout B1 (above) and B2 (below)

Layout B2 is based on the existing channel. The left half of the channel has an elevation 1.5 – 4.0 m lower than the right half in order to offset the flow. The bottom width and elevation of the channel upstream the energy dissipator are kept the same. The channel side-walls are however increased to accommodate the design flood. Due to the enlargement of the dissipator, the spillway channel becomes somewhat shorter, and an almost vertical slope is formed down to the energy dissipator.

9.3 Hydraulic model studies

Due to the complexity of the problem in terms of flood magnitude and channel geometry, hydraulic model studies are necessary. The purpose is to examine the function of the two layouts for spillway floods up to a level somewhat higher than the new design flood, so that potential damage downstream can be avoided in extreme flood situations.

Model tests in a scale of 1:50 were made for both layouts. Figure 9.7 shows the m0del for layout A. The model downstream the spillway was built in 3-mm sheet metal to speed up construction and facilitate possible modifications in geometry. Based on the Froude's number the waterway (final design) was roughened with macadam to reproduce the prototype roughness (M = 30–35). The model was constructed in such a way that, without replacing the side-walls, a longitudinal bottom profile could be easily removed and a new one was placed. For each layout, the following hydraulic aspects were identified:

- Overall behavior of flow, water stage and flow velocity in the channel between the spillway and energy dissipator and requisite height of channel sidewalls.

- Function and efficiency of energy dissipator; effects of plunging water jet from the spillway channel (Layout B) and strong aeration on requisite sidewall height; and proposals for geometry modification if needed.
- Water stage and flow velocity in the canal downstream the dissipator and requisite canal height and design of erosion protection.

Figure 9.7 Letsi model test - flow behavior at design flood (Layout A3 with radial bottom profile)

9.3.1 Layout A – test findings

The main test results for Layout A are summarized as follows.

Layout A1 (stepped spillway channel)

The measured (prototype) flow velocity in the spillway channel ranges from 16–23 m/s from up- to downstream. The flow enters the energy dissipator with a velocity of ~24 m/s, almost independent of the flow rate. The pressure of the air trapped at the steps fluctuates about zero atmospheric pressure, indicating that the flow at the steps is well aerated.

The steps, with a bottom elevation difference of 4–8 m, aerate effectively the flow and dissipate energy. Below the flow rate 10-12 m^3/s per unit width, the flow behaves in a acceptable way in the channel. With increasing flow rate, however, the steps become improper in relation to the flow depth and velocity, extensive water cascade and spray is generated all the way from the first step to the energy dissipator. At the design flood, the water cascade is as high as 20-30 m over the side walls. The stepped spillway channel is therefore not a practical solution for Letsi and is abandoned without further tests.

Layout A2 (constant bottom slope)

It is found that to achieve a constant bottom slope between the spillway and the energy dissipator is too costly due to extensive excavation. Model tests are therefore not made for this layout and are instead focused on Layout A3, which has similar flow behaviors.

Layout A3 (radial bottom profile)

The hydraulic condition at the design flood is shown in Figure 9.7. The measured flow velocity changes from 16 – 27 m/s in the spillway channel, which is obviously higher than that in Layout A1. The channel works well from low flow rate to the design flood discharge, without any hydraulic complications. The whole channel is free of water cascade, except at the front of the hydraulic jump in the dissipator, where higher side walls are needed locally.

Due to the inadequate length of the original design on one hand and higher flow velocity on the other, the energy dissipator is extended downstream by some 20 m. After the modification, the hydraulic jump and energy dissipation behave satisfactorily. The behavior of flow in the canal downstream the energy dissipator is similar to that in Layout A1.

9.3.2 Layout B – test findings

For Layout B, the hydraulic difficulties are associated with that fact that it bends to the left, while the water tends to flow straight. The model is shown in Figure 9.8.

Figure 9.8 Model setup for existing spillway channel

Layout B1 (stepped spillway channel)

As in Layout A1, the steps aerate the flow and contribute to energy dissipation. However, due to the bent channel, the cross-sectional distribution of the flow is strongly uneven at high discharges, which implies that the right side-wall should be very high in order to prevent overtopping (Figure 9.9).

Figure 9.9 Existing spillway channel with stepped bottom profile

Due to the uneven flow distribution from the channel, the high-velocity water plunges into the energy dissipator in a concentrated manner. This gives rise to low efficiency of energy dissipation and extensive, unacceptable water cascade over the left sidewall of the dissipator. The main outflow into the canal is directed obliquely towards the left canal bank. As a result, strong and pulsating waves are generated in the first 150 m of the canal. The stepped channel leads to uncontrolled flow situations at high discharges and is abandoned.

If slopes are provided between two adjacent steps, the overall flow behavior in the waterway unfortunately deteriorates – the flow distribution becomes more uneven and the left half of the channel is almost drained. Energy dissipation becomes less effective, with stronger waves generated in the canal, implying higher risk for erosion in the riverbanks.

Layout B2 (modification of existing channel)

As described above, the left half of the existing channel has a lower elevation than the right one. Thanks to this elevation difference, part of the water is prevented from flowing from left to right and relatively even cross-sectional distribution of flow is obtained up to a flood discharge of 1000-1100 m^3/s. This situation is also verified in previous spillway test releases.

At higher discharges, however, the elevation difference becomes too little in relation to the flow depth and inadequate to reflect sufficient amount of water and achieve an even flow distribution. As a result, unacceptably much water flows in the right half of the channel and high wall height is thus required on the right side to prevent overtopping. The resulting flow condition in the energy dissipator and canal downstream is hardly acceptable and reminds one of the situations with Layout B1.

To improve the hydraulic condition, a partition wall is suggested in the middle of the channel to counteract the flow. Due to inertia, the water runs up high against the left side of the wall. It is therefore unpractical yet uneconomical to build a wall high enough to prevent overtopping. Moderate overtopping can be allowed without causing unacceptable problem. Different heights of the partition wall are tested. Tests also show that the wall needs to extend over the whole channel length.

If the wall height is less than 3 m, it is not effective to prevent the flow from left to right, still too much water runs on the right side of the channel at high flow discharges. With a wall higher than 5 m, reasonable flow distribution is achieved in the channel. However, the water cascade becomes extensive in the right half due to that fact that the water that overtops the partition wall plunges down from a higher position.

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On a trial-and-error basis, the final profile of the partition wall is given a varying height. It changes from 2.5 m at the upstream end to 5 m in the middle and gradually reduces to 3 m at the downstream end. By doing so, reasonable flow condition is produced in the channel and water cascade and spray over the right channel wall occurs in an acceptable manner at the spillway discharge up to the design flood, Figure 9.10. As it is almost economically impossible to design a partition wall to achieve even flow distribution in the channel and at the same time to get rid of the water cascade and sprinkle, the final choice of the partition wall profile is a compromise.

Figure 9.10 Modified existing spillway channel with partition wall, flow behavior at the design flood

With this partition wall, the plunging spillway water into the energy dissipator is evenly distributed alongside and behaves in a satisfactory way. The downstream end of the left sidewall is modified in order to direct the flow away from the dam body and reduce the water cascade onto it. With the enlarged dissipator in both depth and plan, enough water volume is provided as buffer for effective energy dissipation. The left sidewall of the dissipator is increased to account for the high water level in it and to partially prevent the water cascade from it.

As described above, with Layout B1 (stepped spillway channel), strong oscillating waves are produced in the canal downstream the energy dissipator. The situation is tremendously improved to a satisfactory level with Layout B2, to which the partition wall is added. The partition wall plays a central roll in the improvement of the overall hydraulic behavior in the modified waterway.

9.4 Evaluation of layouts

Of all the proposals, the radial bottom profile (A3) of the new channel and the modification of the existing channel with partition wall (B2) are two economical and hydraulically workable solutions of interest. Their advantages and disadvantages are summarized as follows.

For Layout A3 the advantages are

- hydraulically more proper with a straight channel.
- can be built with the possibility of full spillway release during larger part of the construction time.
- shorter time with regard to the requirement of lowering upstream water level.
- safer solution at high spillway discharges up to the design flood and without complication.

Its disadvantages are

- high flow velocity upstream the energy dissipator with certain risk for erosion in the rock.
- higher refurbishment costs.
- larger uncertainty in the construction costs due to the uncertainty in the elevation and quality of the rock foundation.

For Layout B2 the advantages are

- lower refurbishment costs.
- less impacts on the environment.
- somewhat lower flow velocity in the spillway channel, which has proved to be resistant and stable to the flow from the past years of spillway operation.

Its disadvantages are

- risk for inconvenience during construction in case of spillway release.
- risk for costs for temporary measures in case of spillway release during construction.
- water cascade and spray from the energy dissipator close to the dam at high floods.
- difficulty with rock excavation in connection with the enlargement of the energy dissipator.
- longer time with regard to the requirement of lowering upstream water level, which leads to extra loss in power production.

Cost estimations are made for all layouts. Layout A3 is ~25% more expensive than layout B2. The hydraulic model tests have resulted in major changes in the design, which include the considerable enlargement of the energy dissipator and increase in the side-wall height of the spillway channel and affect the final cost estimations.

The final solution chosen for upgrading Letsi was Layout B2, i.e. the existing spillway channel was modified. The engineering measures adopted in Letsi are shown in Figure 9.11.

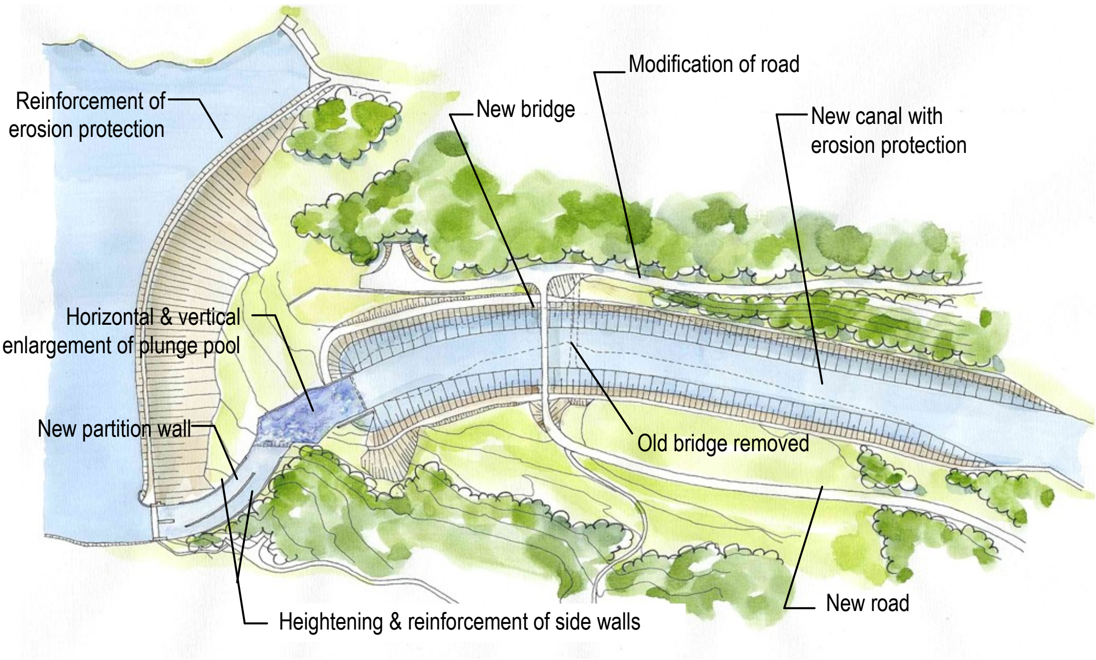


Figure 9.11 Engineering measures implemented for Letsi

Publications concerning Letsi are as follows.

Yang, J (2002), Rehabilitation of Letsi dam- hydraulic studies of new spillway discharge channel (*Ombyggnad av Letsi – hydrauliska studier av ny utskovskanal*), Report No. U 02:02.

Yang, J (2002), Letsi dam, refurbishment - hydraulic investigations of modified existing spillway channel (*Letsi kraftstation, ombyggnad – Hydraulisk utredning av befintlig utskovskanal*), Report No. U02:36.

Yang, J (2002), Rebuilding of Letsi Dam, alternative B – optimization of spillway channel (*Letsi ombyggnad , alt. B – optimering av vattenväg*), Report No. U 02:90.

Bond, H, Yang, J, Cederström, M & Halvarsson, A (2003), "Letsi Dam – Refurbishment for Safe Passage of Extreme Floods, Hydraulic Considerations", WaterPower XIII, July 2003, Buffalo, NY.

Yang, J, Halvarsson, A, Bond, H & Cederström, M (2006), "Modification of flood discharge structure for updated design flood at Letsi dam", Dam Safety 2006, Sept. 2006, Boston, MA.

10 Ligga

Ligga was commissioned in 1954. The power station was originally equipped with two units of 85 MW each. A third, larger one was put into operation 1982. All the turbines are of the Kaplan type with vertical axis, operating at a head of ~40 m and a total flow rate of ~1050 m^3/s. The total installed capacity corresponds to 345 MW.

The embankment dam is constructed with an impervious core of moraine and is founded on rock, Figure 10.1. It has a crest length of 350 m and a crest width of 10 m. The maximum height of the dam is 35 m above the rock foundation. The volume of active reservoir storage is 6 Mm^3 at the FRRL.

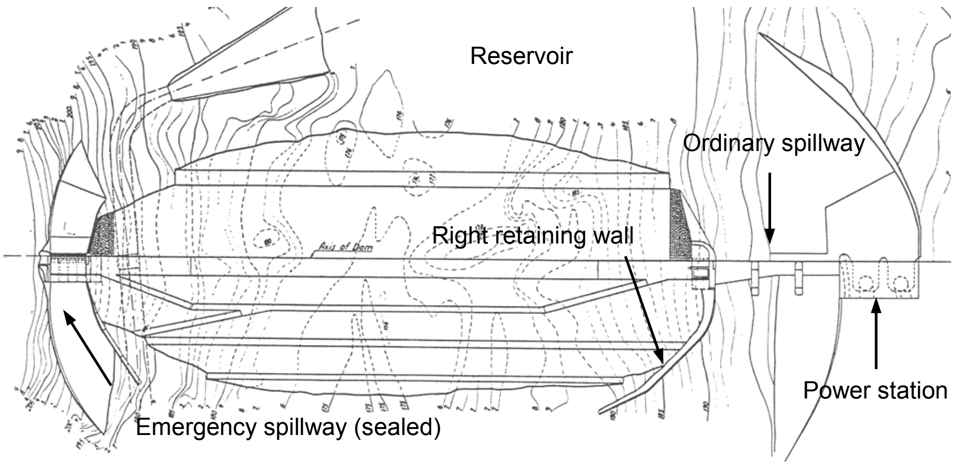


Figure 10.1 Ligga dam, layout

The spillway is situated on the left side of the dam and to the right of the power station. It has three 20-m openings with upwards moving tainter gates operated by remote-controlled electric winches. Vertical steel stop-logs are used for back-up closing. All gates and guide slots are electrically heated. The maximum discharge capacity corresponds to ~2200 m^3/s at the FRRL. There exists also a 20-m emergency spillway close to the right abutment intended to discharge 800 m^3/s. This opening is, at the moment, not in use and sealed with a concrete wall.

The spillway channel of the dam is formed by rock excavation in its middle and is bounded by natural bedrock of coarse-grained granite, Figure 10.2. Through the channel, there exist even diabase dikes with a width of 20 – 60 cm. The past years of spillway operations have seen severe erosion and damages in the fractured rock. Rock instability becomes visible even at moderate spillway discharges 800–900 m^3/s. Furthermore, there is risk for erosion at the toe of the embankment dam due to high downstream water stage in combination with wave motions during spillway release.

Figure 10.2 Spillway channel is bounded by bedrock of coarse-grained granite

10.1 Planned engineering measures

A safety evaluation of the dam was completed in 2005, followed by a preliminary study to determine the extent of the upgrade required to meet the revised design criteria. The upgrade, initiated in 2006, involves re-shaping of the spillway channel; replacing the reservoir rip-rap for wave protection; construction of a supporting berm downstream of the dam; new instrumentation with a leakage gauging system along the dam toe and optic fiber device for temperature measurements; renovation of concrete structures and finally installation of back-up power and control systems for the spillway gate hoists.

The dam is classified as a high-consequence dam according to RIDAS (1B). According to this, the toe of the dam must have sufficient capacity to withstand a theoretical design leakage without progressive erosion initiated. The dam toe is constructed from rather coarse material, which is assumed to be sufficiently erosion resistant. Studies of the filter material from the dam construction period indicate, however, that parts of the dam might have insufficient drainage capacity. The main body of the downstream supporting material consists of fine-grained blast fill from the excavation of the underground power station. As the supporting fill lacks also horizontal drainage filter, this could in turn cause building up of high pore pressure in the fill in case of exceptional leakage.

Theoretically, high pore pressure may cause a seeping outflow in the slope to appear as high as 12 m from the ground, where the fill is more fine-grained. Calculations show that it can't be excluded that continuous erosion of the dam toe is initiated above the reinforced dam toe in case of a leakage. To deal with this, a supporting rock berm of coarse material will be placed on the slope. As the current slope consists of rather coarse material, the berm can be placed directly on the slope, without any filter or other preparation. The added berm aims to both increase slope stability during extreme conditions and to prevent erosion caused by the design leakage.

The upstream riprap on the dam does not meet the revised design criteria. The wave action is not a problem whereas the reservoir is small. Situated close to the Arctic Circle, the ice load working on the riprap is however extensive. The current rock riprap, repaired some ten years ago, has again suffered notable damage just below the full retention level. This is where the ice pries loose individual rocks in the riprap due to daily reservoir drawdown in the winter. A new riprap of interlocking rocks, with a min. diameter of 0.6 m and a max. of 1.2 m, will be constructed. This is the min. requirement to avoid serious short-term damage in the upstream dam slope.

The dam lacks active dam safety instrumentation. As it is a high hazard dam, an extensive program for instrumentation will be carried out. The min. requirement involves continuous recording of leakage, measuring points for displacement reading and standpipes to record hydraulic pressure in the dam downstream filter and bedrock. To meet this, a 400 m long and up to 1.5 m high concrete guide wall will be constructed along the dam toe to collect leakage water to an automated weir. A number of standpipes will be installed. Measuring points will be placed along the up- and downstream side of the dam crest.

The right retaining wall, between the spillway and the dam, has suffered damage in its rock foundation. Historically, it was observed that, during spillway discharges, a circulating zone appeared along the toe of the dam. The dam toe at the end of the retaining wall was before reinforced with boulders. Those are, however, estimated to be stable for the load of no more than 600–700 m^3/s discharge. In addition to reshaping the spillway, it is decided to reinforce the lower part of the retaining wall at the same time. The dam toe downstream of the wall will be protected from erosion by the construction of the rock berm mentioned above.

An inspection of the bedrock indicated that heavy erosion occurred during operation of the spillway. No records could however be found from any full-scale discharge test, showing the impact on the spillway channel from somewhat extreme floods. Determining if the spillway channel has sufficient discharge capacity to safely pass the design flood, it was earlier decided to perform a discharge test.

A review of historical flood data indicated, however, that the largest damage occurred during the 1993 August high flood on Lule älv. At that time, the spillway discharge peaked at no more than 900 m^3/s during one hour, with an average discharge of 430 m^3/s during a 24-hour period. This proved to be enough to cause some 800 m^3 rock mass to fall out from the spillway channel, at the dam toe immediately downstream of the right retaining wall. The flood was released through the middle and right spillways openings. The left opening was not used. This proved afterwards to be unfortunate as the bedrock in the spillway channel is of notably less strength on the right side. The damaged area was immediately reinforced with a one-meter thick layer of concrete anchored in the bedrock. The purpose is to prevent any further erosion that could undermine the foundation of the spillway structure.

The consequence classification requires that it should be feasible for the dam to safely pass the 100-year flood without obvious damage occurring. In addition to this, it must be possible to pass the design flood without seriously endangering the overall dam safety. Since the 100-year flood is ~1100 m^3/s and severe damage has been noted at a discharge of below 900 m^3/s, the discharge capacity test is canceled. The reason is that a prototype test is likely to be unwillingly aborted before even the discharge reaches the 100-year flood. It would not be possible to determine the impact of the 2000 m^3/s design flood. Therefore, it was decided to instead perform hydraulic model tests to determine the required reshaping of the spillway channel. The model tests are also used to determine the actual spillway capacity.

10.2 Re-shaping the spillway channel

Studies have elucidated the need of re-shaping the spillway channel, so that the design flood, around 2000 m^3/s, can be discharged without jeopardizing the structure integrity of the dam. Hydraulic model tests are still the best way in this context. That is the reason why a physical model is built to examine possibilities of channel modification, Figure 10.3.

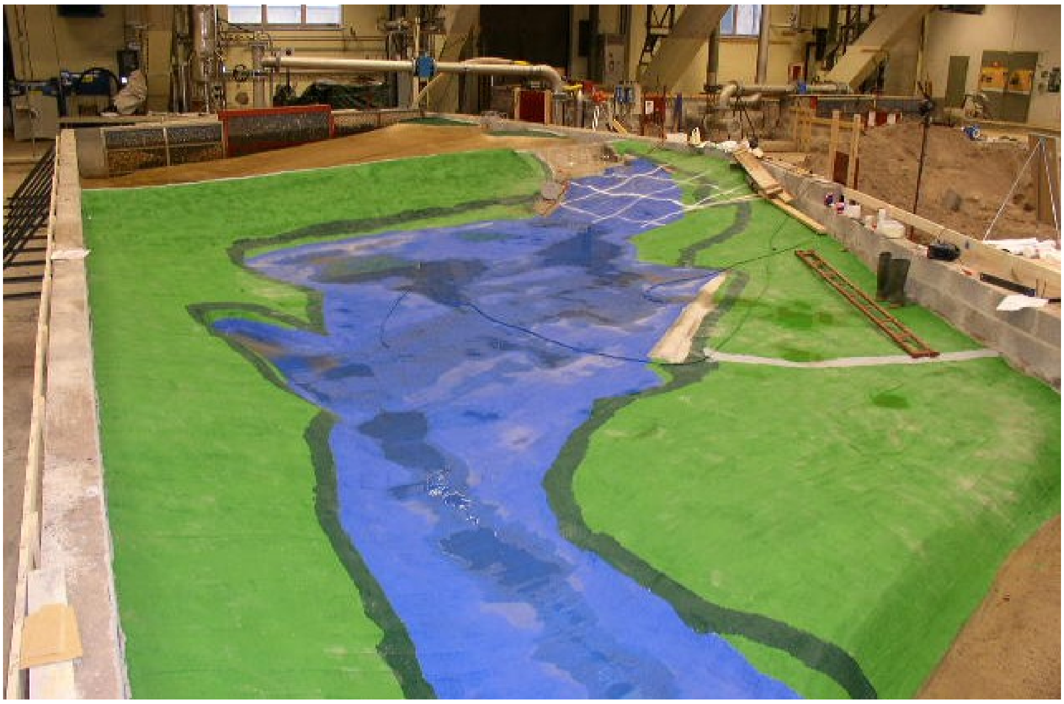


Figure 10.3 Model of Ligga dam with spillway channel

Sidewalls exist on both sides of the channel. Further downstream on the left side of the spillway channel, there is a levee of rock-fill, the purpose of which is to guide the spillway water and prevent it from flooding the forest behind. The modification of the channel is governed by the following principles.

- The spillway water is reasonably distributed in the channel, so that it can resist the impact of the flow up to the design flood; erosion and damage in the rock due to high velocity and pressure fluctuation are avoided. This is the principal goal of the model study when the channel is re-shaped.
- The risk for erosion in the dam toe should be minimized. In the existing situation of spillway discharge, there exists a somewhat strong circulating flow zone downstream the right side wall, with subsequent risk for erosion.
- Overtopping of the rock-fill levee and flow into the forest downstream must be avoided. Flow velocity and wave motions at the levee should be kept to an acceptable level.
- The right part of the channel is characterized by poor rock quality and should not be touched. This implies that rock excavation can only be done on the left side and the flow is in the best possible way directed to this side.
- Concreting and use of concrete sidewalls should be as far as possible minimized as large costs are involved.

It is desirable that the channel modification should provide between 22000 – 25000 m^3 excavated rock material, which constitutes the second important goal of the study. The excavated material is sized and used for improvement in the erosion protection upstream and for construction of toe berm for increased downstream slope stability. The rebuilding of the dam would be more expensive if the requisite rock material is transported from other places and the environmental impacts would be greater. This requirement affects even how the channel should be excavated – in order that large fragmentation of rock is obtained, the excavated depth is kept as large as possible and excavation of less than one meter in depth is avoided.

Flow patterns in the existing spillway channel are first mapped in order to seek the reason for rock erosion. Several different re-shaping options are examined in the model. The different versions of channel modification do not conclude in an optimal solution with solely favorable flow patterns downstream of the spillway. They all have, in one way or the other, resulted in standing waves across the spillway. Instead of its current shape warped to the right, it would be more favorable to cut a straighter channel downstream. As the ground on the left side of the channel is considerably higher than on the right side, this would, however, be an expensive solution. The spillway channel has, instead, been straightened as far as possible and the channel bed cambered to even the flow over the accessible surface. But the water load on the rock floor will still be rather high.

Based on preliminary test results, two basic ways of excavation, option A and B as shown in Figure 10.4, are considered as potential and chosen for examination in the model. Efforts are made to achieve satisfactory flow conditions after the excavation and at the same time to obtain the desired volume of rock material.

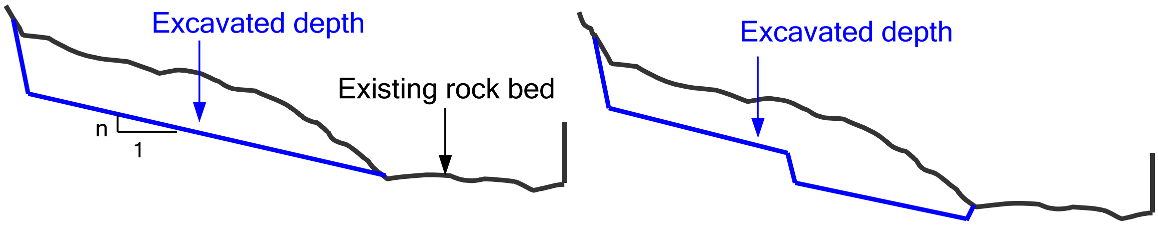


Figure 10.4 Two basic ways of excavation, option A and B

Tests have shown that, with a moderate cross-sectional slope in option A, there is a good chance to procure somewhat reasonable flow patterns. However, the excavated volume amounts to only ~5000 m^3. If the excavated depth at the left bank is increased to a nearly horizontal slope ($n \approx 0$), the channel flow is satisfactory up to ~1500 m^3/s. At higher spillway discharge, too much water runs on the left side of the channel and construction of a concrete sidewall on the left is needed along the downstream section. Otherwise, the water would flow into the forest upstream of the rock-fill levee and give rise to potential floating debris problem for the dam downstream. Due to this, option A becomes less attractive and is abandoned.

By the method of trial and error, satisfactory flow conditions can be achieved if the existing channel is excavated somewhat deeper in the middle part, option B. In this way, a "stepped" profile is given in cross section. The step deflects effectively part of the water and reduces the load imposed on the left bank. With a proper combination of cross-sectional slope and excavation depth, the spillway water is reasonably distributed in the channel. Figure 10.5 shows the channel topography before and after the proposed excavation.

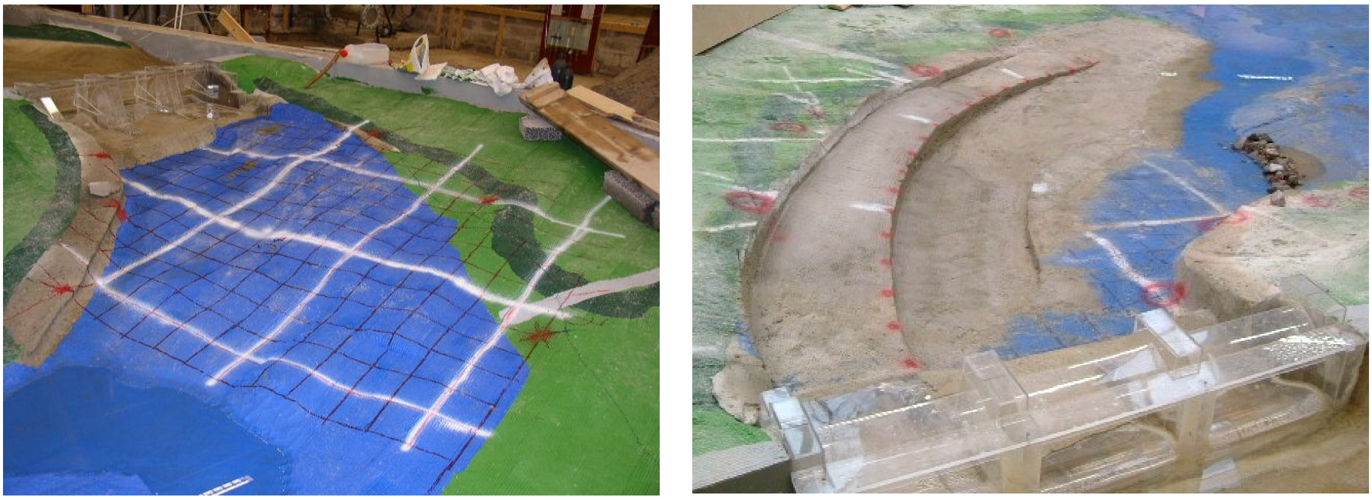


Figure 10.5 Spillway channel – before and after excavation

Up to the design flow ~2000 m^3/s, the main current follows the excavated channel and is directed away from the dam toe, Figure 10.6. The water is calm with little wave motions along almost the dam whole toe. At the downstream end of the right sidewall, there exists however an area with circulating flow at higher spillway discharge than 1500 m^3/s, with risk of undermining the toe. To place erosion protection is suggested to overcome this.

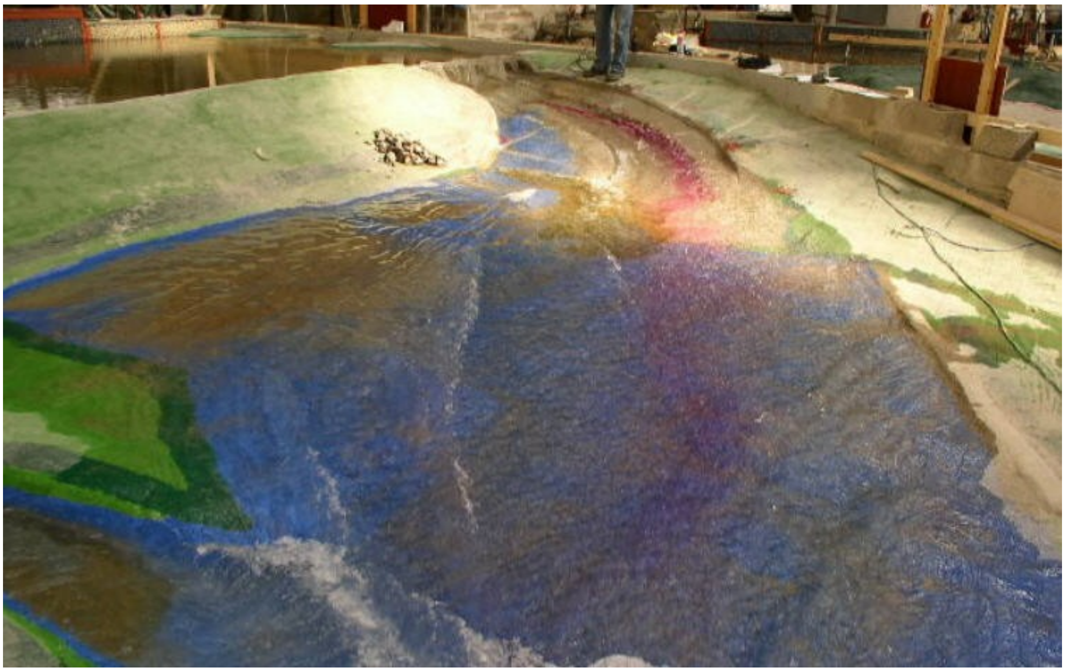


Figure 10.6 Flow conditions in the modified spillway channel (option B)

Along the rock fill levee, the terrain is almost dry if the spillway flow is below 1000 m^3/s. There is a 20 m levee section in the middle with lower terrain than its surroundings. If the discharge is higher than 1400 m^3/s, this levee section is subjected to high flow velocity. At the design flood, the velocity amounts to 4.0 – 5.5 m/s. To avoid erosion and undermining, this section should be reinforced to withstand the high velocity.

The irregularities in the bedrock in the upper part of the channel, caused by erosion damage from historical flood releases, will be evened out by pouring ~1000 m^3 of concrete. This reduces the standing wave and water load to a scale that minimizes the need for rock reinforcement. A setback with this approach is that the water level and wave actions at the levee increase notably. This will be dealt with using blast material of fractions unsuitable for the dam toe and upstream riprap to extend and strengthen the levee. The current levee consists of coarse material that will be used as riprap when the levee is reshaped. Trials are made, where guide walls are used to divert the spillway water towards the center of the channel. This solution is however dropped, as it requires massive and costly concrete constructions to divert the flow to an extent that no measure has to be taken with the levee.

When the re-shaping is finalized, measurements are made of flow velocity, water depth and dynamic water pressure at typical locations of the excavated channel, which serves as the input data of computer simulation of erosion in the fractured rock. The physical modeling itself cannot provide any answer to the rock stability of the channel. It is up to the numerical modeling to verify whether any erosion will occur and any reinforcement is required.

Ligga's model investigations are found in the following publications.

Yang, J, Skärberg, P, Amnell, A, Aurosell, U & Bergsten, M (2006), Ligga dam – hydraulic model studies of rebuilding measures for higher dam-safety level *(Ombyggnad av Ligga kraftstation - modellförsök med dammsäkerhetshöjande åtgärder),* Report No. U 06:01.

Amnell, A & Yang, J (2006), Spillway in Ligga - hydraulic model tests of discharge capacity (*Utskov i Ligga – bestämning av avbördningsförmåga genom modellförsök),* Report No. U 06:35.

Ekström, I, Yang, J, Mörén, L & Cederström, M (2007), "Adapting Ligga to higher design flood, spillway channel modification through physical & numerical modeling", WaterPower XIV, July 2007, Chattanooga, TN.

11 Långbjörn

The Långbjörn, owned by Vattenfall, was commissioned in 1959. The scheme is composed of an embankment dam with an impervious core of compacted moraine, a spillway section and a powerhouse, Figure 11.1. The left embankment dam, including the connecting dam, has a crest length of 600 m, with a maximum height of 33 m. The right one is 80 m in crest length and its maximum height above the rock bed is 24 m.

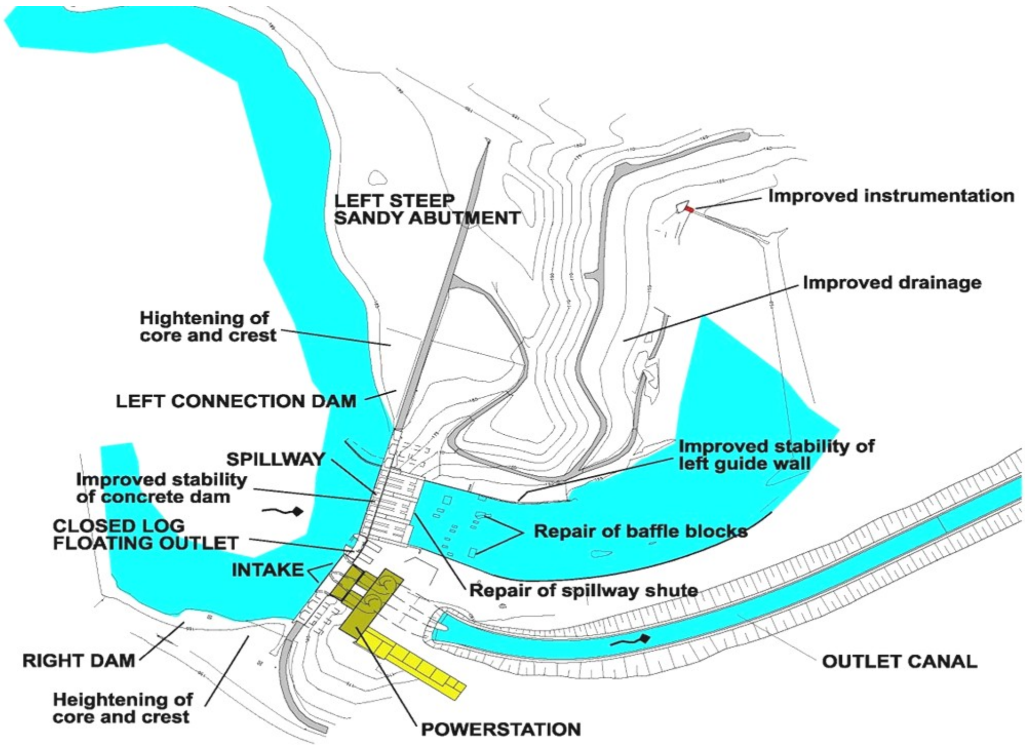


Figure 11.1 Långbjörn dam, layout

The power station is equipped with two generating units with an installed capacity of 40 MW each, operating at a gross head of 33 m and the total turbine discharge is 275 m^3/s. The water from the powerhouse discharges into an artificial canal excavated in rock. The length of the canal is ~750 m.

The spillway section is located in the middle of the river and has three 15-m overflow openings with upward going radial gates. At the FRRL, the spillway discharge capacity amounts to 1550 m^3/s. As the rock was of fairly good quality, it was not considered necessary to construct any artificial stilling basin, Figure 11.2. In the early 1960's, considerable erosion and damages occurred in the rock bed during spillway operation. Through hydraulic model tests, a total of 13 baffle blocks were built downstream of the spillway chute to lift the supercritical spillway water and reduce the impact of the rock farther downstream (Corlin & Larsen 1979). The blocks are partly reinforced and anchored into on the bedrock.

Figure 11.2 Spillway channel and energy dissipator in Långbjörn

Långbjörn is classified as a high hazard dam, its updated design flood is ~30% higher than the existing spillway capacity. As imposed by the higher safety standard, there is a need to rebuild the dam, so that the design flood can be safely released without causing failure of the dam.

The left embankment dam leans on the riverbank revetment**,** Figure 11.1. A cross-section through the dam is given in Figure 11.3, showing the composite of the revetment consisting of silt, fine sand and gravel. The material is easily erodable. The revetment is adjacent to the left sidewall of the stilling basin and has been a frequent source of concern throughout the operating time of the spillway. Furthermore, the revetment is characterized by very steep slope and is not protected against erosion from flowing water. The erosion, especially when the spillway discharge is large and the downstream water level is high, might jeopardize the safety of the dam. If so is the case, proper engineering measures must be taken to prevent it.

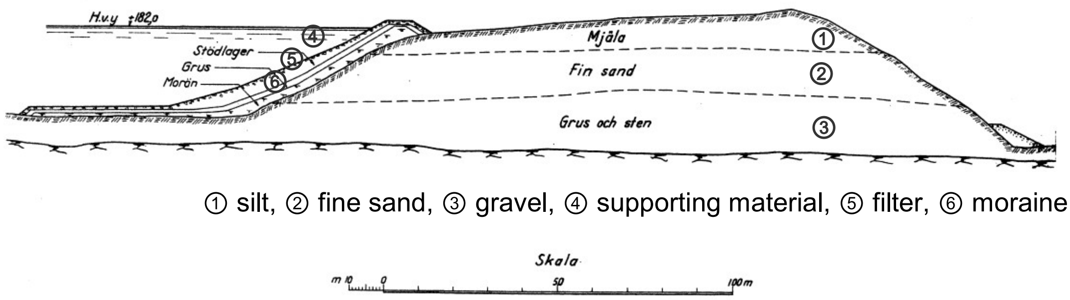


Figure 11.3 Left embankment leaning on a riverbank revetment

In the studies made for Långbjörn, it is found to be essential to investigate the risk of erosion downstream of the dam, both along the river revetment and in the riverbed. The reason is twofold - it has a significant pedagogic value and also contributes to an increased understanding of the erosion process in the downstream area. The most cost-effective way to study the erosion development and evaluate potential rebuilding measures is still physical model testing. With this background, a hydraulic model was built, Figure 11.4. In the first place, a model bed was made in concrete.

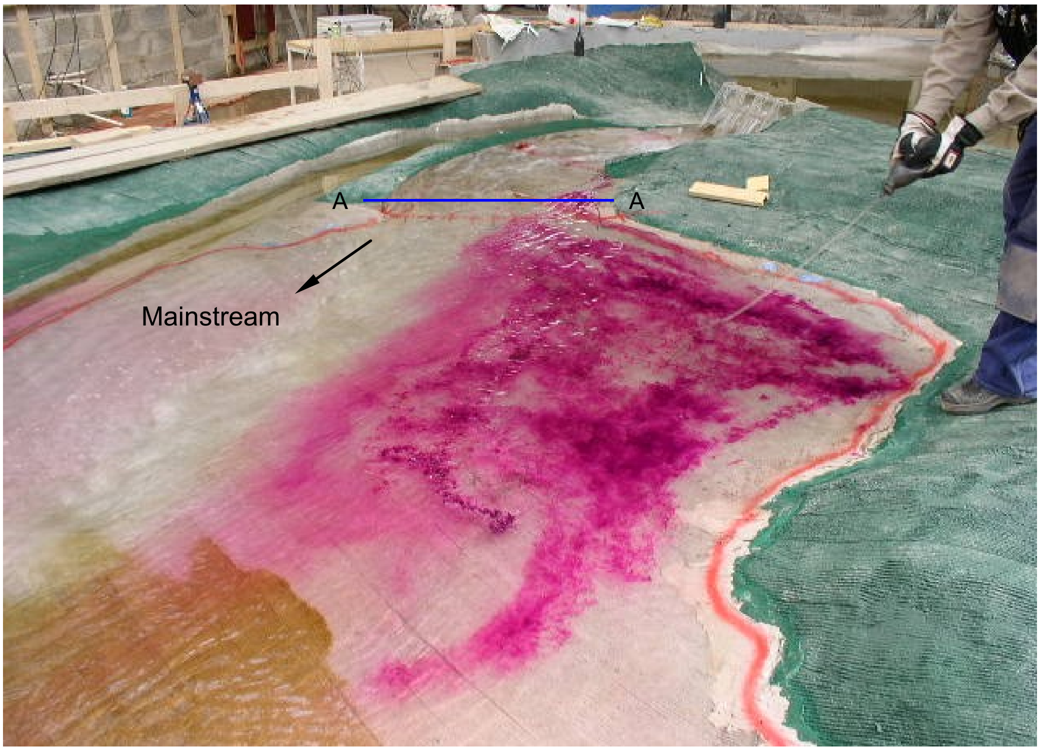


Figure 11.4 Fixed-bed model for Långbjörn dam

A river section of 300 m upstream and 1200 m downstream of the dam was included in the model. Limited this length, a model scale of 1:100 was chosen. Besides conventional tests including spillway capacity and energy dissipation, the studies concentrated on the erosion of the river revetment at high spillway discharges and potential measures. The primary concern was the formation of a large circulating zone downstream of the revetment. If such a flow pattern existed, the material of the revetment would be transported away.

Tests were first made in a fixed bed model. To account for the effect of river erosion on the flow pattern, movable bed tests were also conducted. In the whole downstream area, the model was made erodable. Three types of natural sand, d_{50} = 0.6, 4.0 and 9.0 mm and density ρ = 2250 – 2600 kg/m^3, were used to represent the loose layer of material above the hard rock elevation. In the main stream, the 9.0 mm sand was used; downstream of the revetment, the 0.6 mm sand was placed. Figure 11.5 shows also the placement of the sand materials and the stabilized bed form after erosion.

Figure 11.5 Impact of river erosion on flow pattern downstream, movable bed conditions

It was observed in the tests that the erosion occurred mainly in the main stream, roughly running along the tailrace canal. Immediately downstream of the stilling basin, it was eroded to the rock elevation. Irrespective of whether the model bed was fixed or movable, the flow downstream of the river revetment was almost stationary and wave motions were insignificant. In other words, even the erosion went down to the hard rock elevation, the flow pattern was largely the same as without the erosion; no circulating flow zone was formed, which was contrary to the assumption made before the tests.

The measured velocity at the section where the right sidewall ends (section A-A in Figure 11.4) is given in Figure 11.6. Combinations of different flow situations in the downstream river valley affected hardly the flow pattern.

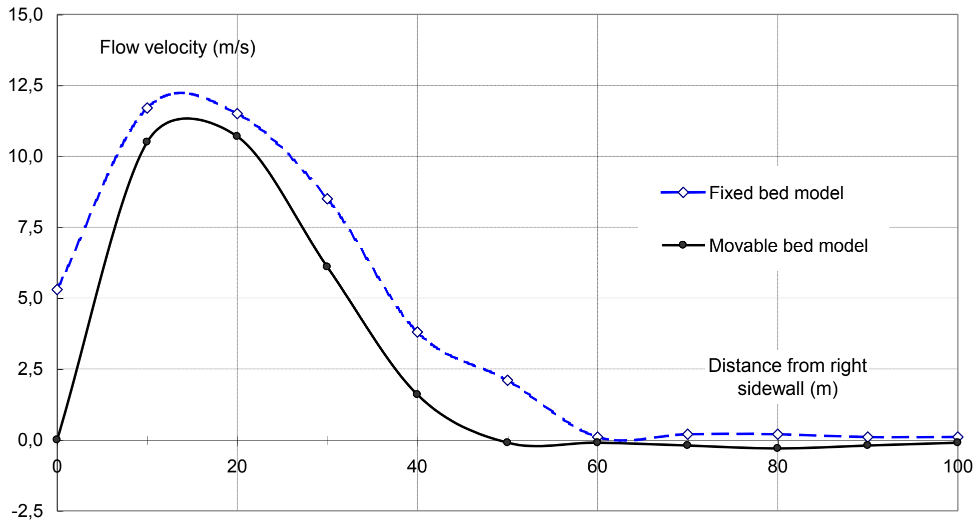


Figure 11.6 Flow velocity at the downstream end of the spillway channel

The left sidewall of the stilling basin was extended in the beginning of the design fearing for the circulating zone downstream of the revetment. The rock foundation is of poor quality, deep excavation is therefore needed to ground it on solid rock. This means that large costs are involved. Thanks to the model tests, this unnecessary measure is avoided, leading to a saving of nearly one million US$.

Publications from Långbjörn model tests are given below.

Yang, J, Amnell, G & Skärberg, P (2006), Långbjörn dam-safety measures, hydraulic model tests *(Långbjörn – dammsäkerhetshöjande åtgärder, hydrauliska modellförsök),* Report No. U 06:58.

Amnell, G, Yang, J & Bodén S (2006), Långbjörn spillway – determination of spillway capacity by model tests *(Långbjörn Utskov – bestämning av avbördningsförmåga genom modellförsök),* Report No. U 06:80.

Yang, J (2007), Långbjörn – water splash from stilling basin and suggested measures (*Långbjörn – skvalp i energiomvandlare och föreslagna åtgärder*), PM nr. 07-122.

Yang, J, Eriksson, H, Gustafsson, A, Stenmark, M & Mikaelsson, J (2007), “Långbjörn dam - adaptation for safe discharge of extreme floods”, Canadian Dam Association Annual Conference, Sept. 2007, St. John’s, NF.

12 Midskog

Midskog was constructed in the beginning of 1940's during a time of energy shortage and an urgent need for more electricity supply. In 1944, the first two units were commissioned; in 1956, the power plant was extended with a third unit. The total turbine discharge is 640 m^3/s and the installed capacity is 145 MW.

The spillway consists of four gated overflow spillway openings and one bottom outlet, with a total discharge capacity of ~2400 m^3/s at the FRRL, +251 m. The flow from the former runs almost perpendicular to that from the latter, Figure 12.1.

Figure 12.1 Midskog dam, aerial view during flood release

Midskog consists of both concrete and rock-fill dams, with the following parts from the left.

- Left rockfill dam, length 490 m and max. height 27 m. The dam has a central core of a concrete wall with a crest level of +252.5 m.
- Concrete dam, length 110 m and max. height 27 m, containing the bottom outlet.

- Gated spillways with a width of 73 m and a max. height of 12 m.
- Concrete dam, length 61.5 m and max. height 15 m.
- Intakes to the turbines, total length 90 m and max. height 17.5 m.
- Concrete dam, length 135 m and max. height 13 m
- Right rock-fill dam, length 80 m and max. height 11 m. The dam has a central core of a concrete wall, with a crest level of +252.5 m.
- Three minor earth-fill dams, with a length of 280, 25 and 175 m and a max. height of 6.5 m.

12.1 Rock erosion downstream of spillway

The revised design flood for Midskog was determined to be ~3100 m^3/s. Different options to increase the spillway capacity were before investigated in a preliminary study. The best option, both economical and technical, was to allow a temporary higher reservoir level than the FRRL in high flood situations. Due to the passive storage, the design flood can be reduced to ~3050 m^3/s at the reservoir water level +252.5 m, i.e. about 1.5 m above the FRRL.

Engineering measures to prepare the dam for a water level of +252.5 m was as follows.

- The rockfill dams were equipped with an L-formed concrete parapet connected to the central concrete core wall. The parapet was given a crest level of +254 m, serving partially as freeboard.
- A concrete wall was added to the concrete dams to the elevation +254 m. The spillway part was not be affected, only the bridge over the piers was lifted by 1.3 m to an elevation that would facilitate floating debris to pass the spillway.

The main concern of the refurbishment of Midskog was the energy dissipation downstream of the flood discharge structures. The concrete crest of the spillway ended immediately after the radial gates, followed by fissured rock. The spillway discharged directly down a steep rock surface where rock erosion had taken place during the past 60 years of operation. The energy dissipation took place direct on the rock surface; no proper energy dissipator was constructed. The rock erosion was so severe that in was necessary to introduce restriction in the operation of the spillway.

A high water level prevailed also downstream, with waves and strong back eddies, which could constitute a threat to the downstream slope of the embankment dams. Thus, to modify the waterway was necessary in order to achieve satisfactory energy dissipation at discharges up to the design flood.

12.2 Shaping the plunge pool

A physical model for Midskog was built in scale 1:50, Figure 12.2, and was designed to investigate hydraulic such issues as

- Discharge capacity
- Water levels
- Flow velocity (both magnitude and direction)
- Wave heights
- Water surface profiles through the overflow spillway
- Pressure conditions downstream of the spillway
- Determination of rock erosion mitigating measures

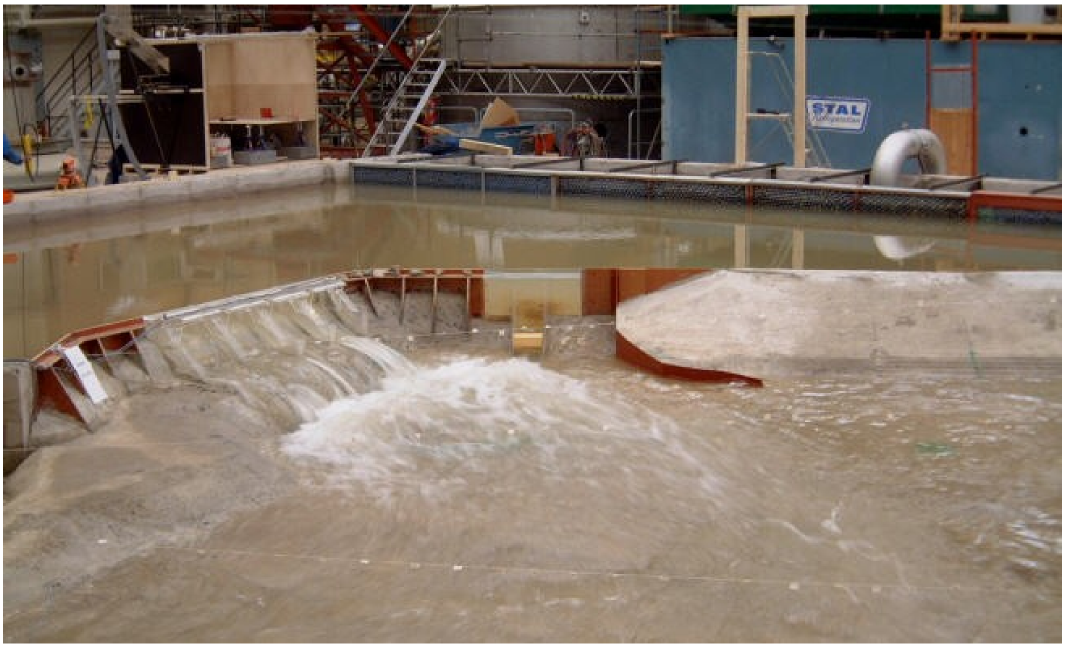


Figure 12.2 Model of Midskog dam, 1:50

The main objective was to reduce the risk of erosion by introducing a plunge pool at the place where the flowing water eroded the rock. The model tests also provided input data of water depth and pressure to the numerical modeling of rock erosion.

Different designs were tested in order to reduce water velocities and impact on the rock. The final design involved addition of ski jumps in the two openings to the left and a plunge pool in which the water jets landed.

The location and sizing of the pool were evaluated with the aid of water pressure fluctuations measured in the model with different layouts, as well as in the subsequent numerical analyses of rock erosion. The min. pool size was sought, which would provide a fair discharge pattern as well as acceptable wave motions, particularly towards the dam toe. The pressure measurements were used to place the pool bottom on a level that would not put more load on the rock surface at the full spillway discharge (400 m^3/s from each opening) than the restrictive flow of about 125 m^3/s from each spillway opening. The restriction was due to the fact that one spillway was in a area with poorer rock quality.

The final pool shaping is illustrated in Figure 12.3, in which one can also see the locations of the pressure transducers used in the pool area.

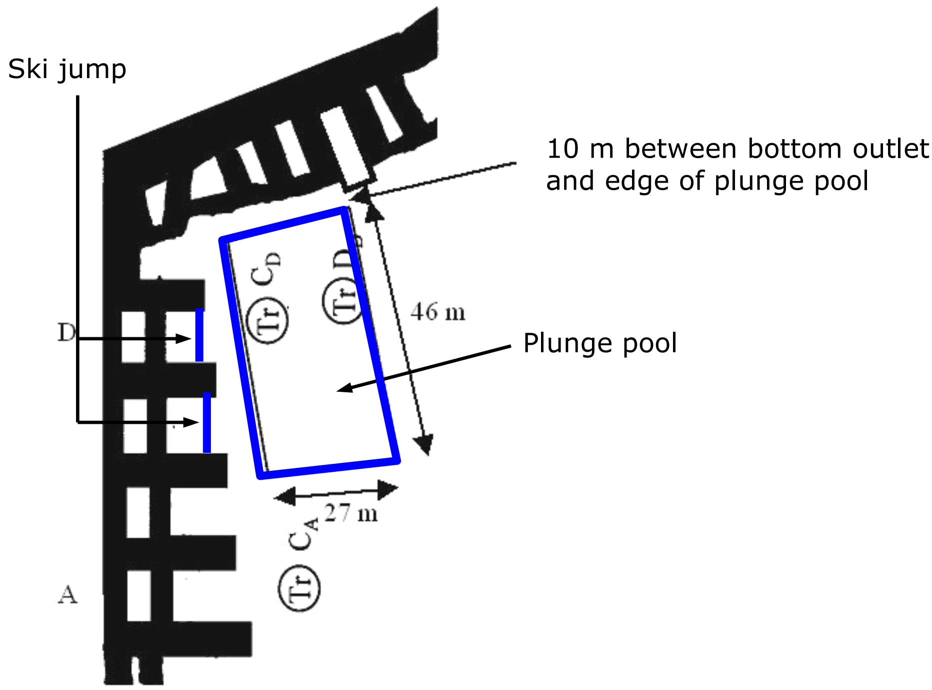


Figure 12.3 Layout of recommended plunge pool

Pressure amplitudes were measured at different combinations of spillway discharges. Typical pressure amplitudes in the pool were in the range of 50 – 340 mm in the model, corresponding to 2.5 – 17.0 m in the prototype.

Pressure fluctuations in the pool were measured at various pool floor elevations. An optimum floor level was established at which any further lowering of the floor only resulted in minor changes in the pressure amplitudes. Below the level +223.0 m, little impact in terms of pressure fluctuations was experienced. Shear loading from the flowing water was estimated from the velocity measurements in the model. The tests of pool floor elevations were carried out to such an extent that the final floor did not further decrease the rock surface load.

The pool geometry and measurements were used as input to the numerical model, with geotechnical assessments of the rock mass, so as to determine the pool function with regard to rock erosion.

The ski jumps and plunge pool after completion is shown in the figure below.

Figure 12.4 Plunge pool after completion

Model studies carried out for Midskog can be found in the following documents.

Billstein, M. (2002), Hydraulic model tests of Midskog dam 2002 - adaptation to revised design flood (*Modellförsök Midskogs kraftstation 2002 - anpassning till nya dimensionerande flöden*), Report No. U 02:52.

Billstein, M, Carlsson, A, Söder, P-E & Lorig, L (2003), Midskog gets physical and numerical. International Water Power & Dam Construction, Volume 55, No. 12, December 2003.

13 Porsi

Porsi, commissioned in 1961-62, is situated on the Lule älv river, Figure 13.1. The above-ground power station and spillway are connected on both sides to embankment dams, founded on bedrock with an impervious core of fine-grained moraine and supporting fill of gravel and blasted rock. Each main dam has a length of about 200 m and a maximum height of 40 m.

The spillway consists of three openings with tainter gates, each with a width of 15 m and a height of 10.4 m. The total discharge capacity is about 2700 m^3/s at the retention water level +78.0 m. Downstream the spillway, there exists a stilling basin excavated in the bedrock. The depth is 6 m and the length 70 m. The basin bottom and the sides are concrete-lined.

13.1 Function of energy dissipation

Based on the new flood criteria, the design flood for Porsi is ~3150 m^3/s, which can be easily discharged by allowing somewhat higher reservoir during the flood. The need for refurbishment at Porsi did not concern the dam body itself, but the energy dissipation and downstream erosion during high flow situations. Erosion in the right riverbank and damage of concrete erosion protection close to the stilling basin already occurred during moderate floods, Figure 13.2.

According to available documentation, the stilling basin at Porsi was originally designed through hydraulic model tests made in 1961 to provide acceptable energy dissipation up to a maximum spillway discharge of about 2000 m^3/s (with the power station in operation, turbine discharge 600 m^3/s, at a downstream water stage of +49.0 m). The highest spillway flow ever discharged, 1190 m^3/s, was the test release made in 1994 in connection with the dam-safety analysis of the dam.

The power station was originally equipped with two units; a third one was added in 1987, giving a total turbine discharge of 950 m^3/s. In connection with the extension, the river channel was dredged and widened from the stilling basin down to the dam Laxede located downstream. The water stage downstream Porsi was now estimated to be 0.5–1.0 m lower at the design flood (as a rule, all turbines are assumed to be out of operation during the design flood). Compared with the original hydraulic conditions when the stilling basin was designed, the energy dissipating capacity was considered much lower.

If the new design flood exceeded the maximal capacity of the energy dissipator, the front of the hydraulic jump would move downstream. This meant that the river channel would be subjected to flow with an estimated velocity higher than 10 m/s and risk of bank erosion and damage that also could affect the dam.

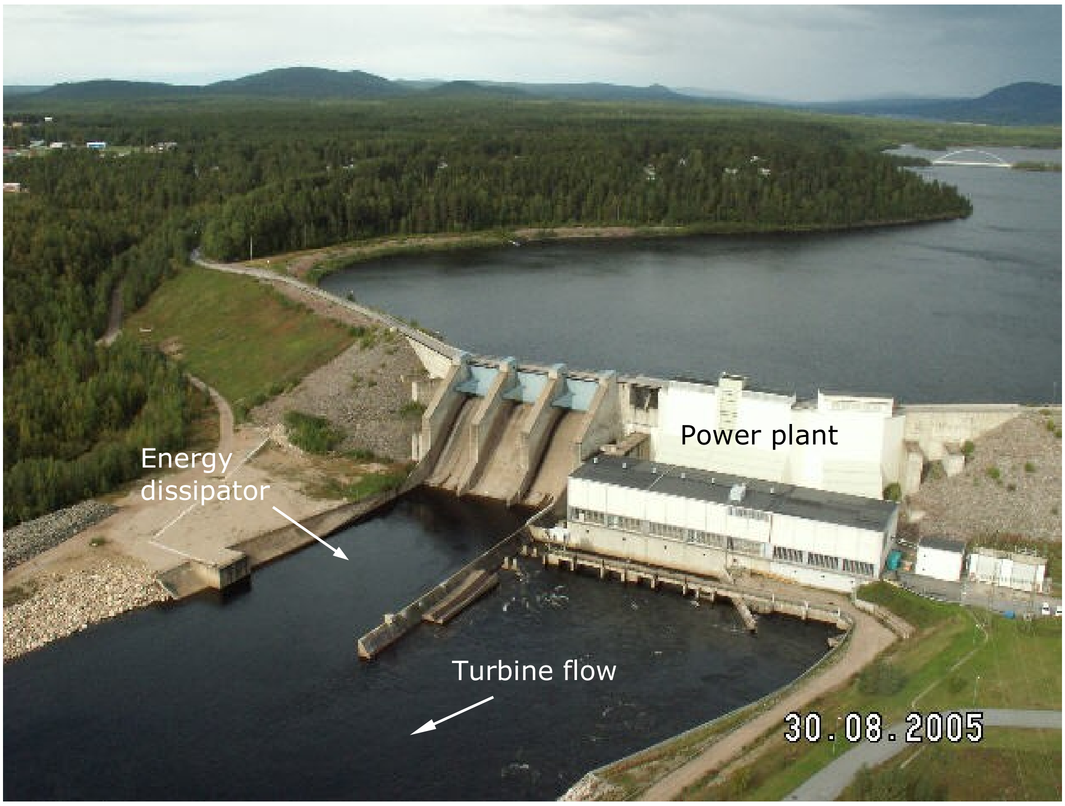


Figure 13.1 Porsi dam, seen from downstream

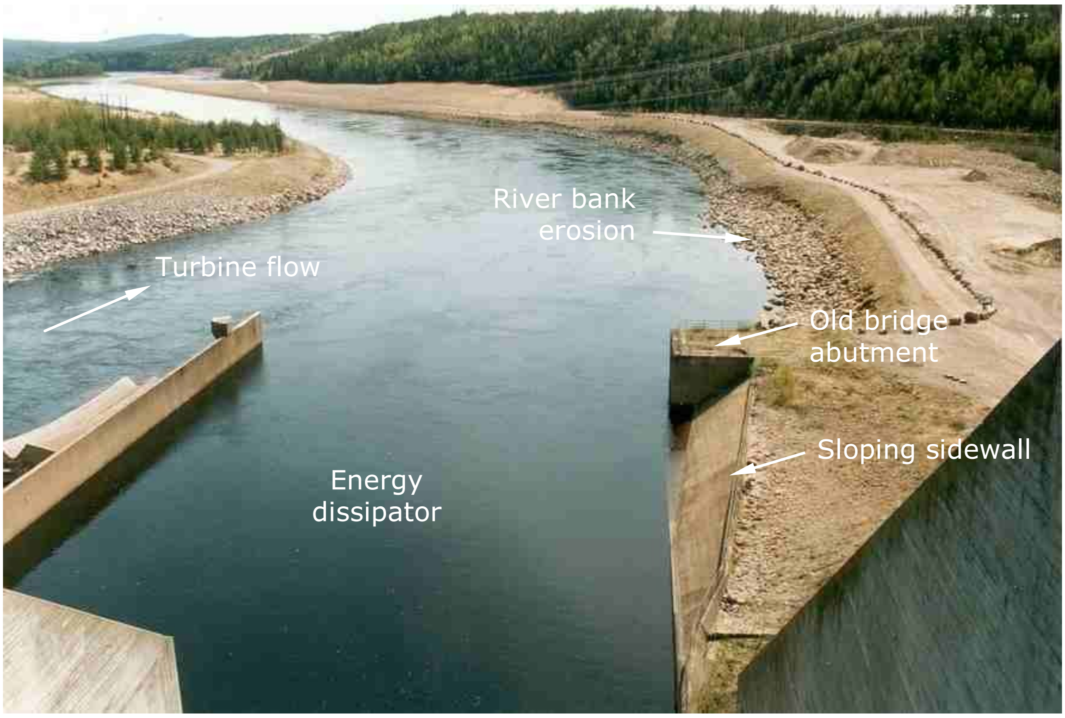


Figure 13.2 Erosion in the right riverbank downstream

13.2 Model set-up

A hydraulic model of Porsi dam, in scale 1:50, was built in 2001, Figure 13.3. The model corresponded to a river length of some 750 m (150 upstream and 600 m downstream) and a width of 300 m, resulting in a model size of 16 m by 7 m. The max. elevation difference was ~48 m (prototype size), giving a model height of ~1 m.

The spillway with gates and crest profile was built in Plexiglas, the stilling basin in plywood and the power station with draft tube in sheet plate. In the beginning of the study, a fixed bed of concrete was adopted downstream. Downstream of the basin, the model bottom was roughened with 8 – 14 mm macadam to the riverbed elevation. The macadam was used to represent the top rock layer with bad quality where erosion took place.

Later, part of the model downstream was made erodable so as to investigate the risk of erosion at extreme floods. When the rebuilding proposals of the stilling basin were evaluated, the model was restored to fixed-bed, with the basin shaped in sheet plate to facilitate modifications.

The purpose of model testing was the determination of the existing capacity of the stilling basin, examination of the downstream flow conditions, evaluation of the risk of erosion and investigations of countermeasures. Deepening or lengthening the stilling basin, adding baffle blocks or their combination were some options.

Figure 13.3 Fixed-bed model of Porsi dam, scale 1:50

13.3 Existing energy dissipation

The tests included in the studies were roughly grouped into two stages. Stage one aimed to elucidate the existing energy dissipation function and consisted of the following tests.

- Test A – determination of spillway discharge with fully open gates. Check if the upstream front of the roadway bridge affected the discharge and specify even the gate opening (lower edge position) where the discharge was not affected.
- Test B – evaluation of energy dissipation conditions. With slowly increased spillway discharge, observe the position of the hydraulic jump and see if the hydraulic jump is displaced out of the stilling basin, partially or totally. Afterwards, tests were made with discharges 3150, 2400, 1800, 1200 and 600 m^3/s. At moderate spillway flows, tests were even made to study the influences of the turbine flow 950 m^3/s on the dissipation function.

  Test documentation included flow pattern in the stilling basin and at downstream cross-sections. A few pressure transducers of 50 Hz were mounted in the basin bottom and sidewalls to record pressure fluctuations during the spillway discharges. Pressure measurements lasted often 30 – 60 min.
- Test C – based on geotechnical survey of the dam, part of the concrete bed in the model was replaced with loose, erodable material. The sloping part of the right sidewall of the basin was also assumed loose and erodable.

  The river bank was thus built with crushed aggregates of (coarse) sand and gravel; for the earthen material behind, suitable fine sand was used, having the following properties, d = 0,1 – 1,0 mm with d_{50} = 0,55 mm, bulk density $\rho_s \approx 1500$ kg/m^3 and compact density $\rho \approx 2500$ kg/m^3.

  The rock bed in the river downstream and other rock surfaces was assumed to be erosion resistant; men modeled generally 1 – 2 m below the surveyed level. The top layer was modeled with gravel.

From the tests conducted in stage one, the following conclusions could be briefed.

At normal tailrace water levels, the energy dissipation did not function satisfactorily at spillway discharges higher than 2400 m^3/s. At the design flood 3150 m^3/s, almost half of the basin was emptied of water, which implied that the existing layout could not tackle the design flood, Figure 13.4(a).

At lower than normal tailrace levels, the energy dissipator functioned, in spite of some degree of deterioration, up to the spillway discharge 1200 m^3/s. At higher discharges, the basin started to lose it function. At 3150 m^3/s, the whole stilling basin was emptied of water and the flow was thrown up at the end of the basin, Figure 13.4(b).

(a) design flood, normal downstream water level

(b) design flood, 1 m lower downstream water level

Figure 13.4 Examination of energy dissipation at different river water levels downstream

The results of measured flow velocity downstream of the stilling basin are given in Figure 13.5 (normal tailrace level) and 13.6 (1 m lower tailrace level). At lower tailrace water levels, the flow velocity increased drastically.

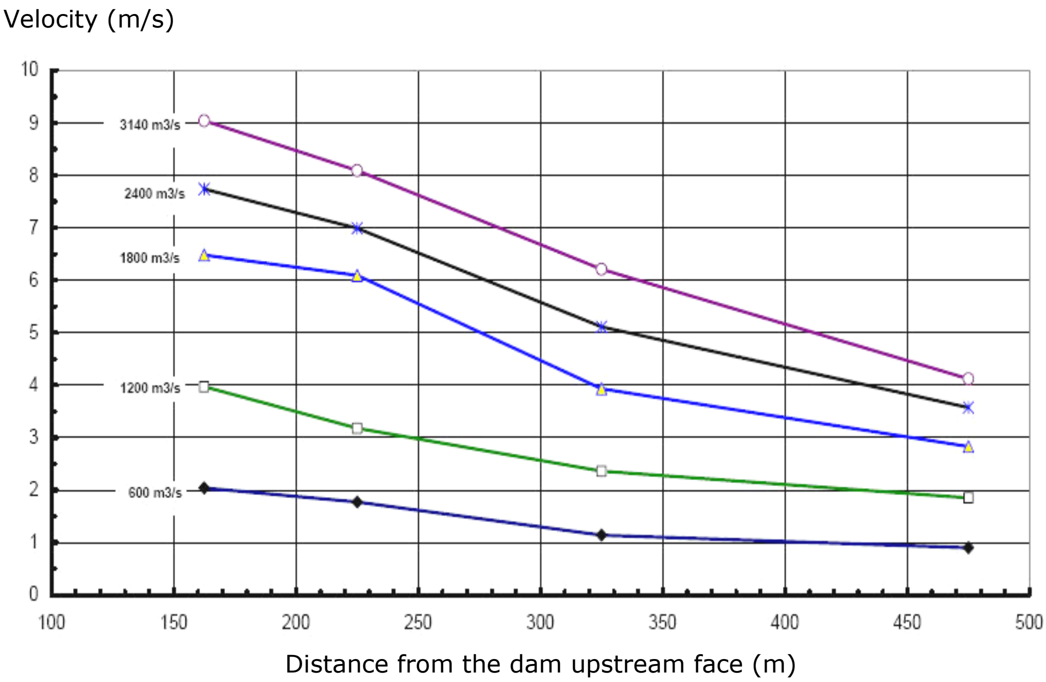


Figure 13.5 Flow velocity downstream of energy dissipator at normal tailrace water level

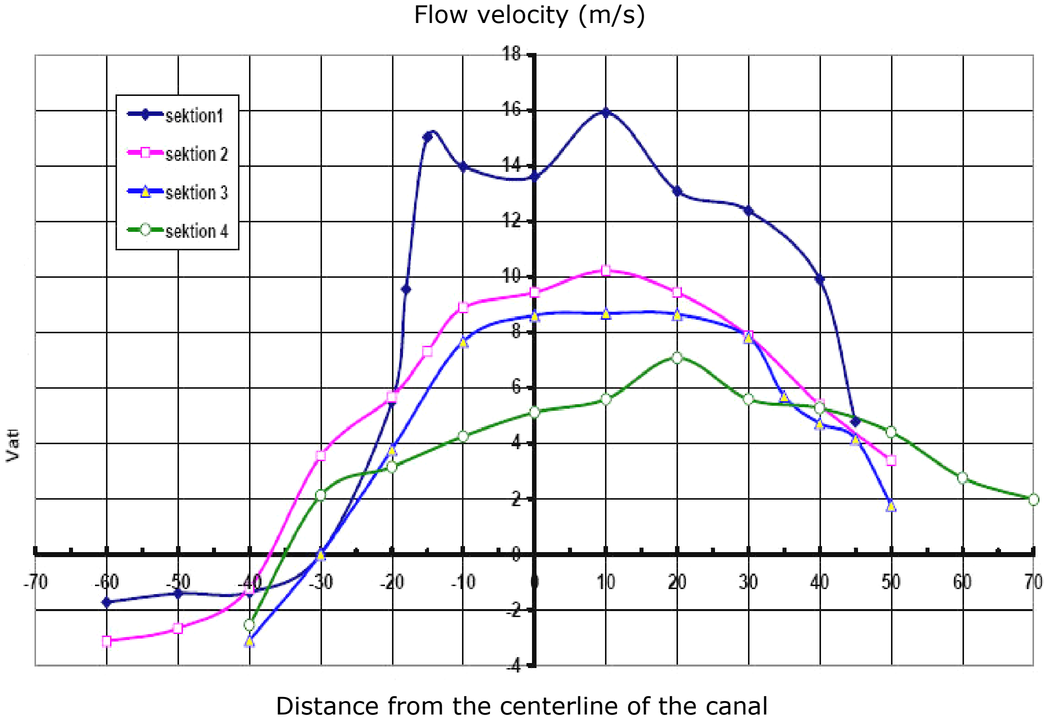


Figure 13.6 Flow velocity downstream of energy dissipator at 1 m lower than normal tailrace water level (section 1, 2, 3 and 4 were located at about 20, 80, 180 and 330 m downstream from the basin end)

The erosion test was run for three days, with a total test time of about 12 hrs and 30 min. The remaining bridge abutment from the dam construction period (Figure 13.2) was kept in place during the test. The inclined sidewall of the energy dissipator upstream of the abutment was, as in the prototype, divided into six parts of equal length. They were loose and not fastened in other structures, but kept on the sand with right slope.

The test started with a discharge of 1800 m^3/s, after about 50 min increased to 2400 m^3/s and after about one hour 10 min to the design flood that was kept the rest of the test, Figure 13.7.

From the erosion tests it could be said that, if the inclined sidewall of the energy dissipator lost its footing and collapsed during extreme floods, a large circulation zone around the abutment would be formed. The size of the zone depended upon how much the sidewall was left. In other words, the inclined concrete wall constituted a protection against erosion in the right bank. The extent to which the erosion would develop was limited by the right vertical side of the energy dissipator and the right spillway pier.

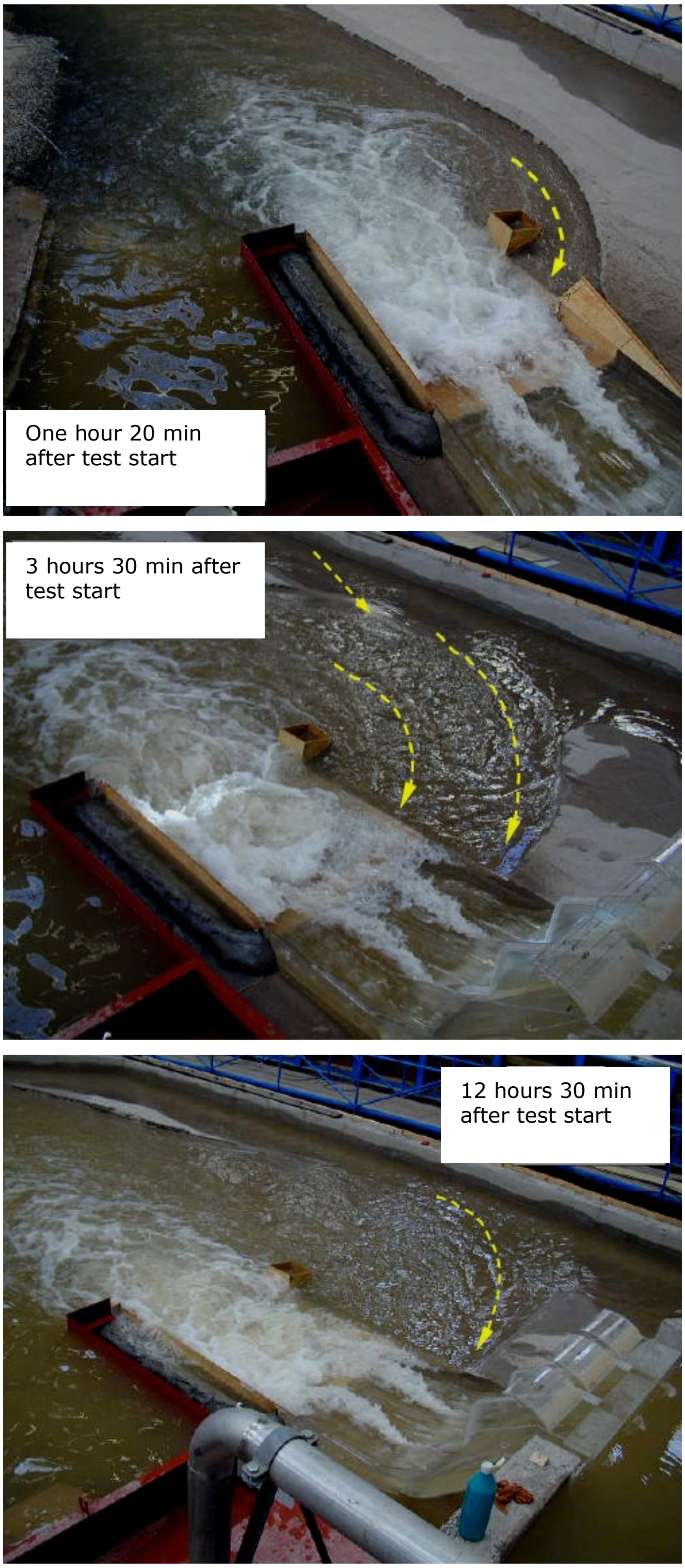


Figure 13.7 Formation of a large circulation zone after erosion in right bank

13.4 Improving energy dissipator

Stage two of the project was mainly composed of restoration of the fixed-bed model after the erosion test and improvement of the energy dissipator. The modified dissipator, shown in Figure 13.8, was approximately 60 m longer and 1.5 m deeper. The completed basin in the model is shown in Figure 13.9.

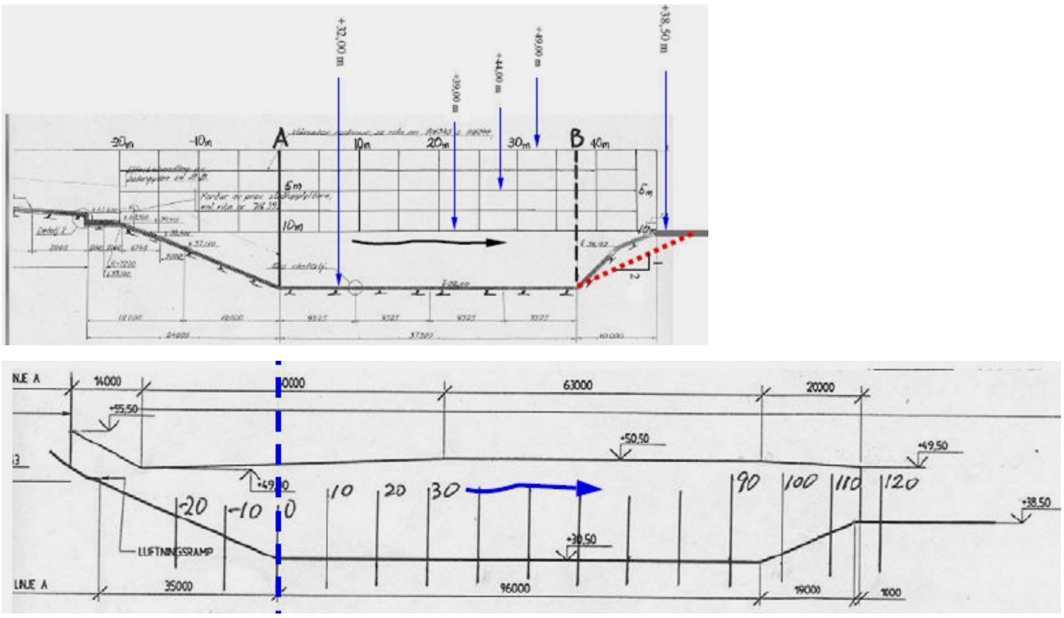


Figure 13.8 **Proposed energy dissipator (drawing below) as compared with the existing (drawing above)**

Figure 13.9 **Proposed energy dissipator in the model**

The following tests were made.

- Test D – modifications of the existing energy dissipator by enlargement and deepening according to a preliminary design. Test and evaluations at discharges of 3150 m^3/s at both normal and 1 m lower tailrace stages.
- Test E – optimization of the new energy dissipator. For the final layout, a number of tests were made and recorded for combinations of flow and downstream water levels.
- Test F – the new dissipator reduced the distance from its end to the left bank, implying narrower water passage for the turbine flow. The tailrace canal was optimized taking into account the power production and risk for erosion. The issue concerned mainly how the left bank could be excavated, so that no notable head energy would occur in normal plant operations.
- Test G – determination of how the river banks downstream could be erosion-protected to handle the design flood at both normal and 1 m lower tailrace levels.
- Test H – test of the energy dissipation function in connection with emergency discharge under the construction period when a cofferdam in form of double sheet piles enclosed the basin at its downstream end.

From the tests it can be stated that the originally proposed layout of the energy dissipator functioned satisfactorily up to the discharge 3600 m^3/s at both normal and lower downstream tailwater levels. There was therefore potential to reduce the basin volume without deterioration of the dissipating function. While maintaining the same basin bottom elevation +30.5 m, the basin was made 11, 22 and 33 m shorter than the original. It could be shown that the energy dissipator could be shortened with about 11 m without affecting its function, Figure 13.10. The resulting flow velocity downstream in the river is illustrated in Figure 13.11.

The tailrace canal was optimized with regard to the energy losses and erosion risk in the left bank. The longer basin reduced the turbine flow passage and caused extra losses for the power production. By acceptable excavation in the left bank, the losses could be reduced.

The energy dissipation and flow pattern downstream in the river were documented for combinations of operation conditions. The conditions of the energy dissipator with a 6 m high cofferdam of sheet piles were also recorded if emergency flood release had to be made during the basin rebuilding.

Figure 13.10 Finalized layout of the dissipator at the design flood and normal downstream water stage

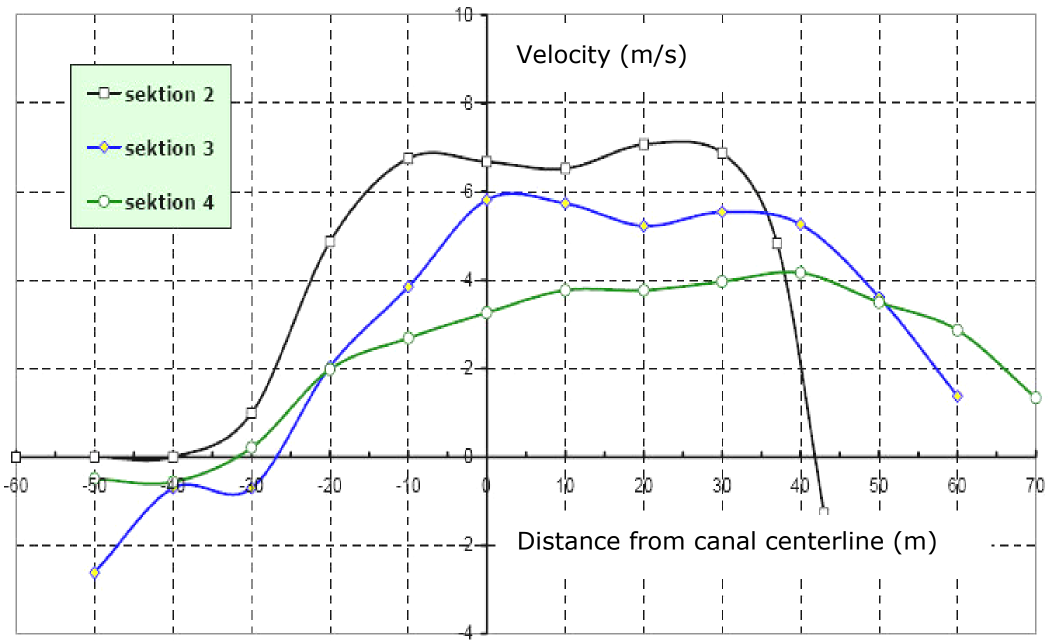


Figure 13.11 Flow velocity downstream of the finalized dissipator at the design flood and normal downstream water stage

Reports from the model tests are listed below.

Yang, J (2002), Porsi dam safety – hydraulic model studies, stage 1 – existing flow situation (*Porsi dammsäkerhet: hydrauliska modellförsök, etapp 1 - nuvarande situation),* Report No. U 02:03.

Yang, J (2003), Porsi dam safety – hydraulic model studies, stage 2 – refurbishment measures (*Porsi dammsäkerhet: hydrauliska modellförsök, etapp 2 – ombyggnadsåtgärder*), Report No. U 03:20.

Yang, J, Johansson, N & Cederström, M (2002), "Towards Safer Dams – Refurbishment Examples in Vattenfall's Dam-Safety Program", HydroVision 2002**,** July/Aug. 2002, Portland, OR.

14 Rusfors

Rusfors was completed in 1962, Figure 14.1. The power station is equipped with one generating unit, operating at a 12 m gross head and a 450 m^3/s turbine discharge. The dam is of en earth-fill type, with a max. dam height of 22 m and a crest length of 940 m. The active storage volume of the dam accounts to 75 Mm^3.

The dam consists of two overflow spillway openings, with upward-going radial gates. The total spillway discharge is ~1600 m^3/s at the FRRL.

Figure 14.1 Rusfors dam with spillway (photo: Leif Kuhlin)

14.1 Damages in stilling basin

The hydraulic concern in Rusfors is not the discharge capacity - the spillway is large enough to discharge the design flood. The concern is instead the damages in the bottom of the stilling basin of the left spillway opening (B), Figure 14.2.

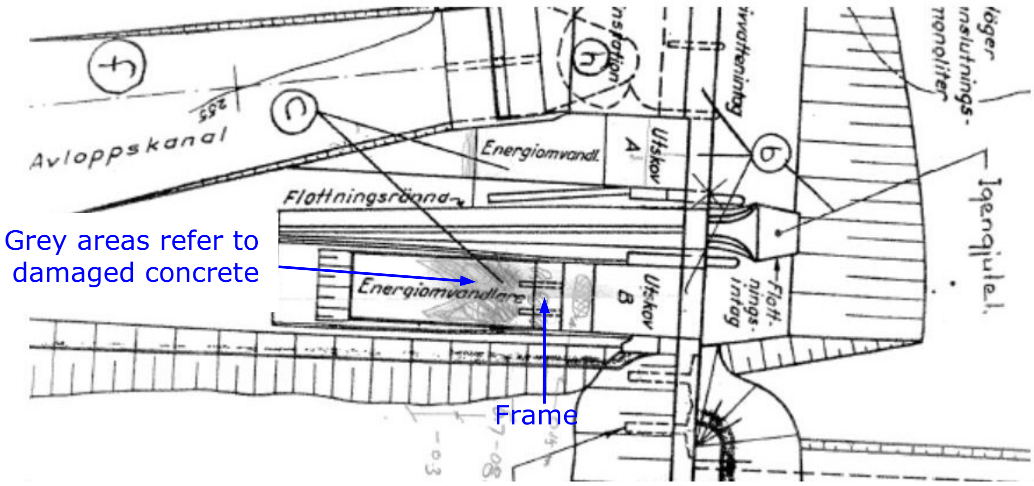


Figure 14.2 Damages in the left spillway opening at Rusfors

Mainly due to previous log floating in this spillway opening, a concrete frame, consisting of two parallel supporting walls and a cover, was left in the in the stilling basin, Figure 14.3. Besides, an abrupt drop in the bottom elevation features the ski jump upstream of the frame.

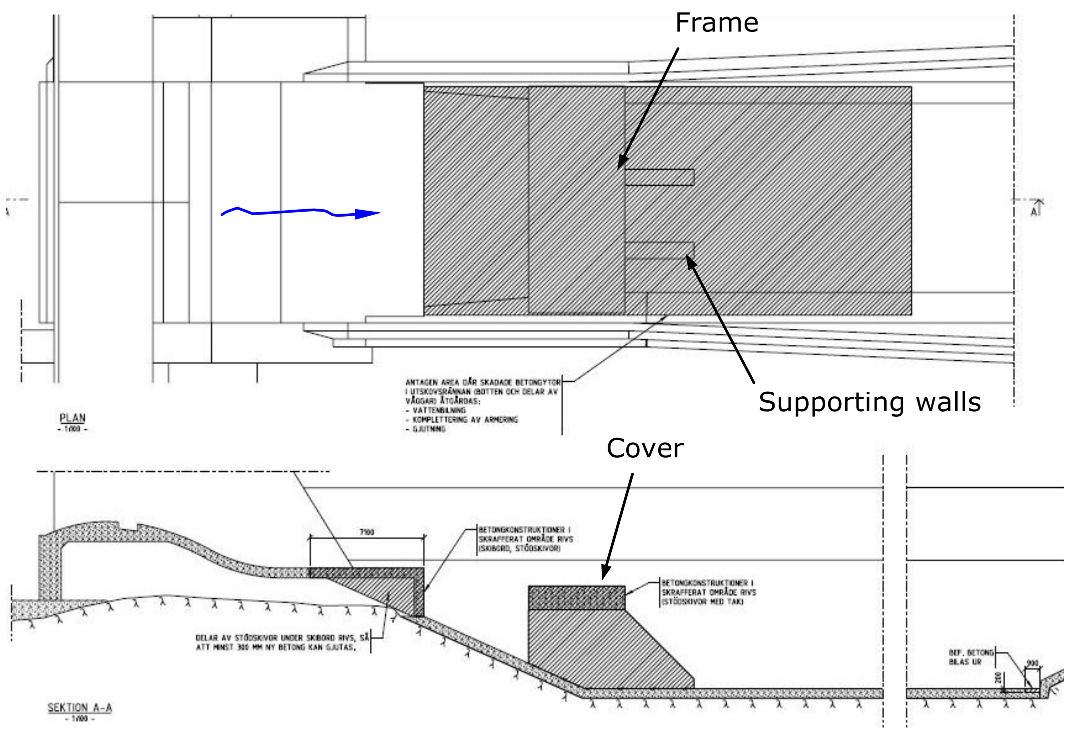


Figure 14.3 Layout of existing structure in the left spillway opening

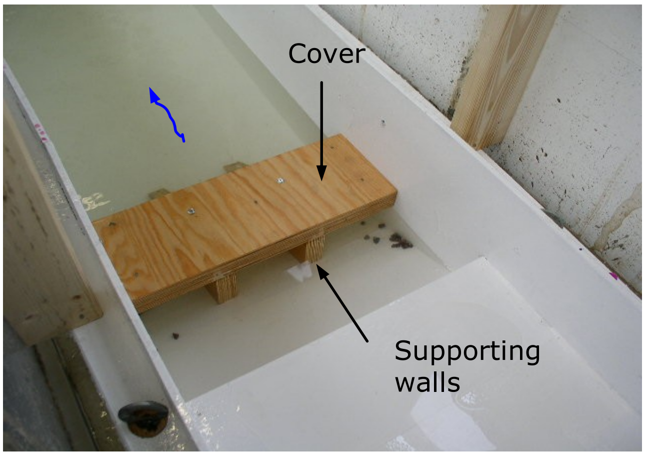
During the years of spillway operation, damages occurred in the stilling basin. Minor damages were found in the bottom downstream of the elevation drop, with a depth of 15 cm and exposed reinforcement bars (re-bars) covering a large area.

Downstream of the frame, concrete and re-bars were removed connecting on to the supporting walls, with a trench of 70 – 80 m wide running towards each sidewall. No re-bars were left in the trenches and only remainders of re-bars were visible along the trench edges, Figure 14.2. Concrete damages continued upstream into the frame, with holes and exposed re-bars. The max. hole length was 12 m. Areas with a eroded depth of 30 - 50 cm stretched over a length of 10 m.

It was not clear under which hydraulic conditions (regular or high spillway discharges) the damages occurred.

14.2 Flume tests

Due to the flow complexity, a model was built in a flume to study the flow behaviors and countermeasures, Figure 14.4. The model covered the whole waterway including the spillway and the stilling basin. The model scale was 1:30.



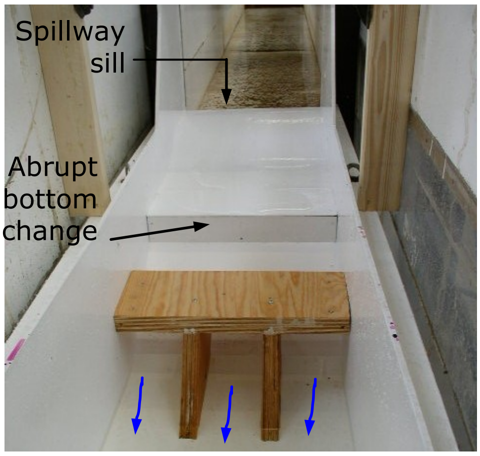


Figure 14.4 Rusfors model of energy dissipator in a flume

It can be shown in the tests that the presence of the frame caused very turbulent flow situations in the basin. As the ski jump had a higher elevation than the cover position, rotations of strong surface flow with waves prevailed in the flume, which was undesirable, Figure 14.5. The supporting walls separated the flow beneath into three "jets" that merged downstream and a kind of wake flow formed also immediately behind each wall. The basin flow downstream of the frame oscillated laterally. It can be deduced that the flow pattern caused by the supporting walls, in combination with e.g. stones and gravels in the water, gave rise to the basin damages.

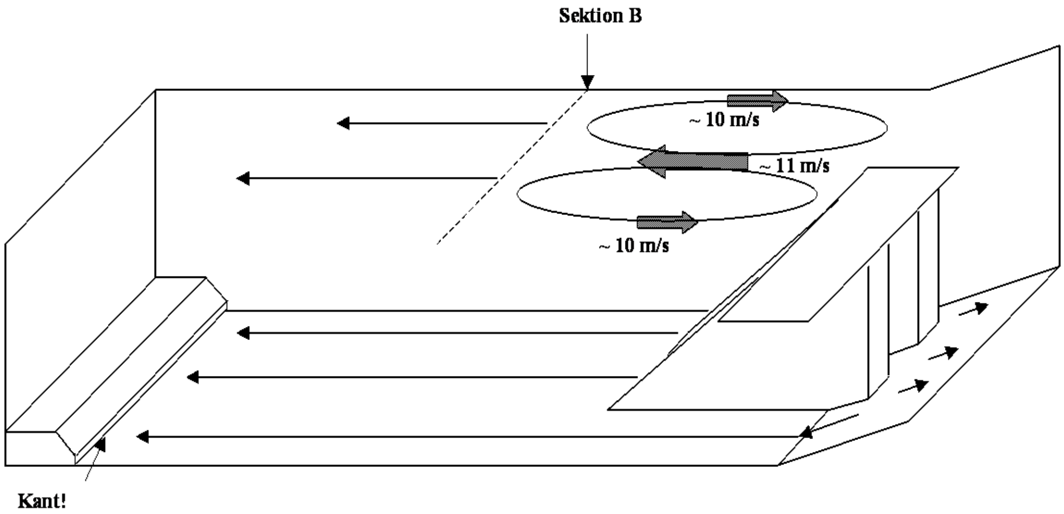


Figure 14.5 Circulations of surface flow with high velocity and waves featured the existing layout

To remove only the frame cover improved slightly the flow but did not solve the problem. Therefore, the tests suggested that the whole frame be removed. In so doing, the oscillating flow pattern disappeared totally. However, the strong surface currents still characterized the basin flow.

It can be further demonstrated that if the lower part of the horizontal ski jump was removed and the abrupt elevation difference disappeared, the basin would be changed to almost a standard energy dissipator, with the main flow on the bottom and a well-positioned hydraulic jump.

In Figure 14.3, the drawing below shows how the frame was removed and how the ski jump was modified. With both the frame removed and the ski jump modified, the flow pattern is shown in Figure 14.6. The final adopted shape in Rusfors is illustrated in Figure 14.7.

The right spillway opening (A) was untouched.

Figure 14.6 Flow pattern after removed frame and modified ski jump

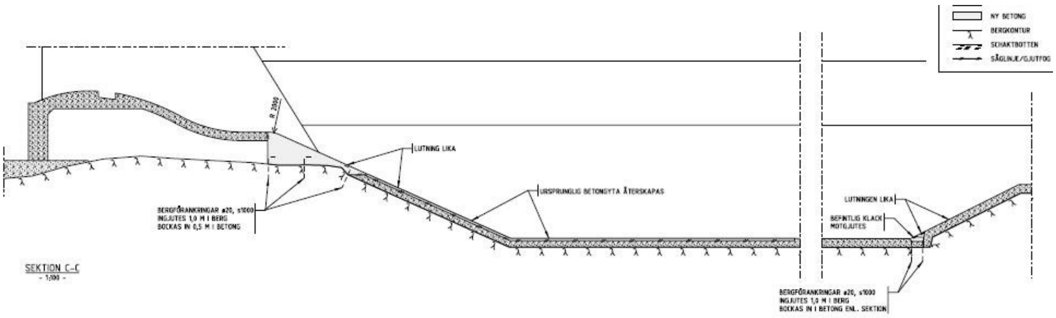


Figure 14.7 Re-shaping the stilling basin in Rusfors - final adopted longitudinal profile

The following report deals with the model tests for Rusfors.

Larsson, P & Yang, J (2006), Rebuilding of Rusfors dam – hydraulic model investigations *(Ombyggnad av Rusfors – hydrauliska modellförsök),* Report No. U 06:55.

15 Satisjaure

The Satisjaure dam was constructed during 1962 – 67. The dam is of an embankment type, with a maximum height of 30 m, Figure 15.1. The dam is equipped with a bottom spillway with two openings, whose dimension is 4.5 m (wide) by 6.8 m (height) each, Figure 15.2 and 15.3. According to the model tests made in 1962, the maximum capacity at the FRRL (+457.0 m) is 810 m^3/s. The main data of the scheme is listed below in Table 15.1.

Figure 15.1 Satisjaure dam (photo Leif Kuhlin)

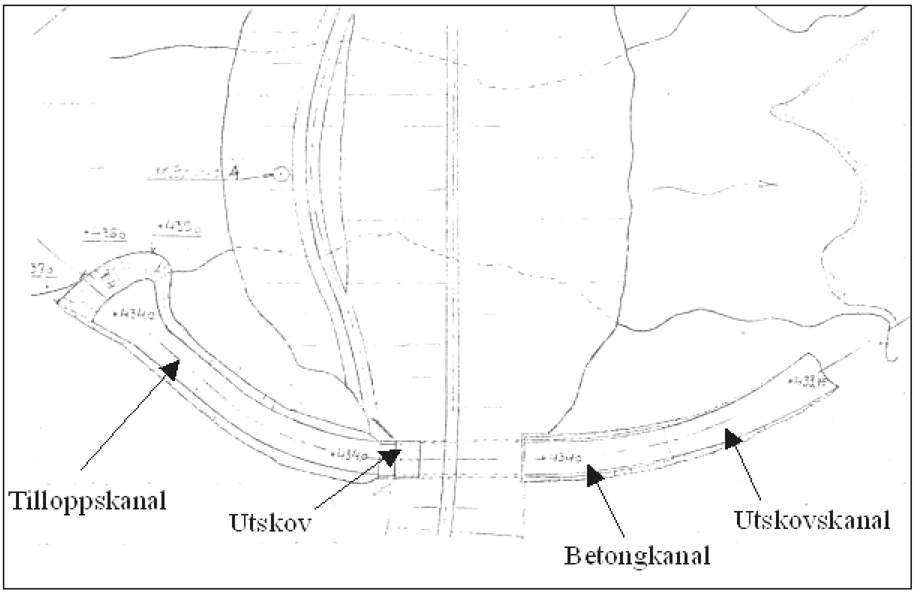


Figure 15.2 Dam and spillway layout

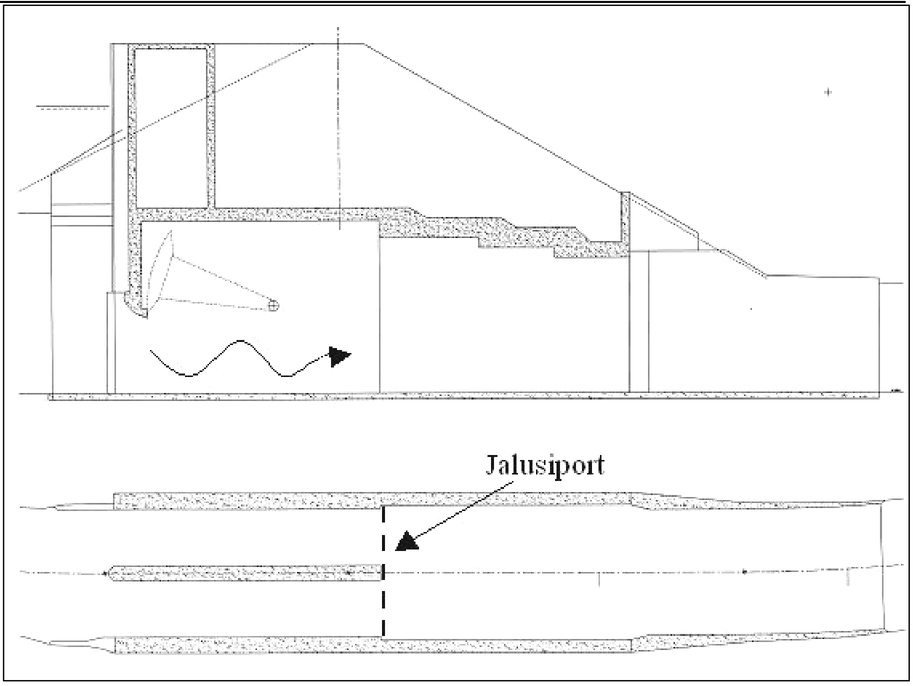


Figure 15.3 Bottom spillway

Table 15.1 Satisjaure dam data

Item	**Data**
Active storage volume	$1240*10^6$ m^3
Catchment area	2324 km^2
Design inflow	1100 m^3/s
Max. discharge capacity	810 m^3/s
FRRL	+457.0 m
Low reservoir limit	+438.0 m
Crest length	1450 m
Crest elevation	+462.0 m
Impervious core elevation	+458.5 m
Outlet sill elevation	+434.0 m
Gate type	Radial gates
Outlet opening width	2x4.5 m
Outlet opening height	6.0 m

15.1 Safety and environmental concerns

In 1994, dam-safety evaluations were made for the dam, in which it was concluded that the rock foundation the dam tests on has, despite of fractures and shear cracks, generally good quality.

As part of the evaluations, the bottom spillway was tested for flood release, with both openings fully open. The reservoir level was at +456.6 m, the discharge corresponded to 780 m^3/s and lasted about 10 min. During this short test, it was observed that the spillway channel was overtopped on its right side and tail-water level was high against the dam toe. After the release, the spillway channel and the riverbed downstream were inspected. At both places, erosion was found, partly in the rock and partly in the fine materials lying in the river.

The design inflow of the dam is 1100 m^3/s and the spillway design flood is 810 m^3/s. Discharge of the design flood with the existing spillway during a long period of time would certainly pose a risk of rock erosion in the spillway channel, which in turn affects the dam safety. Furthermore, there exists risk for erosion of the fine materials and river vegetations, which is, from the environmental point of view, not acceptable.

15.2 Model testing

The previous model study for Satisjaure was made in Älvkarleby 1962 before the dam was constructed. The model for the dam rebuilding was constructed in 2002, with a chosen model scale of 1:50. The model was 8.0 m long and 3.5 m wide, corresponding to a prototype size of 360 m by 170 m. The emphasis of the study was the energy dissipation and downstream area, but the approaching flow conditions were also properly modeled. Figure 15.4 shows the model.

The issues that were examined in the model included

- Bottom outlet discharge capacity
- Documentation of the flow pattern in the existing layout in terms of flow velocity and water stages in the downstream area
- Suggestion and optimization of a stilling basin to minimize erosion risk downstream
- Documentation of the flow pattern with the final stilling-basin layout
- Control if any operation restrains should apply to the bottom spillway
- Measurement of water depth at potential location for the cofferdam

As was in the flood test of 1994, the right sidewall of the stilling basin was overtopped at the design flood, which was undesirable, Figure 15.5. Measurements pointed also to very higher flow velocity downstream in the river valley.

Figure 15.4 Model of Satisjaure, in scale 1:50, with energy dissipation before any excavation was made

Figure 15.5 Overtopping of right channel side at design flood

A number of tests were made, with efforts of trying to find out a proper layout of the stilling basin. The final basin shaping worked out is illustrated in Figure 15.6. A few steps were given at its upstream part to facilitate construction. Besides, a vertical deflector was added on the right wall of the rectangular canal to deflect the flow a little bit from the wall. The defector was positioned upstream of the steps and at the location where the concrete wall ended.

Figure 15.6 Devising a new stilling basin through rock excavation

Figure 15.7 demonstrates the improved energy dissipation at the design flood.

Figure 15.8 illustrates the flow velocity vectors before and after the excavation of the dissipator at the design flood discharge.

Figure 15.9 show even the comparison of the velocity magnitudes at section no. 15. The flow velocity reached as high as 11 – 12 m/s in the original layout. With the proposed energy dissipator, the velocity was reduced to a maximum of some 6 m/s.

Figure 15.10 depicts the energy dissipator under construction.

Figure 15.7 Energy dissipation in the excavation basin at the design flood

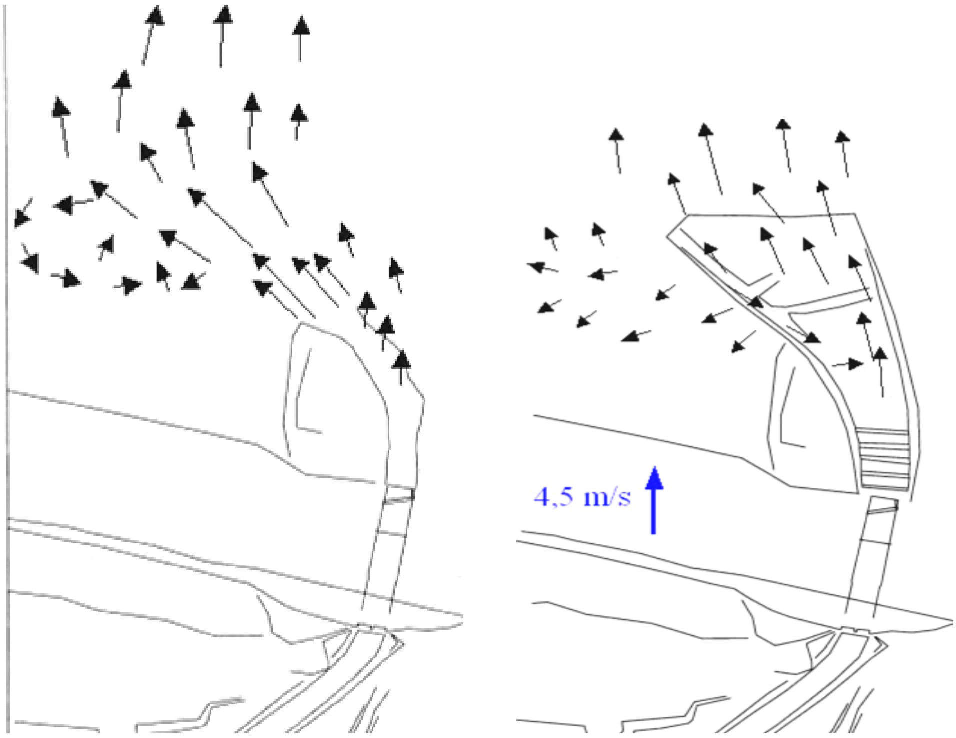
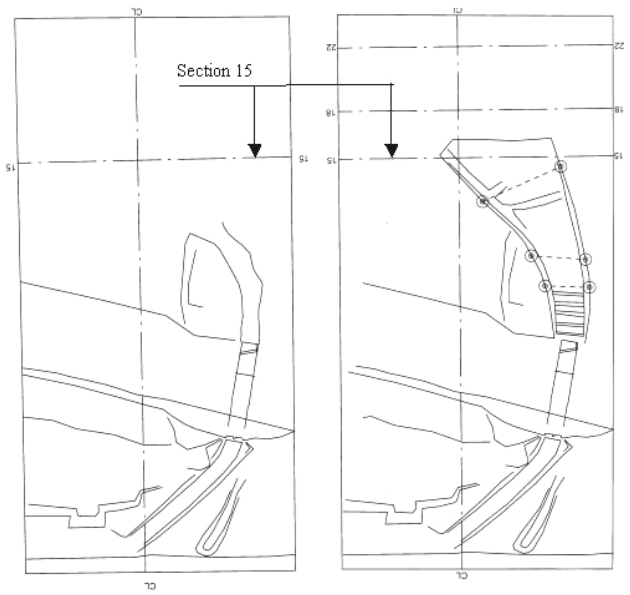
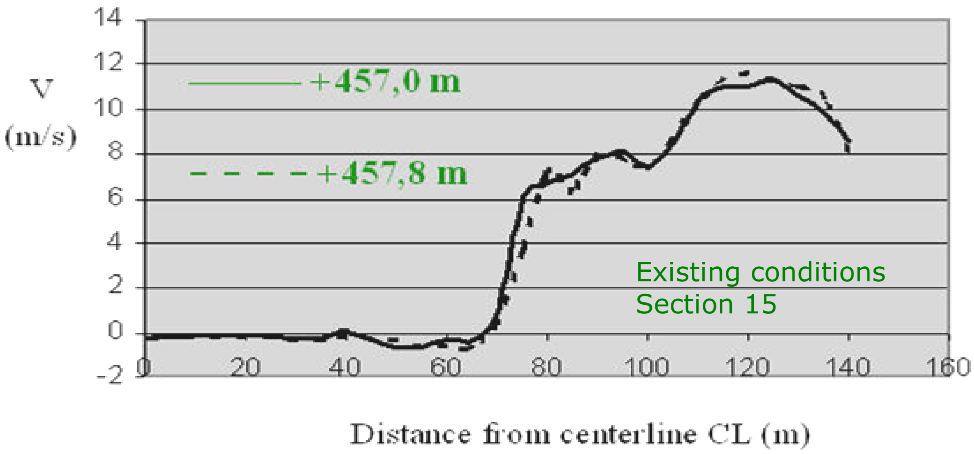


Figure 15.8 Comparison of flow pattern before and after the basin excavation





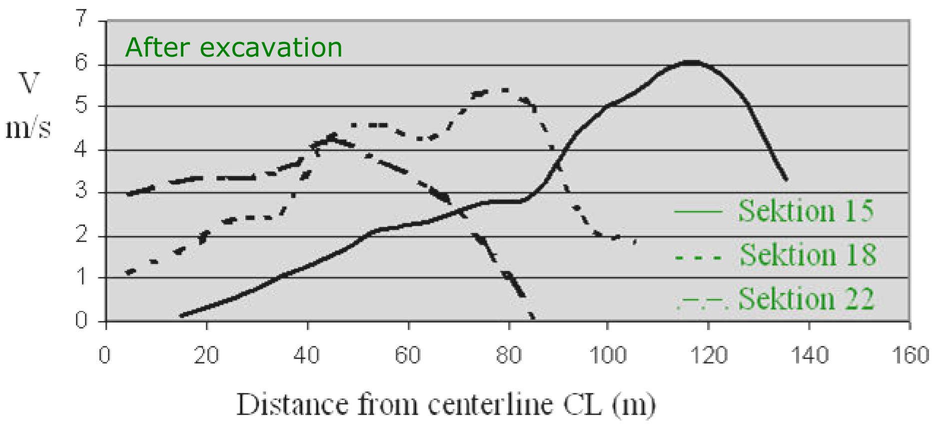


Figure 15.9 Flow velocity downstream before and after basin excavation at the design discharge

Figure 15.10 Proposed stilling basin under construction

Satisjaure model tests are summarized in the following reports.

Billstein, B (2003), Hydraulic model tests Satisjaure 2002-03, adaptation to new design flood (*Modellförsök Satisjaure 2002-03, anpassning till nya dimensionerande flöden*), report U 03:32.

Yang, J & Amnell, G (2005), Satisjaure dam - spillway discharge capacity, hydraulic model tests (*Utskov i Satisjaure – avbördningsförmåga, hydrauliska modellförsök*), Report No. U 05:26.

16 Stenkullafors

Stenkullafors was commissioned in 1983. Its layout and aerial view are shown in Figure 16.1 and 16.2. The dam, partially founded on grouted bedrock, is of an earth-fill type with an impervious core of moraine, a filter of natural sand and shoulders of a natural mixture of sand, stones and boulders. The dam has a total crest length of ~600 m and a maximum height of 30 m above the ground. The intake to the power station is located to the right of the spillway structure. The power station is equipped with one Kaplan turbine with an installed capacity of ~60 MW, operating at a gross head of ~23 m and a nominal turbine discharge of 285 m3/s. The average annual output amounts to ~230 GWh.

The original spillway section was of a conventional design of two 10-m overflow openings with tainter gates. The FRRL of the dam is +326.5 m, at which the original spillway discharge capacity corresponded to approximately 1250 m3/s. The spillway is equipped with an energy dissipator in natural rock and the tailrace canal is surrounded with embankment on both sides.

The design discharge is ~40% larger than the existing capacity. With the existing spillway, the resulting reservoir water level would be approximately 2 m above the FRRL. The revised flood must be safety discharged, while the dam structural integrity is maintained.

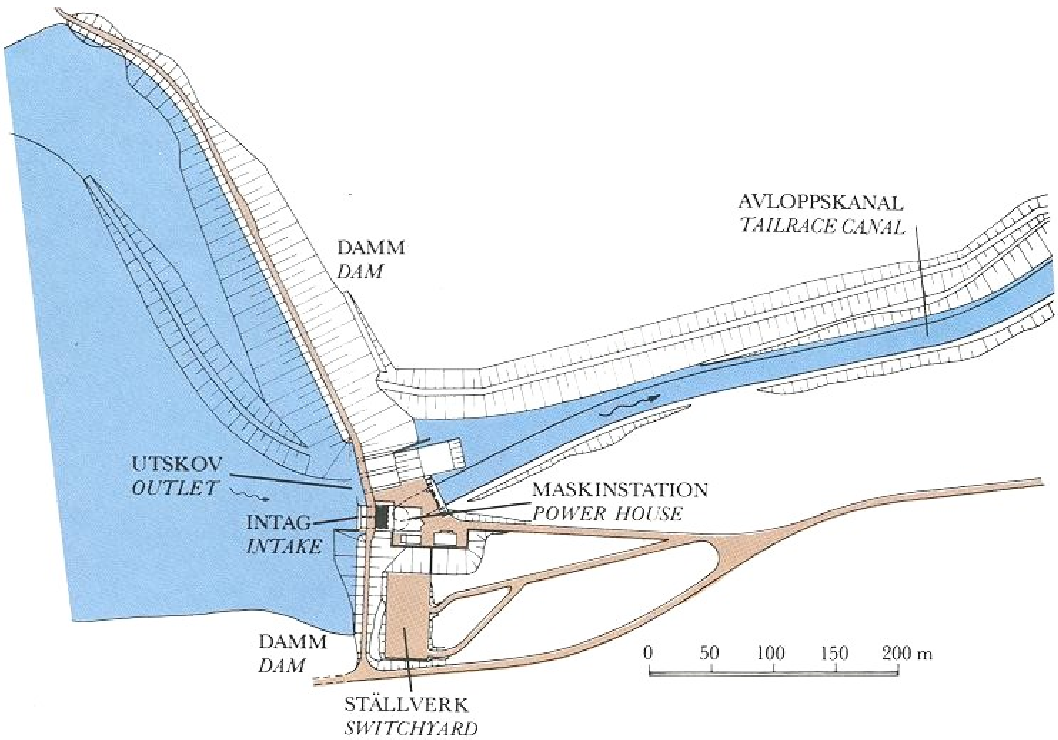


Figure 16.1 Stenkullafors dam layout

Figure 16.2 Stenkullafors dam, with spillway and tailrace canal, aerial view

16.1 Engineering measures

The higher safety requirements have called for a number of engineering measures such as dam reinforcement, including construction of a toe berm, strengthening of the spillway structure, new erosion protection on the upstream slope, etc. Restricted by a number of factors, the reservoir water level is not permitted to rise above the FRRL during the design flood. A higher level than the FRRL might involve safety risk in two ways. The dam is partly founded on the bedrock and partly on previous soil of sand and gravel grouted with cement and silicate; there might be increased seepage leakage in the foundation. There is also risk for higher pore pressure in the dam body that might give rise to internal erosion.

Based on the cost-benefit analysis, modifying the overflow spillway is chosen as the primary measure to handle the flood. The final decision is to rebuild the **right** spillway opening to a lower sill elevation while its width remains the same. The left opening is left untouched. The reason is twofold. More water will be discharged through the rebuilt opening (no matter which one), resulting in more concentrated jet flow into the stilling basin. The right opening is directed to the middle of the spillway channel and the flow pattern in the basin is more favorable and symmetrical. Rebuilding of the left opening would undoubtedly lead to highly distorted flow pattern downstream. Another reason, probably more conclusive, is that the upstream vertical front of the left opening intersects the dam slope. This would lead to geometric difficulties and also make it difficult to place the arch beam stop-logs for the spillway closure. The riverbed upstream of the right opening is even and suits better the placement of the stop-logs, Figure 16.3.

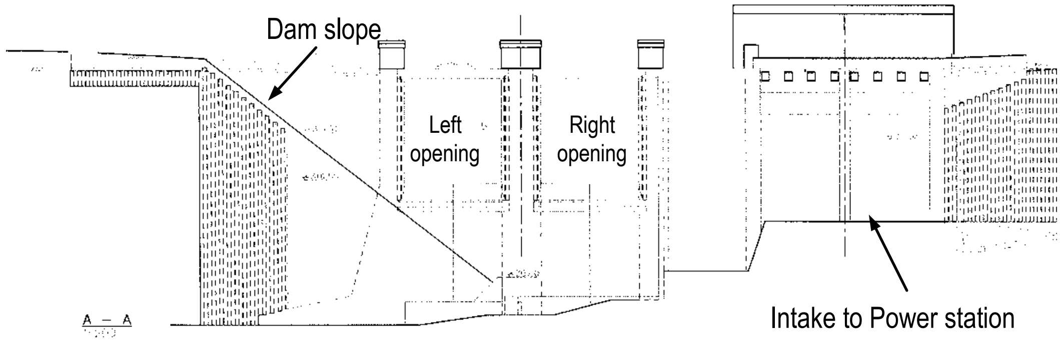


Figure 16.3 Existing spillway layout, looking downstream

16.2 Hydraulic model studies

A hydraulic model, in scale 1:50, was built to study different ways to modify the right spillway opening, Figure 16.4. The model was used to examine the following issues associated with dam safety.

Included in the tests were the spillway discharge capacity, floating debris, energy dissipation and risk for erosion in the spillway channel. The purpose was to work out a relatively optimal crest profile, so that the design flood was discharged roughly at the FRRL.

The existing spillway crest had a standard WES shape, with a sill elevation at +316.5 m. The river bottom upstream of the spillway was 12.5 m below the sill elevation, so there was potential to lower the sill, without much worsening in the flow to the spillway. Usually, a standard WES crest is considered to have an optimal hydraulic behavior as far as the spillway capacity is concerned. Any sill lowering implies, to varying extent, deterioration as the new crest becomes somewhat flatter as to avoid extensive chipping of concrete. However, there is always potential to optimize the crest shape in an overflow opening.

Spillway modification is a compromise between rebuilding costs and hydraulic requirements. A favorable crest reduces the height of the radial gate and releases the same discharge as a less favorable crest with a lower sill elevation. Three crest shapes, B, C and D, with sill el. at +311.7, +312.5 and +312.2 m respectively, were examined in model tests, Figure 16.5. The aim was to release the design flood at FRRL and at the same time to optimize the crest so as to reduce the height of the new radial gate. To avoid extensive chipping of concrete, all the crests were given a flatter shape than the existing.

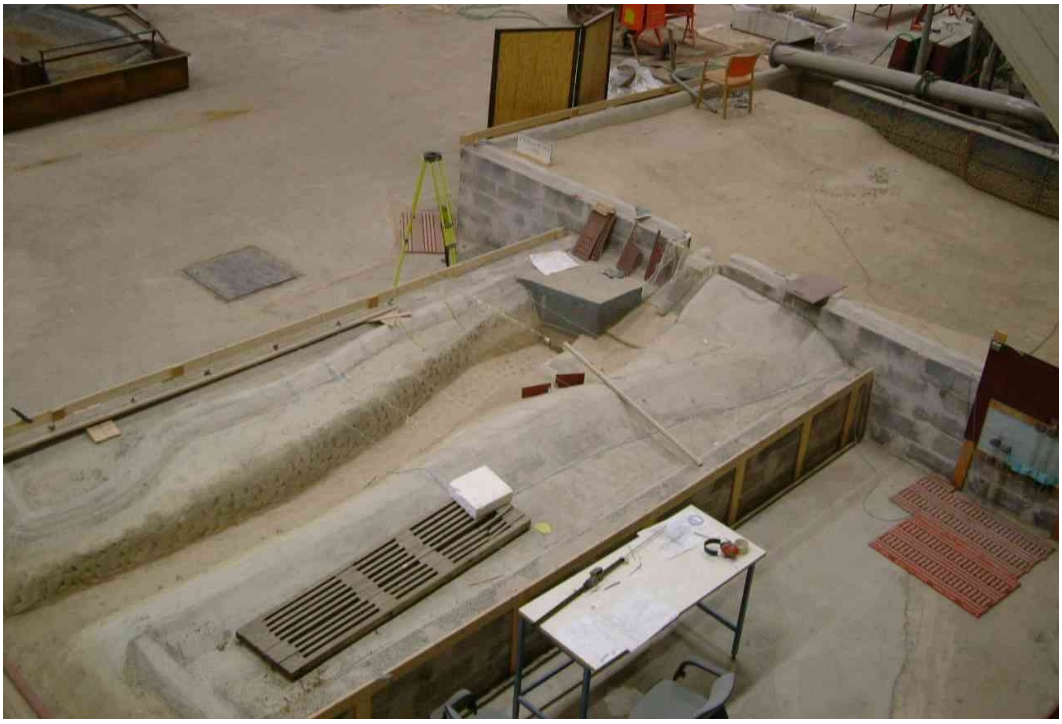


Figure 16.4 Model of Stenkullafors, scale 1:50

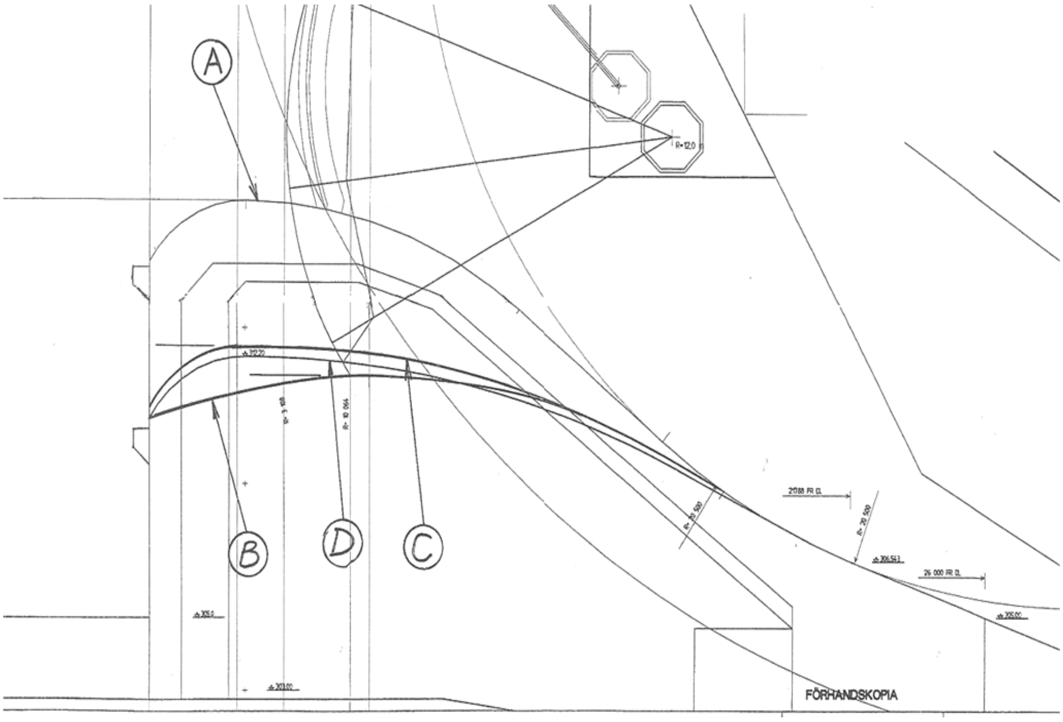
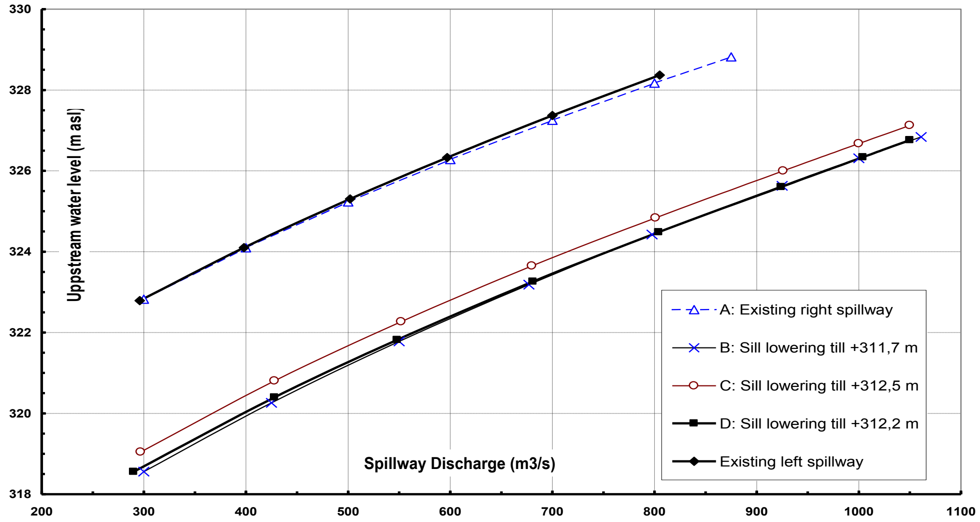


Figure 16.5 Optimizing spillway crest - shape A: existing crest, el. +316.5 m; shape B: el. +311.7 m; shape C: +312.5 m & shape D: +312.2 m

The test results of Q-Z and discharge coefficient are presented in Fig. 16.6. The Q-Z relationship of the left opening is also included. Of all the tested shapes, shape B gave the lowest discharge coefficient and was the most unfavorable profile. Compared with shape B, shape C reduced the gate height by 0.8 m. However, the resulting water level exceeded FRRL by about 0.2 m, which was not accepted.



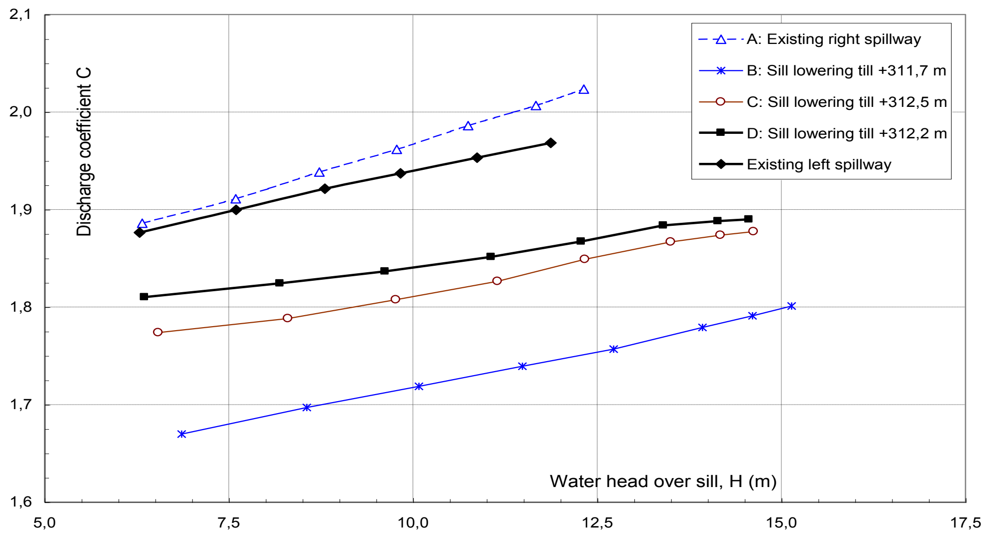


Figure 16.6 Stenkullafors crest optimization – result of Q-Z and discharge coefficient

Shape D was an improvement from C. Its downstream part was given somewhat lower elevation to reduce the pressure acting on the flow. Although the sill in shape D was located 0.5 m higher than in B, they gave almost identical reservoir water level, which was conducive to its favorable crest profile. With shape D, the design flood was discharged at FRRL and the gate was 0.5 m lower than with shape B.

The crest finally chosen had a sill elevation at +312.2 m, i.e. 4.3 m lower than the existing sill elevation, Figure 16.7 and 16.8. The discharge capacity before and after the sill lowering is shown in Figure 16.9. The design condition was changed at a later stage. Due to this, the design flood increased as an upstream dam was allowed to break in case of high floods. With this crest and new design conditions, the design flood would be discharged at a reservoir level some 40 cm above the FRRL.

Figure 16.7 Spillway modification - crest lowering and optimization through model tests

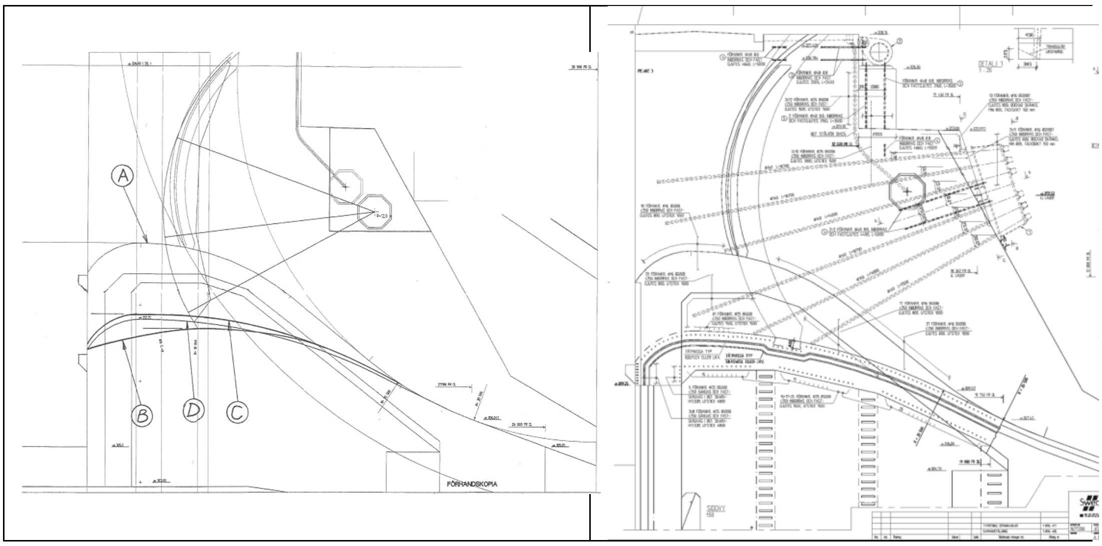


Figure 16.8 Spillway crest optimization in the model (left) and final adopted crest profile (right), with a sill elevation of +312.2 m, i.e. 4.3 m lower than the existing sill elevation

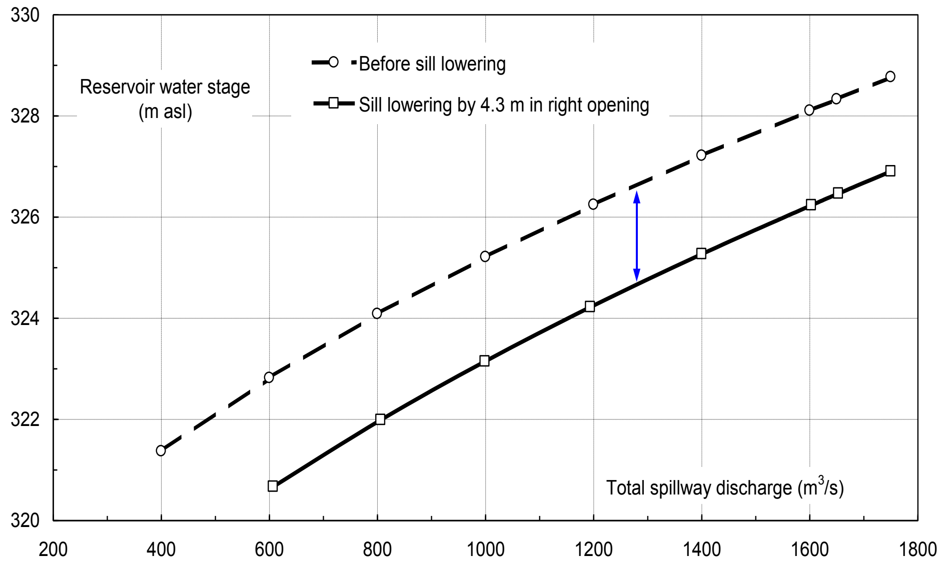
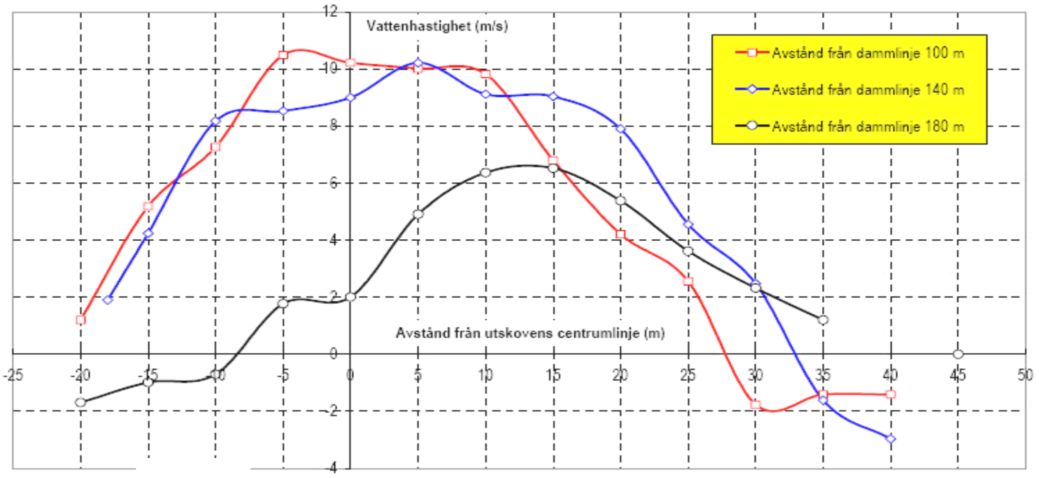


Figure 16.9 Reservoir water level before and after sill lowering in right spillway opening

Figure 16.10 shows the flow patterns and energy dissipation after the chosen sill lowering at the spillway discharge 1650 m^3/s. The corresponding flow velocity downstream is given in Figure 16.11.

Figure 16.10 Energy dissipation after chosen threshold lowering



Figur 15.13 Höjd avbördning i Åsele - hastighet nedströms energiomvandlare vid 1650 m^3/s, sedd nedströms (vid tröskelsänkning till +312,2 m i höger utskov)

Figure 16.1[illegible] ured flow velocity after sill lowering in Stenkullafors

Figure 16.12 shows the spillway at Stenkullafors before and after the modification. The sill elevation was lowered by 4.3 m in the right opening. The left opening remains unchanged.

Figure 16.12 Spillway at Stenkullafors before and after the rebuilding

16.3 Comments

Vattenfall has now rebuilt the Stenkullafors dam to adapt it to the updated dam-safety guidelines. The engineering measures included reinforcement of the dam, stabilization of the dam toe against slope slide and seepage damage, and new slope protection in the reservoir. To avoid any problems with increased leakage in the foundation or internal erosion in the dam body, extra retention of water during the design flood was not a design measure. The modification of the right spillway opening was the major measure to deal with the increased design flood.

With a large distance from the spillway sill to the river bottom, there was potential to achieve a favorable crest when the existing spillway was modified. Through hydraulic model tests, a practically optimal crest shape was adopted, and a new, higher segment gate was installed. The new sill elevation was located 4.3 m lower than the existing. To avoid expensive chipping of concrete, a somewhat flatter crest profile was adopted. The solution for the spillway rebuilding is unique and cost-effective when considering that all construction cost that might arise due to higher reservoir water level than the FRRL were avoided.

Plans for handling the hazard of high flood during the construction, when the discharge capacity was reduced to half, were also worked out in the early stage of the project. The construction period was thus limited to the winter season so as to reduce the flood risk to an accepted level.

To successfully carry out the re-building, it was important to create safe working conditions and high accuracy of concrete casting and at the same time to have a tight yet realistic time schedule. Under these circumstances, the dam owner chose a partnering agreement with the contractor to create a cooperative work and to define common targets.

Publications from Stenkullafors model tests are listed below.

Yang, J (2003), Stenkullafors dam: adaptation to new design flood – hydraulic investigation *(Stenkullafors kraftstation: anpassning efter nytt dimensionerande flöde – hydrauliska utredningar),* Report No. U 03:23.

Yang, J & Göthlin C. (2003), Stenkullafors power station: spillway discharge characteristics after rebuilding *(Stenkullafors kraftstation – utskovsavbördning efter ombyggnad)*, Report No. U 03:24.

Yang, J & Cederström, M (2007), "Modification of spillways for higher discharge Capacity", Journal of Hydraulic Research, Vol. 45, No. 5, 2007.

Yang, J, Hellstadius, K & Cederström, M (2008), "Adapting the Spillway for Updated Design Flood at Stenkullafors Dam, North Sweden", Intl. Symp. at 76th ICOLD Annual Meeting, June 2008, Sofia, Bulgaria.

Yang, J, Hellstadius, K & Cederström, M (2008), "Case study of spillway modifications – a means to a higher level of dam safety", Journal of Geotechnical Engineering, Vol. 30, No. 11, 2008.

17 Storfinnforsen

Storfinnforsen, situated on Faxälven, was completed in 1953. The dam is totally ~1200 m long, of which the concrete dam accounts for ~800 m (600 m buttress dam, plant intake and spillway), Figure 17.1. The buttress dam comprises 66 independent "units" or monoliths. Each monolith is composed of an 8 m wide buttress head and a web. The highest monolith is 39 m high.

Figure 17.1 Storfinnforsen dam spillway

The FRRL of the dam is +273.0m. The flood discharge structures of the dam consist of the following parts.

- Two 20-m overflow spillway openings with upward tainter gates and a total discharge capacity of 900 m^3/s at the FRRL.
- One 12-m overflow spillway with downward sector gate and a discharge capacity of 250 m^3/s at the FRRL.
- Two bottom outlets, with a dimension of 5.5 m (width) by 3.65 m (height) and a sill elevation of +236.0 m (i.e. 37 m under the FRRL). The left outlet was permanently sealed long time ago. The right outlet operates with a upward radial gate.
- A log flume is closed with stop-logs and its capacity is excluded from the total capacity of the dam.

17.1 Safety concerns

The design flood of Storfinnforsen can be totally handled through the overflow spillways. Its right bottom outlet does not need to participate in the flood discharge. However, it does need to fulfil other dam-safety functions, partly of lowering the reservoir when required and partly of acting as reserve. According to the dam owner, the right bottom outlet was never used when the dam was accomplished. Figure 17.2 shows the discharge canal downstream of the outlets.

Figure 17.2 Discharge channel downstream of the bottom outlets

In order that the right bottom outlet can be safely used for flood discharge, the radial gate must be in sufficiently good operational conditions and no detrimental effects would occur in the form of unstable flows or cavitation. It is not clear from the available documentation if the bottom outlets can be used at full reservoir levels or if they were only intended for use during the dam construction.

Therefore, the possibility of discharging floods at full reservoir retention levels and emptying the reservoir completely through the outlet must be verified. Mechanical issues of the bottom spillway were studied separately. Certain hydraulic issues were examined in a model study.

17.2 Model tests

The model for Storfinnforsen was constructed in scale 1:30, Figure 17.3. A length of 150 upstream and 200 downstream was included. The model width corresponded to a 90 m dam section, with the bottom spillway in the middle. The right outlet was equipped with a radial gate that could be easily maneuvered from downstream.

Figure 17.3 Storfinnforsen model in scale 1:30

The following issues were included in the model study.

- Outlet discharge capacity
- Investigation of discharge – water stage relationship downstream through 1D numerical modelling. This forms the downstream boundary condition for the model study.
- Flow conditions in the spillway channel (hydraulic jump and energy dissipation)
- Engineering measures against overtopping of the right sidewall
- Flow pattern in the channel due to discharge from left outlet and both outlets
- Strategy of reservoir emptying taking into account reservoir inflow and outflow to reservoir downstream (mathematical modelling)

The test of discharge capacity showed that the result was identical with that from model tests in 1950.

Calculations of downstream water stage at different outlet discharges were made with the computer code HEC-RAS. The reasonable value of Manning's coefficient fell within the range n = 0.03–0.05, which affected the water stage at the dam by ±0.5 m.

When discharging from the right outlet, the tests with the flow pattern in the channel can be summarised as follows.

- The channel bent to the left. That was why a strong non-uniform flow was formed in the channel. A calm back eddy (flow circulation) featured the left side of the channel.
- The main current took place along the right side of the channel and against the right sidewall. Irrespective of the outlet opening, the right sidewall was overtopped at high reservoir water levels. The overtopping water had a tendency to flow straight downstream outside the channel and would undermine the wall from behind, Figure 17.4. Necessary measures were required to prevent the overtopping and undermining.
- The flow velocity in the channel was as high as 20–21 m/s.
- Due to low downstream water stage, no hydraulic jump was formed, either in the channel or in the river downstream.
- Standing waves prevailed in the river downstream of the channel. At high reservoir water levels, the waves were at a location of some 30 m from the downstream edge of the right sidewall.

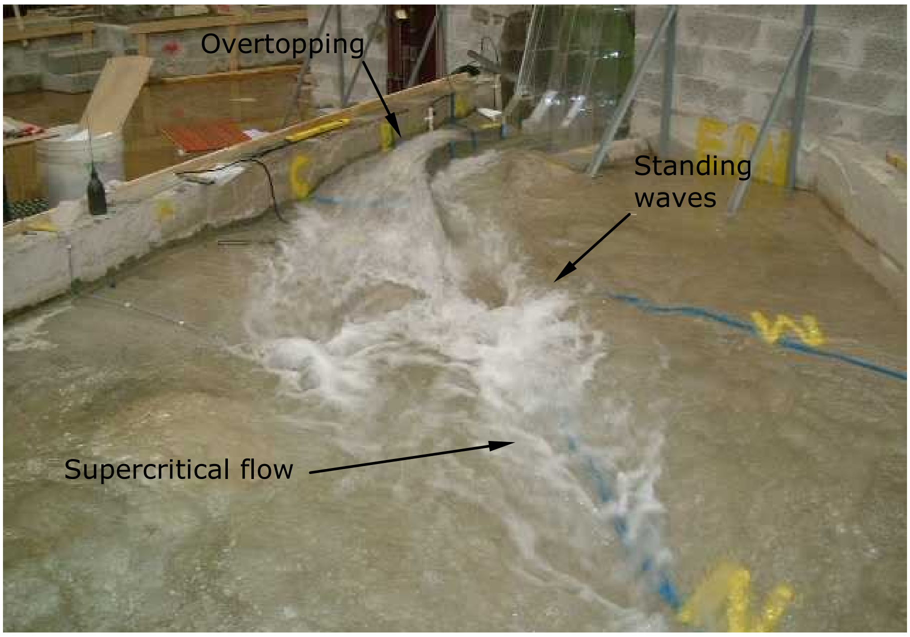


Figure 17.4 Discharge from right outlet - overtopping of right sidewall at high reservoir water levels

In the model, several measures in the form of baffle wall against overtopping of the right channel wall were tested, Figure 17.5 and 17.6.

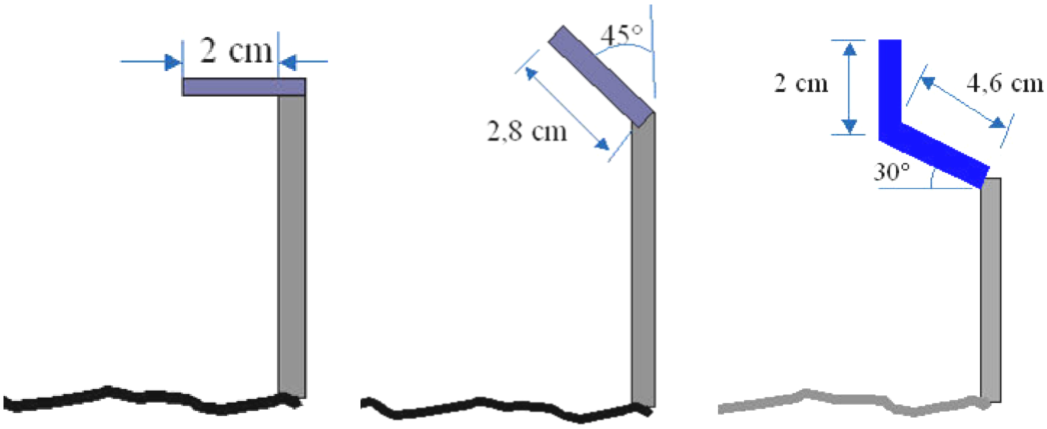


Figure 17.5 Shape of baffle walls in model scale - measures against overtopping of right channel wall

Figure 17.6 Use of 30° baffle wall against overtopping

The baffle wall reflected part of the water back to the channel and moved the beginning point of overtopping further downstream. Due to the fact that the channel bent to the left, it was not practical to try to prevent the overtopping along the whole length of the wall. To overtop the downstream part of the wall could be allowed, as the water over the wall flowed basically into the river course. Construction of the 30° baffle wall, with a vertical part that corresponded to supposed concrete thickness, seemed to be a feasible measure. The pressure acting on the baffle walls were also measured in different flow situations for the sake of design, Figure 17.7.

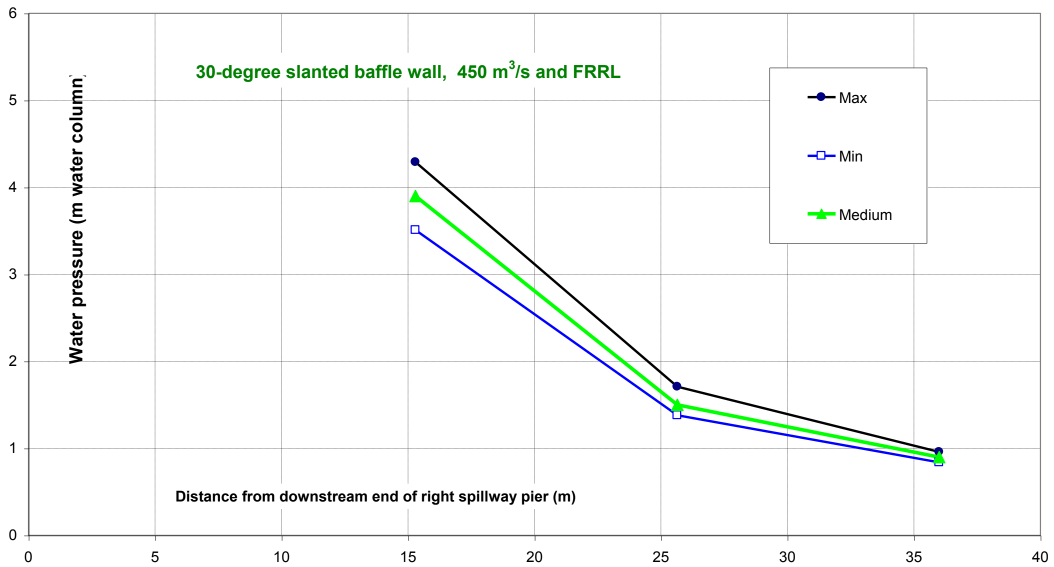


Figure 17.7 Right outlet discharge – prototype water pressure acting on the 30° baffle wall at full gate opening and full reservoir level FRRL

The discharge from the left outlet was examined also in the model. The outlet flowed directly against the right channel wall, implying that the wall was overtopped already at very low reservoir water levels. The water ran up the wall and air entrainment was extensive, Figure 17.8. Accounted from the channel bottom, the water ran as high as 19–21 m at the FRRL and splashed down on the riverbed some 12–15 m downstream of the channel, resulting in supercritical flow in the river. It was estimated that 60–65% of the outlet flow went outside the channel.

With both bottom outlets in fully open positions, the water did not run as high as with the left outlet. However, the overtopping was more serious in terms of water amount. About 70–75% of the total flow went outside the channel, Figure 17.9. Obviously, the bottom spillway was originally not designed for use at high reservoir levels.

It was recommended that, when the right outlet was fully open, the highest allowable reservoir stage for safety discharge from the left outlet was roughly +250.0 m with the 30° baffle wall and +248.0 m without any baffle wall.

Figure 17.8 Flow discharge only from the left outlet

Figure 17.9 Flow discharge from both bottom outlets

The need of air supply to the outlet flow, caused mainly by the high flow velocity of more than 20 m/s, was identified in the model to avoid cavitation in the channel bottom and negative air pressure downstream of the radial gate, Figure 17.10. However, limited by the model scale, the required air account, being a function of flow discharge, could not be determined. CFD modelling would fill the gap in this regard.

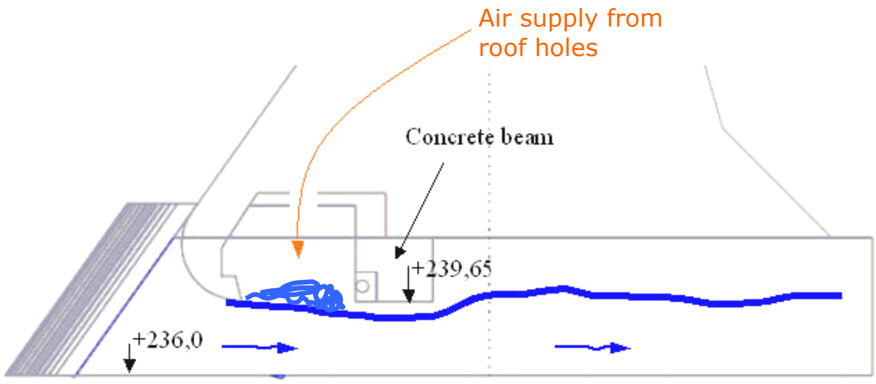


Figure 17.10 Need of air supply through openings above the gate to the flow

In the prototype, there are four holes in the roof of the gate chamber, which was also reproduced in the model with an equivalent hole area by scale. The air supply to the flow would be partly through the holes and partly through the space between the flow and the horizontal concrete beam for the gate bearing. Tests showed that, irrespective reservoir water level, free surface flow prevailed in the whole rectangular channel at all gate openings.

However, the space was quite limited at large gate openings. If the holes were sealed, the water surface would occasionally get in contact with the lower edge of the beam, and the water became thus “white”, with diffused water surface (as compared with clear water when the holes were all open). This implied that the water was aerated from beneath the beam and negative air pressure was produced in the gate chamber. The holes were therefore needed for aerating the flow. Numerical modelling should be made to clarify the issue.

Storfinnforsen model studies are described in the following report.

Yang, J, Amnell, G & Sundqvist, P (2008), Storfinnforsen – studies of hydraulic functions of bottom spillway *(Storfinnforsen kraftverk - utredning av bottenutskovs hydrauliska funktion),* Report No. U 08:25.

18 Stornorrfors

Stornorrfors is located on the lower Ume Älv, about 30 km from the Gulf of Bothnia, Figure 18.1. The power station has an installed capacity of ~600 MW. The main dam consists of a gated spillway section and a buttress dam with connecting rock-fill dams. It has a maximum height of ~20 m and a crest length of ~500 m.

The spillway consists of three openings, denoted as B, C and D from right to left, with a total width of 60 m, Figure 18.2. The flood discharge capacity is ~3300 m3/s at the FRRL. There is also a forth spillway opening prepared as reserve (denoted as A). It is however constructed with a concrete wall and the intention is to blast it if needed. This spillway is partly occupied by the upper part of a salmon ladder.

Figure 18.1 Aerial view of the Stornorrfors dam

Hydrological modeling carried out for the catchment confirms the need to increase the spillway capacity of the dam by some 35% from the existing level (reserve spillway not included). Vattenfall has decided that the rehabilitation of the dam should aim at discharging the design flood at the FRRL. The reason is to exempt the upstream area above the FRRL from being flooded, thus minimizing the damage to the environment – this area has a high touristic interest.

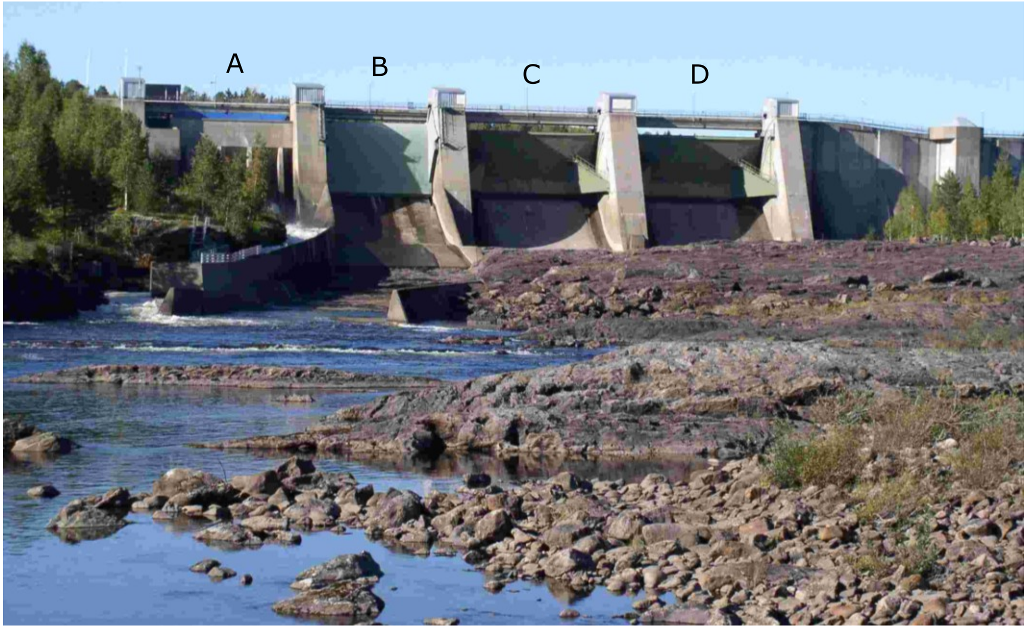


Figure 18.2 Spillway at Stornorrfors. Opening A is sealed with a concrete wall and partly occupied by the salmon ladder

18.1 Proposed rebuilding layouts

In a dam-safety analysis, a preliminary selection process leads to the retention of three rehabilitation alternatives of those originally envisaged.

- Option 1 – construction of a new spillway in a canal, excavated through an island, Tvärön, to the right of the dam.
- Option 2 – using the reserve spillway opening and construction of a new ladder for salmon
- Option 3 – building of a new spillway in the buttress dam by removing sections of it.

18.1.1 Option 1 – new Tvärön spillway

The terrain to the right of the dam, called Tvärön, is a suitable location for placement of a new spillway, separate from the existing. For this reason, geological investigations were conducted to map topsoil thickness, rock stratum and potential faults in the area.

A preliminary layout of a new spillway is shown in Figure 18.3. The canal is excavated in rock. From the hydraulic point of view, the spillway is ideally placed. First, the channel intake is skewed in relation to the spillway axis, so that it is directed towards the main river flow. Secondly, the narrow flow passage between the main river and the dam is avoided. Downstream, the water from the spillway is released at a distance from the fish ladder, which means that the spillway probably does not disturb the attracting water flow so much as spillways C and D do. About 20 m^3/s water is released through two valved pipes in the reserve spillway to lead salmon to the fish ladder.

The spillway is ~20 m wide and has a threshold elevation of about +65 m, the same as spillways C and D. The channel, concrete-lined upstream of the spillway, has a mild slope and an increasing width towards the river. Downstream, the chute is un-lined. So as to reduce the load acting on the unprotected rock, its width increases in the flow direction. Significant air entrainment is expected during flood release.

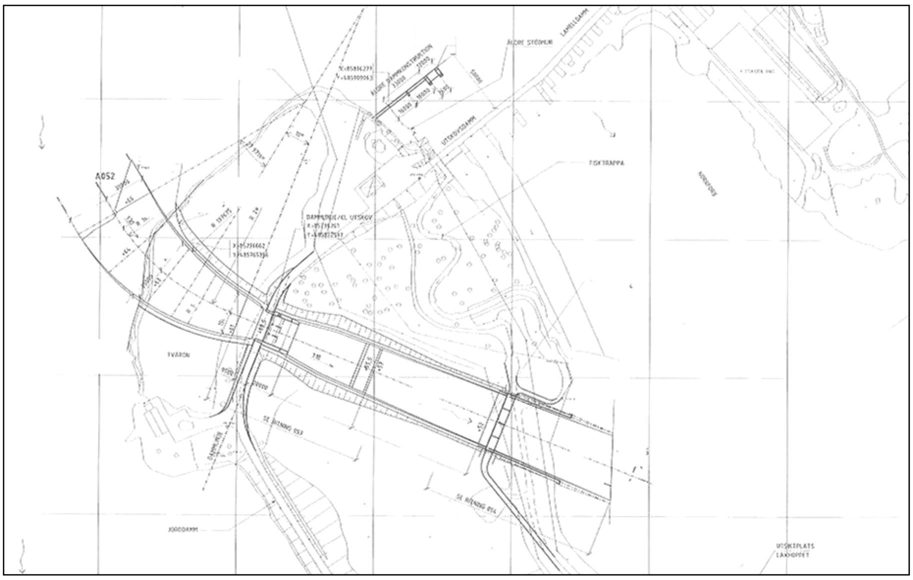


Figure 18.3 Option 1 – new spillway excavated in Tvärön

18.1.2 Option 2 – modification of old reserve spillway

Spillway A, a reserve spillway, is located at the very right of the dam and its opening has the width ~20 m, Figure 18.4. Part of it, 4 m, is now used as a fish ladder and the rest is a concrete wall. The original idea with the reserve spillway is to blast the wall in case of a severe flood.

If the fish ladder is left untouched and only the remaining part is rebuilt to an overflow spillway, the requirement to discharge the design flood at the FRRL could not be met. Consequently, it is proposed that the whole width of the opening should be utilized for flood release; the fish ladder be replaced by a new one, constructed separately to the right of the spillway. A preliminary layout of the spillway modification is shown in Figure 3. The spillway has a threshold elevation of +65 m and its crest is formed as standard WES profile. The radial gate is about 15 m high.

Downstream of the modified spillway, the waterway is widened by excavation in the right bank and a concrete sidewall is built as to obtain desired flow pattern at the junction of the fish ladder with the river. This is important especially at low flow discharges from the spillway.

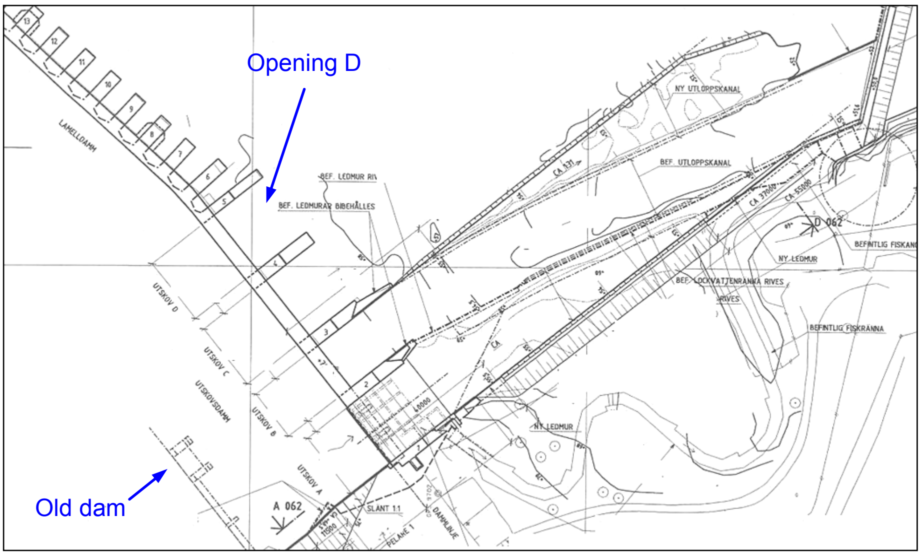


Figure 18.4 Option 2 – rebuilding of sealed reserve spillway

18.1.3 Option 3 – new spillway in buttress dam

A buttress dam, about 100 m long and 20 m high, followed by a connecting rock-fill dam, abuts the spillway section at left. The third rehabilitation option is to build a new spillway in the buttress dam. The proposed layout is illustrated in Figure 18.5.

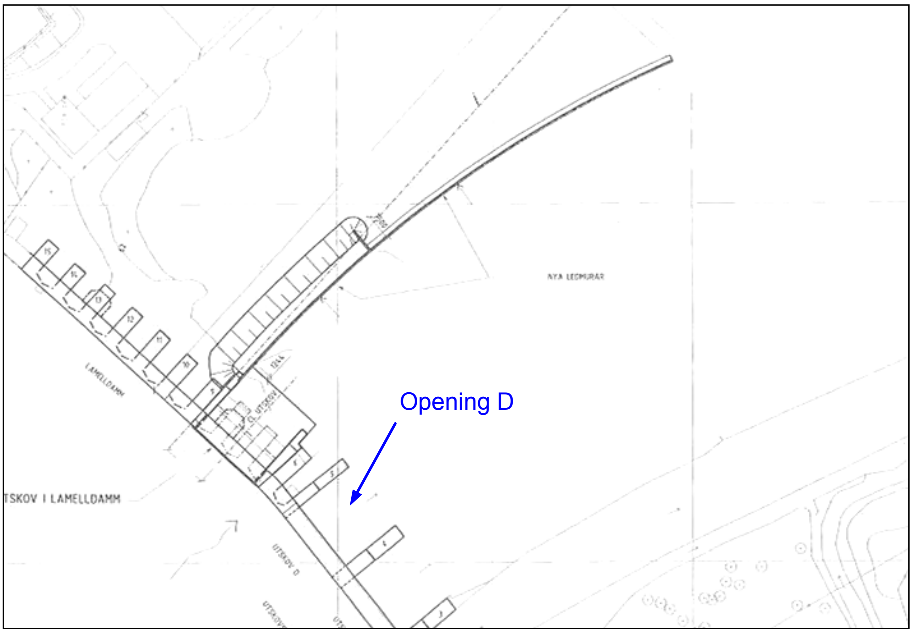


Figure 18.5 Option 3 – new spillway by removing buttress sections

Close to spillway D, two buttress "units", are removed. The supporting buttresses, often called webs, are concrete-reinforced by thickening and used as spillway piers. The new spillway is given a width of 22 m and a crest elevation of +66.0 m. Downstream of the spillway, a sidewall is suggested, extending from the left spillway pier. The purpose is to prevent the fish hatchery on the left bank from flooding and erosion in the connecting dam that adjoins the buttress dam.

18.2 Layout evaluations

Due to the complex topography of the river valley at the dam site, a hydraulic model was built so as to assist in the upgrading. The model studies aimed at examination of the spillway capacity of the options and identification of flow impacts and consequences downstream the dam, Figure 18.6.

Figure 18.6 Model of Stornorrfors, scale 1.100

Limited by the large area to be included in the study, the model was built in a scale of 1:100. The river some 1000 m upstream and 700 m downstream of the dam was included, resulting in a model of some 21 m in length and 10 m in width. The spillway options were built in and tested one after another according to a predetermined test program. The three options in the model are shown in figures below (Figures 18.7, 18.8 and 18.9).

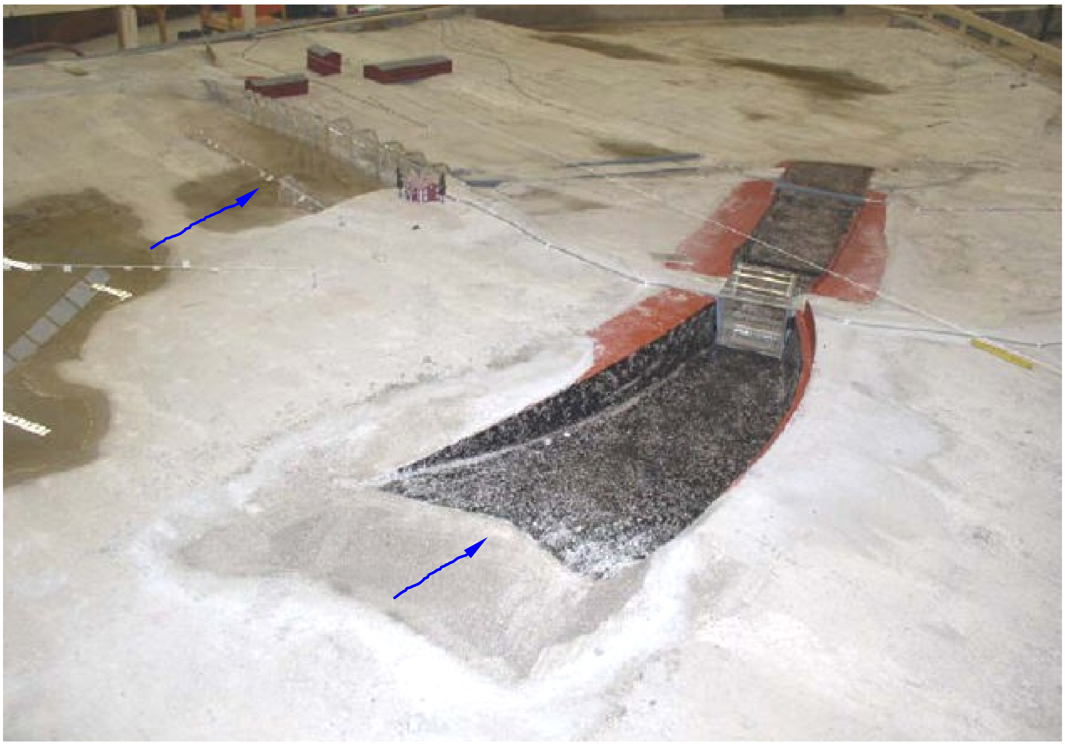


Figure 18.7 Option 1 – new spillway excavated in Tvärön

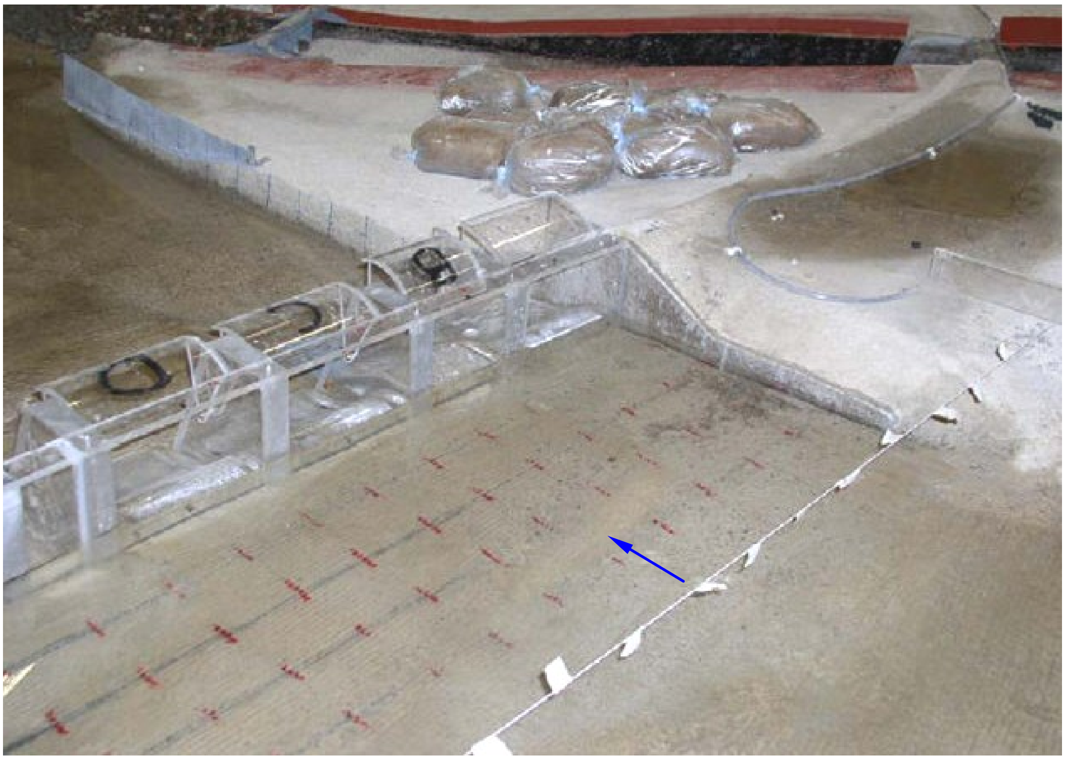


Figure 18.8 Option 2 – rebuilding of sealed reserve spillway

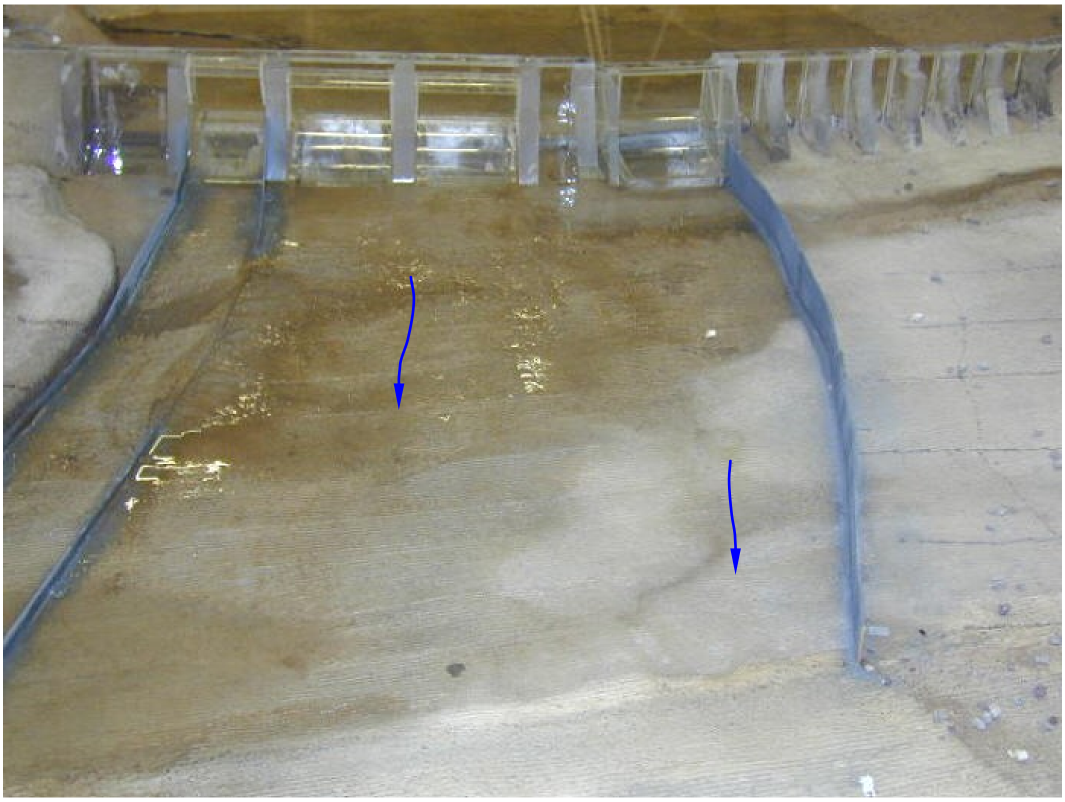


Figure 18.9 Option 3 – new spillway by removing buttress sections

With the help of pigment and light polystyrene balls, flow patterns at the design flood, both up- and downstream, was visualized with photography and videotaping. Downstream water stage and wave activities along the riverbanks were mapped. The need for a training wall, in Option 2 and 3, was examined, its configuration, in terms of placing, length and height, was optimized.

Hydraulic model studies of these alternatives are made and evaluations are provided from the hydraulic point of view. The final choice of refurbishment for the dam, Option 2, is based on overall technical, environmental and economical evaluations.

18.2.1 Spillway discharge capacity

In all the three options, the spillway is designed to have a standard WES (Ogee) crest and is equipped with a radial gate. The test result of spillway capacity is summarized in Table 18.1. As far as discharge capacity is concerned, the three options are almost equal; the requirements to pass the new design flood can be met by all.

Table 18.1 Spillway capacity at FRRL of three rehabilitation options

Rehabilitation option	Single spillway capacity (m^3/s)
1	1200
2	1150
3	1200

Abutting the right river bank and at a distance of about 50 m upstream of the existing dam, there is a part of an old concrete dam, built probably before 1920–30. It is 50 – 55 m long and its crest lies 4 – 5 m below the FRRL. Model tests show that if the old dam is removed, the spillway capacity can be increased by 100 – 150 m^3/s, depending upon the options.

18.2.2 Upstream consequences

Concerning the flow pattern in the upstream area, there are some differences among the options. For the spillway through Tvärön (Option 1), it takes water directly from the main river; the flow field upstream of the dam is similar to the existing. In the other two cases, all the water has to pass the narrow section between the main river and the dam, where a larger difference in the water level is present.

Figure 18.10 shows the distribution of surface-water flow velocity at the flow discharge ~1000 m^3/s in the river (Kiviloog et al. 2003). Controlled by the narrow river bend further upstream, denoted as M-M, the main flow tends to follow the left bank after it. Driven by this, a large zone of circulation is formed in the river. The model tests point to roughly the same flow pattern up to the spillway discharge around the design flood.

Figure 18.11 shows the flow velocity profiles measured at three cross-sections, running parallel to the dam and at a distance of 670, 430 and 200 m from it. Irrespective of which rehabilitation option to choose, it is always the same reach of the left riverbank that is subjected to high flow velocity and risk for erosion. Erosion protection should be constructed to protect the bank against extreme flood events.

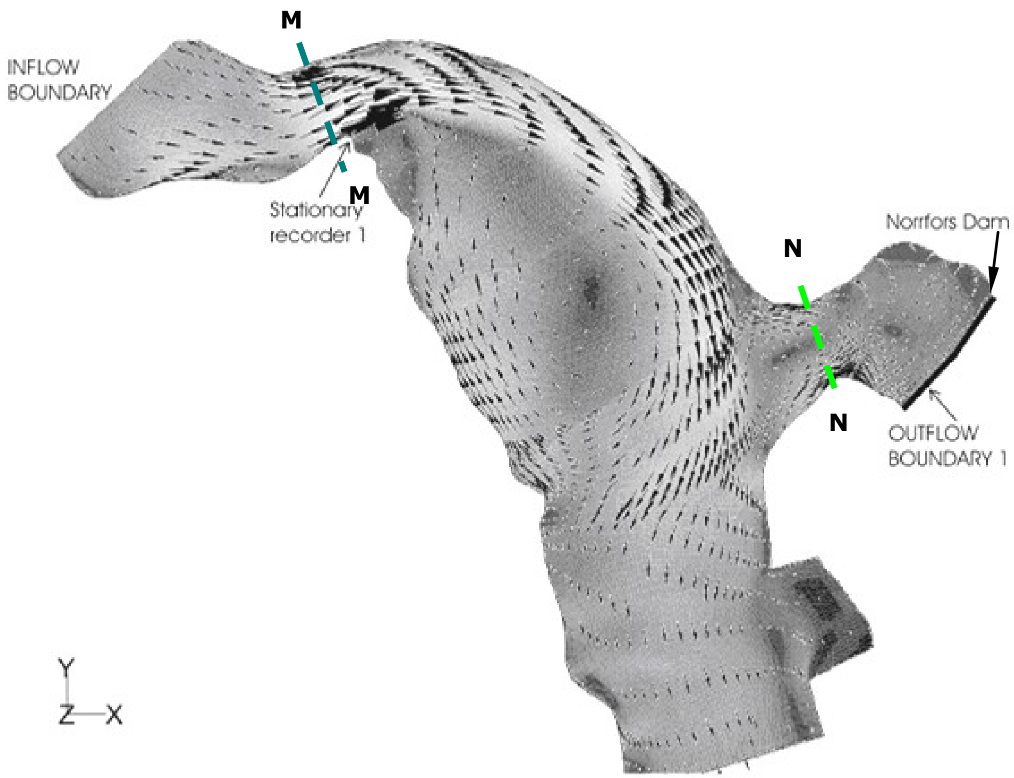


Figure 18.10 Flow pattern in the reservoir - surface-water flow velocity at river discharge ~1000 m^3/s.

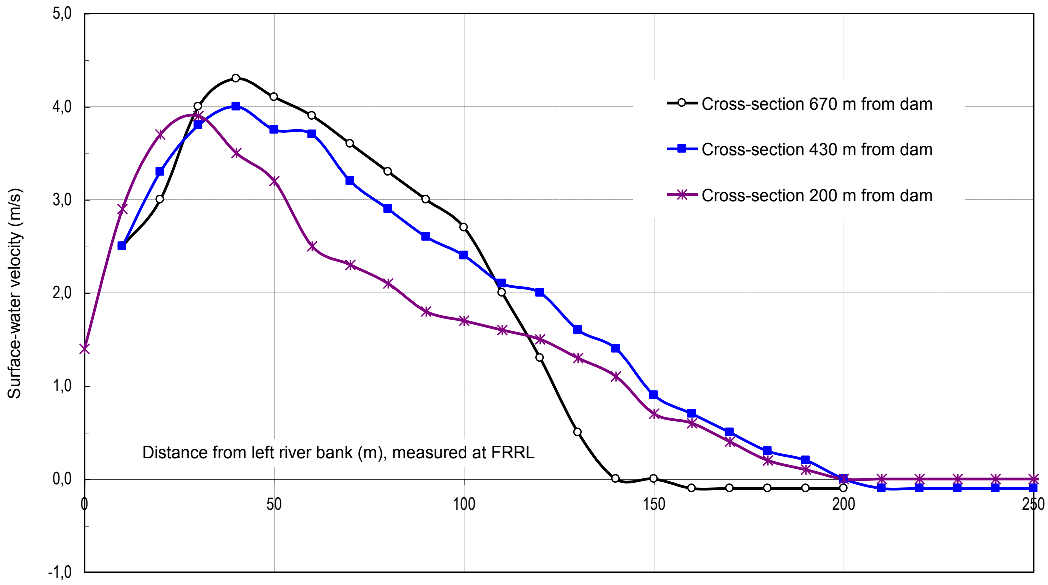


Figure 18.11 Flow velocity profiles at sections 670, 430 and 200 m from the dam, at design flood (measured in Option 1)

18.2.3 Downstream consequences

A moderate slope in cross-section all the way from left to right characterizes the terrain of the river course downstream of the dam. Conditioned by this, a predominant portion of water from the spillway, whatever the options, flows in the right half of the river. Due to the presence of huge stone blocks, "white water" with spray is expected.

Left river bank – Model tests show that, no matter which option one chooses for rehabilitation, shallow water and mild wave activities prevail along the bank at spillway discharges up to the design flood. In the buttress dam option, the new spillway is situated to the left of the existing ones and a training wall is required to prevent the fish hatchery from being flooded. In the other two options, the spillway flow behaves in a way not much different from today's situation, i.e. discharge from the existing three openings.

Right river bank – The concentrated water current occurs with high flow velocity in the right half of the river and this is accompanied by strong pulsating waves on the right bank. It seems that the mean water stage at the design flood is only insignificantly higher than that in normal flood situations, e.g. 2400 m^3/s. The wave motions are, however, much stronger.

There is a place for visitors on the right bank, located some 400 m from the dam. In all the options, the waves reach roughly the same elevation. There exists a risk that this place is overtopped by the waves at the design flood. In relation to mean water stage, the highest wave run-up occurs in Option 1, while the lowest in Option 3. No matter which option one adopts, the place should be closed for visitors during extreme floods.

In the reserve spillway option, there is a need to build a training wall downstream after the excavation in the right bank.

18.2.4 Evaluations of rehabilitation options

Option 1 is placed in the terrain to the right of the dam. The new spillway can be operated separately from the existing one. As excavated in the canal, the option gives rise to better approaching flow conditions, its discharge coefficient is 3 – 4% higher than that in the other two options. In terms of flexibility, construction method, etc, its construction is less complicated. Its disadvantage is that it imposes the largest influence on the environment – the whole canal, about 300 m in length, needs to be excavated.

The cost for Option 1 is estimated at US$8.5 million, compared with US$5.5 million for Option 2 and US$5.0 million for Option 3. The original idea is to sell the excavated material from the canal to highway construction, thus giving a cost reduction by ~US$3.0 million. Later laboratory tests show, however, that the material contains certain minerals that are unsuitable for the purpose. In view of its high costs, Option 1 is not longer attractive after this information was found.

As far as spillway discharge capacity is concerned, Option 2 and 3 are two roughly equivalent solutions. They are of the same level of cost. In either option, the construction must be conducted behind some kind of temporary closure; and a training wall needs to be built, on the right bank in Option 2 and on the left in Option 3. As located close to the left river bank, Option 3 gives simpler hydraulics – the water from the spillway flows straight on downstream and does not interfere much with the water from the other spillway openings. As compared with the existing conditions, it imposes the least influence on fish migration in the downstream area.

The following publications touch upon Stornorrfors.

Yang, J & Amnell, G. (2004), Measures for increasing spillway capacity at Stornorrfors – hydraulic model studies (*Åtgärder för ökad avbördning i Stornorrfors – hydrauliska modellförsök*), Report No. U 04:03.

Amnell, G & Yang, J (2005), Stornorrfors spillway – determination of spillway capacity by model tests *(Utskov i Stornorrfors - bestämning av avbördningsförmåga genom modellförsök),* Report No. U 05:01.

Yang, J & Cederström, M (2005), "Upgrading of Stornorrfors Dam for Safe Release of Extreme Floods", WaterPower XIV, July 2005, Austin, Texas.

19 Experiences and issues of attention

Safety issues and hydraulic concerns that arise from the rebuilding and upgrade of a dam often include the following aspects.

- Erosion in dam slope near spillway
- Discharge capacity
- Spillway modifications
- Construction of new spillway
- Spillway channel reshaping & rebuilding
- Aeration and air entrainment
- Enlargement of existing stilling basin/plunge pool
- New stilling basin/plunge pool
- Erosion in dam toes
- Erosion in river banks downstream

19.1 Dam slope erosion

The dam slope close to the spillway is prone to erosion due to high flow velocity and high turbulence level caused by spillway flood release. Erosion is often evidenced in operation, as is the case for Stenkullafors, Figure 19.1.

Figure 19.1 Erosion in dam slope near spillway

Many dam slopes are protected with rip-raps, arranged or unarranged. The risk of erosion should be examined in model tests, especially if the design flood is higher than before. Figure 19.2 and 19.3 show the measured results of mean flow velocity for Letsi and Stenkullafors. One can see that the near slope velocity may vary from one dam to another.

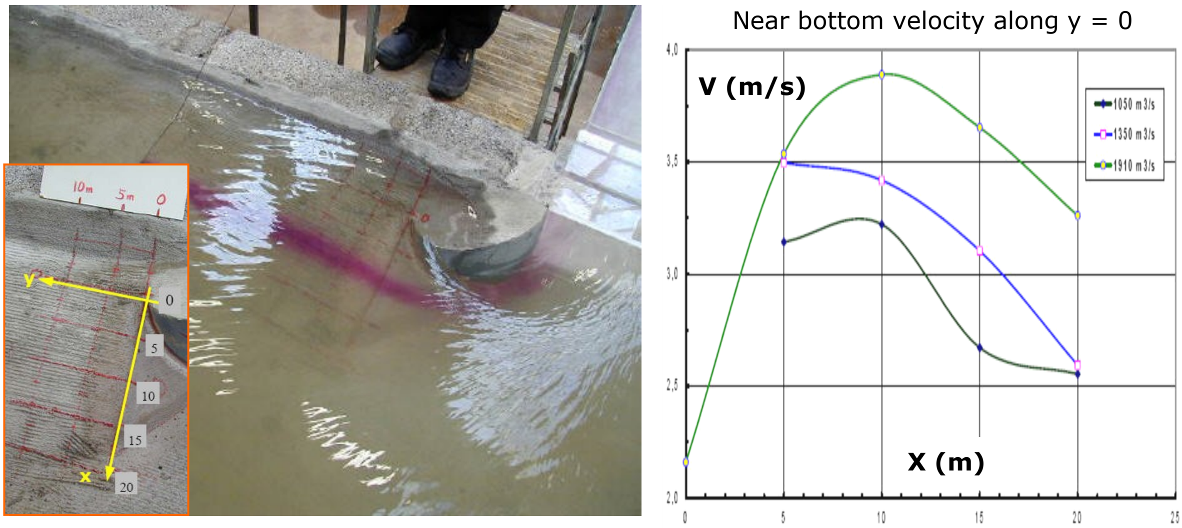


Figure 19.2 Letsi - near-bottom velocity on left dam slope

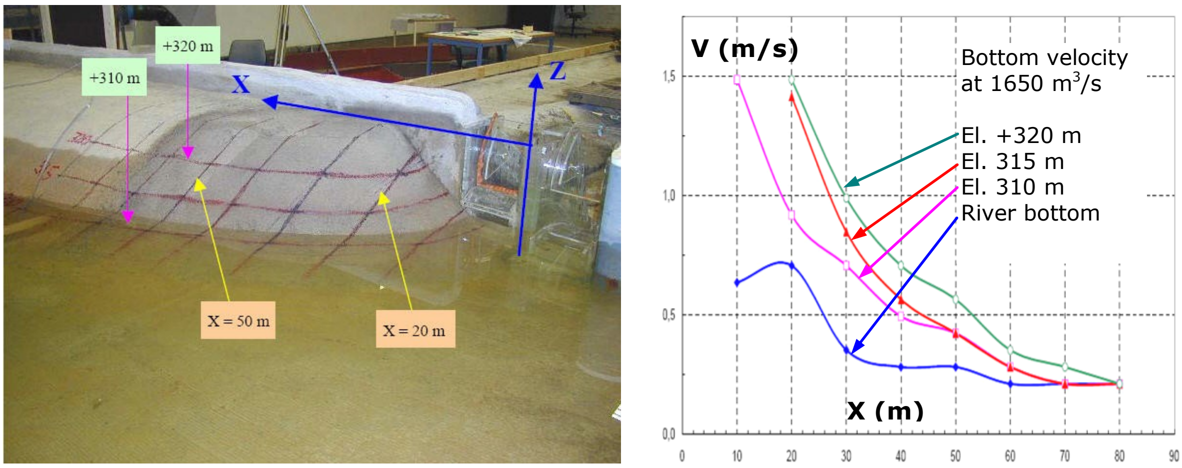


Figure 19.3 Stenkullafors - near-bottom velocity on left dam slope

Close to the spillway, the rip-rap design should have a higher safety factor (as compared with the rest of the upstream slope). For deep reservoirs and even bottom outlets, the high turbulence level at the spillway intake must be taken into consideration. There might be needs to heighten or extend the existing guide-walls or construct new ones. In some cases, even the use of rip-raps as slope protection close to the spillway is questioned. Maybe concrete lining or mat should be adopted, as is the case for Letsi, Figure 19.4.

Figure 19.4 Letsi - use of concrete lining on left dam slope close to spillway

19.2 Discharge capacity

Determination of the discharge capacity of a spillway belongs to conventional yet critical tests in a model study. Its accuracy affects the whole project.

It is important that the spillway, including guide-walls and dam slopes, is built with high accuracy and magnetic flow meters are used when possible. Even the terrain immediately upstream of the spillway plays a role in this context. Besides correct approaching flow conditions, a reach of the terrain 6 – 8 times the spillway size should also be modeled accurately.

In some models, it has been found that the present model studies give somewhat higher spillway discharge capacity than previously conducted tests. Reasons are sought but no persuasive arguments can be found. Sharp-edged weirs, either contracted rectangular or triangular, were probably used for flow measurements in model tests 30–40 years ago and they usually have lower accuracy than magnetic flow meters, with an accuracy around ± (1- 3)%. When doubts arise, to use double magnetic flow meter is one way to clarify the issue.

The discharge coefficient, as function of either head or discharge, is usually plotted to check the quality of the measurements. The flow discharge, Q (m^3/s), through an un-gated spillway opening can be written as

$$Q = CBH^{1.5}$$

where C = discharge coefficient, B = opening width (m) and H = water head (m) above threshold elevation. Depending upon the head, the normal interval of coefficient C is C = 1.70–2.00. Flatter spillway crest profiles are often characterized by lower C-values. Some examples are given in Table 19.1.

Table 19.1 Examples of spillway discharge coefficient

<table>
<tr><th colspan="2">Spillway</th><th>C</th></tr>
<tr><td colspan="2">Bergeforsen</td><td>1.90 – 2.02</td></tr>
<tr><td colspan="2">Gallejaur</td><td>1.75 – 1.85</td></tr>
<tr><td colspan="2">Laxede</td><td>1.73 – 1.90</td></tr>
<tr><td rowspan="2">Stenkullafors</td><td>Left</td><td>1.88 – 2.03</td></tr>
<tr><td>Right (after sill lowering)</td><td>1.80 – 1.90</td></tr>
</table>

19.3 Spillway modifications

To modify the spillway of a dam is often one way to cope with the problem of insufficient discharge capacity. Depending upon the design-flood magnitude and rebuilding costs, spillway modification can be made separately or in combination with dam heightening.

To change a bottom outlet to the overflow type of gated spillway by removing its parapet wall is an effective way to increase the discharge capacity. In an overflow spillway, the discharge is directly proportional to $H^{3/2}$, compared with to $H^{1/2}$ in a bottom outlet. The opening width remains usually the same, a higher gate is however required. As is the case for Ajaure, the requisite raise in the dam height is reduced by nearly 4 m.

In the case of several spillway openings, to modify only one opening is often the most inexpensive measure. Cost savings rest mainly with construction and gate replacement. When deciding which opening to rebuild, one should take into consideration the impact (energy dissipation) immediately downstream, so that favorable flow patterns are achieved. Construction limitations should also be weighed in when the choice is made.

Many of the existing spillway crests are designed in the light of the standard WES shape, having an optimum hydraulic behavior as far as the discharge capacity is concerned. To reduce the volume of chipped concrete, the crest shape is usually given a flatter profile after sill lowering, with somewhat deteriorated discharge coefficient.

There is always potential to optimize the crest shape when an existing sill of an overflow spillway is lowered. A favorable shape reduces the requisite gate height and releases the same amount of flood as a less favorable shape with a lower sill elevation and lower discharge coefficient. The distance from the sill to the river bottom plays a role in choosing the profile. A larger distance implies larger potential to find a favorable solution, as is the case with Stenkullafors. If the sill is very close the river bottom, sill lowering contributes to only little increase in discharge capacity and has therefore little effect on lowering the reservoir level, as in the case for Midskog, Figure 19.5.

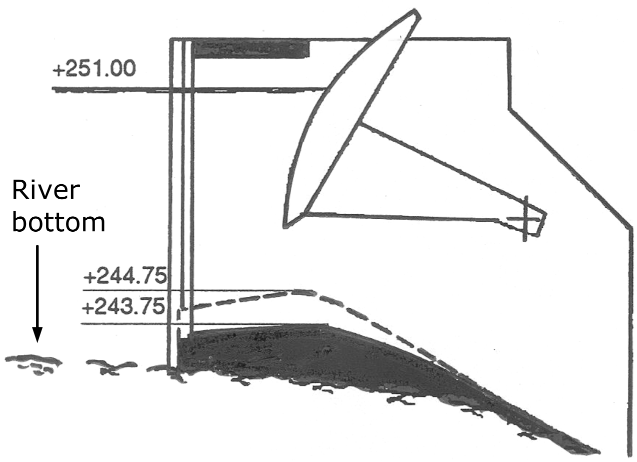


Figure 19.5 Midskog – tested spillway sill lowering that resulted in limited increase in discharge

Spillway modification is often a trade-off between costs and hydraulics. How an existing spillway is rebuilt should take into account other aspects in a refurbishment project, such as costs for raising the dam and flood risk during construction. In many cases, hydraulic model tests need to be made, the purpose of which is to optimize the crest modification and examine the consequences immediately downstream. As to crest profile optimization, even CFD can be a choice.

In Ajaure, the left bottom outlet was rebuilt to a gated overflow opening. In Stenkullafors, the sill elevation in the right gated overflow opening was lowered by 4.3 m. In Halvfari, the sealed space below the existing spillway will be rebuilt to a bottom outlet.

19.4 New spillway

To add a new spillway is often one of the proposals raised in an rebuilding project. Construction of a new spillway takes place often with the possibility of using the existing one, which reduces the risk of handling floods during the construction.

With a new spillway, the energy dissipation and flow patterns downstream need often be examined. A new tunnel spillway was proposed in Höljes. A separate spillway was before proposed for Bergeforsen but abandoned due to cost reasons. However, recent discussions as to potential measures for the dam have again pointed to an addition of a new spillway when the dam is to be built a second time.

19.5 Spillway channel modifications

Most of the existing spillway channels were designed for floods lower than the updated design floods. Besides, many of them are, to a varying degree, curved in plan, which causes complications due to the increased flood magnitude. To re-shape and optimize the channels, physical modeling is a must.

In Harsprånget, the channel was re-shaped, through excavation in rock, in such a way that the water became reasonably distributed in cross-section in the channel and directed away from the dam toe, thus reducing the impact on the dam during flood discharge.

In Ligga, a stepped cross-sectional profile was worked out, preventing too much water from running straight forward into the forest. At the same time, the impact on the dam toe close to the spillway and on the weak rock on the right channel side was mitigated.

In Höljes, the existing channel was widened by some 100% to the right. If the channel was given the same cross-sectional elevation, the flow would be strongly uneven, with cross waves and wave reflections propagating further downstream. The channel was therefore shaped with an elevation difference in the middle and a 2 m high partition wall along it. Besides, both the existing and the new spillway piers were extended and streamlined to avoid their introduction of disturbance into the flow.

In Letsi, there was already a differentiated bottom in the channel. Due to the extra water, a partition wall had to be added and optimized to obtain reasonable flow distributions in cross section. The channel sidewalls were also raised to prevent overtopping, Figure 19.6.

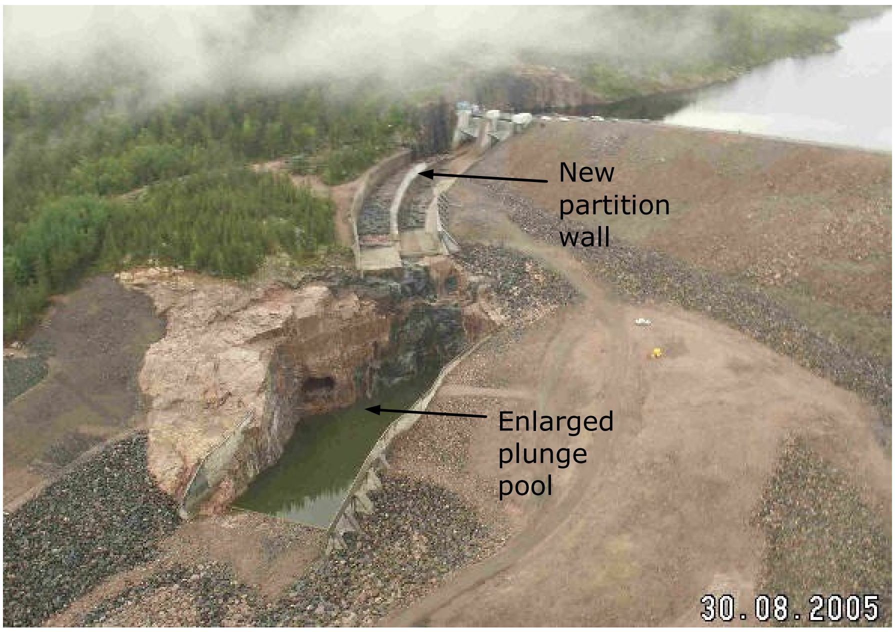


Figure 19.6 Letsi - spillway channel and plunge pool after rebuilding (taken before the test flood release on Aug 30, 2005)

Some spillway channels are surrounded, either partially or totally, by weak rock sensitive to erosion. Risk for erosion cannot be modeled according to scale in a physical model; computer analyses are usually made. However, physical model tests can often provide the input data, e.g. water depth, flow velocity and dynamic water pressure, needed for the numerical modeling. In connection to model tests, rock erosion potential was calculated in 2D and evaluated for e.g. Midskog and Ligga.

19.6 Air entrainment and air supply

The issue of air entrainment is in itself complicated, and is closely connected to high flow velocity and turbulence intensity. In a hydraulic scale model, its effect is quite limited. The entrained air causes the surface water to swell, becoming so-called "white water". This requires in turn higher sidewalls and is actually both an economical and safety concerns.

There are empirical formulas that can be used to estimate the effect of air entrainment in (spillway) channels open to atmosphere. Safety margins should be large enough to avoid overtopping of walls. In Ajaure, the channel walls were first raised after the model tests. However, very little safety margins were adopted by the designer, which led to overtopping at several locations during a test discharge. The sidewalls had to be heightened a second time.

One should not either forget that air entrainment can also be caused by irregularities or singularities in the waterway, such as axels of spillway gates under water surface, abrupt changes in geometry, etc.

Downstream of bottom outlets and in tunnels, model tests can often indicate the need of air supply to the flow to avoid cavitation, vibration, etc, as in the case for the Storfinnfors dam. In this regard, CFD modeling can be used to predict the requisite amount of air supply, allowing one to then devise proper engineering countermeasures.

19.7 New stilling basin/plunge pool

Due to the extra amount of water to be released, environmental concerns or the deterioration in geological conditions downstream, construction of a new stilling basin or plunge pool can be needed.

In Midskog, a new plunge pool was tested and constructed to mitigate the risk of rock erosion during spillway discharge. In Satisjaure, a new stilling basin was excavated to obtain better energy dissipation and reduce the velocity impacts downstream.

In Gallejaur, tests were made of a new stilling basin. However, due to the canal layout (improper canal alignment), the basin did not give any improvement in the energy dissipation and the idea was therefore abandoned.

19.8 Enlarging stilling basin/plunge pool

Some of the existing stilling basin/plunge pools, designed and constructed before for lower spillway discharges, were insufficient to dissipate the new design floods, which was found either in the model tests or in many cases already by the designer before the model tests. To enlarge them became therefore a fact.

The enlargement can be implemented in several ways. In Letsi, the plunge pool was made much larger in plan and also a few meters deeper. In Porsi, the stilling basin recommended was some 50 m longer and 1.5 m deeper. In Höljes, even baffle blocks were used to reduce the basin volume required at the design flood.

19.9 Erosion in dam toes

The concern for erosion in a dam toe, in many cases well grounded, is mainly due to two causes.

- Direct erosion – spillway discharge causes high flow velocity along or in the close vicinity of the dam toe, probably also accompanied by strong waves. The effect of only wave motions is usually not a severe problem, if the dam toe is protected by e.g. rip-raps.
- Flow circulations – as a result of inadequate energy dissipation, the riverbank can be gradually eroded. Driven by the main flow current from the energy dissipator, backward flow builds up along the eroded bank. If erosion further develops, a flow circulation zone is usually formed. The circulating velocity increases with time, with the risk of progressive erosion towards the dam toe.

  The rock-bed elevation immediately downstream of the energy dissipator and in its vicinity is an important factor in erosion studies and should be identified.

  An energy dissipator is usually surrounded, on both sides, by concrete sidewalls. The walls, if high enough and kept in place, make up a limiting factor in the erosion development, implying that the erosion is then confined. However, if part of the wall(s) or the whole wall loses its stability and collapses, the circulation zone would be getting larger, leading to aggravating backward erosion in the dam toe.

  Possible instability of stilling basin sidewalls should be considered as one of the failure modes in a dam-safety evaluation.

The risk of erosion in the dam toe in e.g. Ligga and Harsprånget is due to direct erosion. The risk is avoided by re-directing the spillway water away from the toes.

The erosion potential in e.g. Porsi, Halvfari and Stenkullafors is a consequence of back flow circulations. In Porsi, a cutoff wall, in the form of sheet piles, was, for this purpose, constructed to the right of the stilling basin.

In the dam toe on the left side of the stilling basin, the designer in Långbjörn suspected erosion due to large circulation zones with wave activities. However, performed model tests showed that the circulating zone was non-existent and the proposed extension of the left sidewall, which was very costly, was not necessary.

19.10 Erosion downstream

Concerning erosion downstream in a channel or river course, model tests provide a designer with necessary data of water depth, flow velocity and wave motions. To withstand the design flood, erosion protection needs often to be strengthened.

In Stenkullafors, it was discussed that the existing rip-raps along a long reach of the spillway channel would be grouted together with cement to form a rigid body. To avoid building up water pressure beneath, a matrix of drainage holes must be arranged. In Letsi, the canal downstream was new, constructed from the beginning with erosion protection in the form of rip-raps of sufficient stone dimensions. In Halvfari, the existing rip-raps in the right bank would be partially replaced.

Even in the ongoing dam rebuilding projects, there is a questionable practice of using or accepting rip-raps as erosion protection immediately downstream of the energy dissipator. If improper constructed, the transition from the concrete sidewalls to the ripraps is a weak link in this context. But the main issue is that the rip-raps are usually designed in the light of empirical formulas based on the flow velocity only, a design procedure suitable for long, uniform channels and canals. However, strong wave motions usually prevail immediately downstream of the dissipator, the effect of which is often ignored by the designer, thus leading to locally under-dimensioned rip-raps. Damages have been evidenced in a number of spillway channels, in some cases even at low spillway discharges.

20 Other model studies

There have been hydraulic model studies at Vattenfall R&D that were not directly related to dam-safety refurbishments, but to other hydraulic issues of either existing or new dams. A partial list is given below for those who are interested.

Porjus (Vattenfall), Figure 20.1

Yang, J. (1998), Porjus – physical model tests with downstream flow patterns (*Porjus kraftstation – fysiska modellförsök med nedströmsvattenstånd*), Report No. US 98:19.

Yang, J. (1998), Porjus – additional tests with downstream flow patterns (*Porjus kraftstation – fortsatta modellförsök med nedströmsvattenstånd*), Report No. US 98:23.

Ljunga (Fortum), Figure 20.2

Yang, J. (1998), Ljunga power station – hydraulic model studies of intake channel and tower (*Ljunga kraftverk – hydraulisk modellstudie av intagskanal och intagstorn*), Report No. US 98:11.

Älvkarleby (Vattenfall), Figure 20.3

Yang, J. (2000), Älvkarleby power station – Study on sediment transport and its control through model tests, Report No. US 00:12.

Yang, J. (2001), Älvkarleby power plant – Sediment trapping and removal system for intake channel (*Älvkarleby Kraftverk – Sedimentfångnings- och utsugningssystem*), Report No. U 01:24.

Yang, J & Johansson, N, "Sediment Problem at Älvkarleby Power Station", WaterPower XIII, July 2003, Buffalo, NY.

Yang, J & Johansson, N., "Sediment Trapping and Removal System in Existing Waterways & its application", 9th Intl. Symp. on River Sedimentation, October 2004, Yichang.

Canjilones (Panama power owner)

Yang, J. (2001), Panama, Canjilones power station, penstock bifurcation – Model tests of head losses, Report No. U 01:46, 14 pages.

Yang, J. (2001), Panama, Canjilones power station, penstock bifurcation, 2nd Design – Model tests of head losses, Report No. U 01:55.

Vatnsfell (Iceland State Power Board), Figure 20.4

Yang, J. (1999), Vatnsfell Hydropower Project, Iceland - Hydraulic model studies of flood discharge structures, Report No. US 99:8.

Yang, J. (2002), Bottom outlet at Vatnsfell – problem with stilling basin and possible countermeasures, Report No. U 02:04.

Yang, J (2002), Vatnsfell Dam – Countermeasures to improve flow behaviors in stilling basin – model studies, Report No. U 02:39.

Yang, J, "Stilling Basin at Vatnsfell Dam, Iceland - Problem of Water Cascade and its Solution", HYDRO 2005, Oct 2005, Villach, Austria.

Yang, J & Stefánsson, F, "Vatnsfell dam, Iceland – experimental study of flood discharge structures", HYDRO 2006, Sept. 2006, Porto Carras, Greece.

Yang, J, "Investigations at Vatnsfell", Journal of International Water Power and Dam Construction, Vol. 59, No. 9, 2007.

Figure 20.1 Model of Porjus dam and downstream

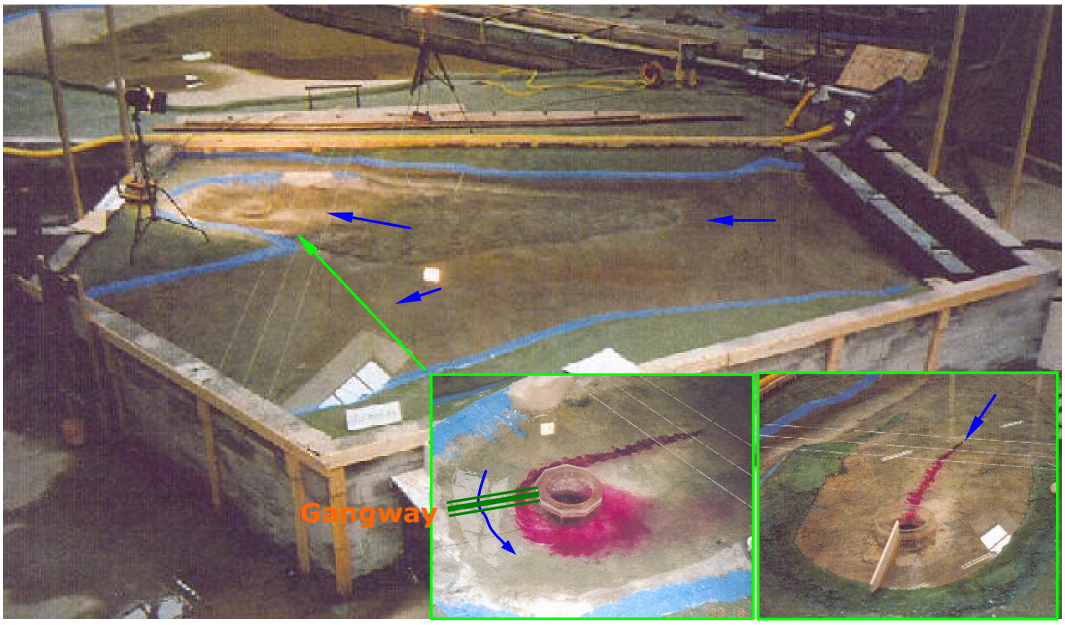


Figure 20.2 Model of Ljunga power plant intake shaft and tower

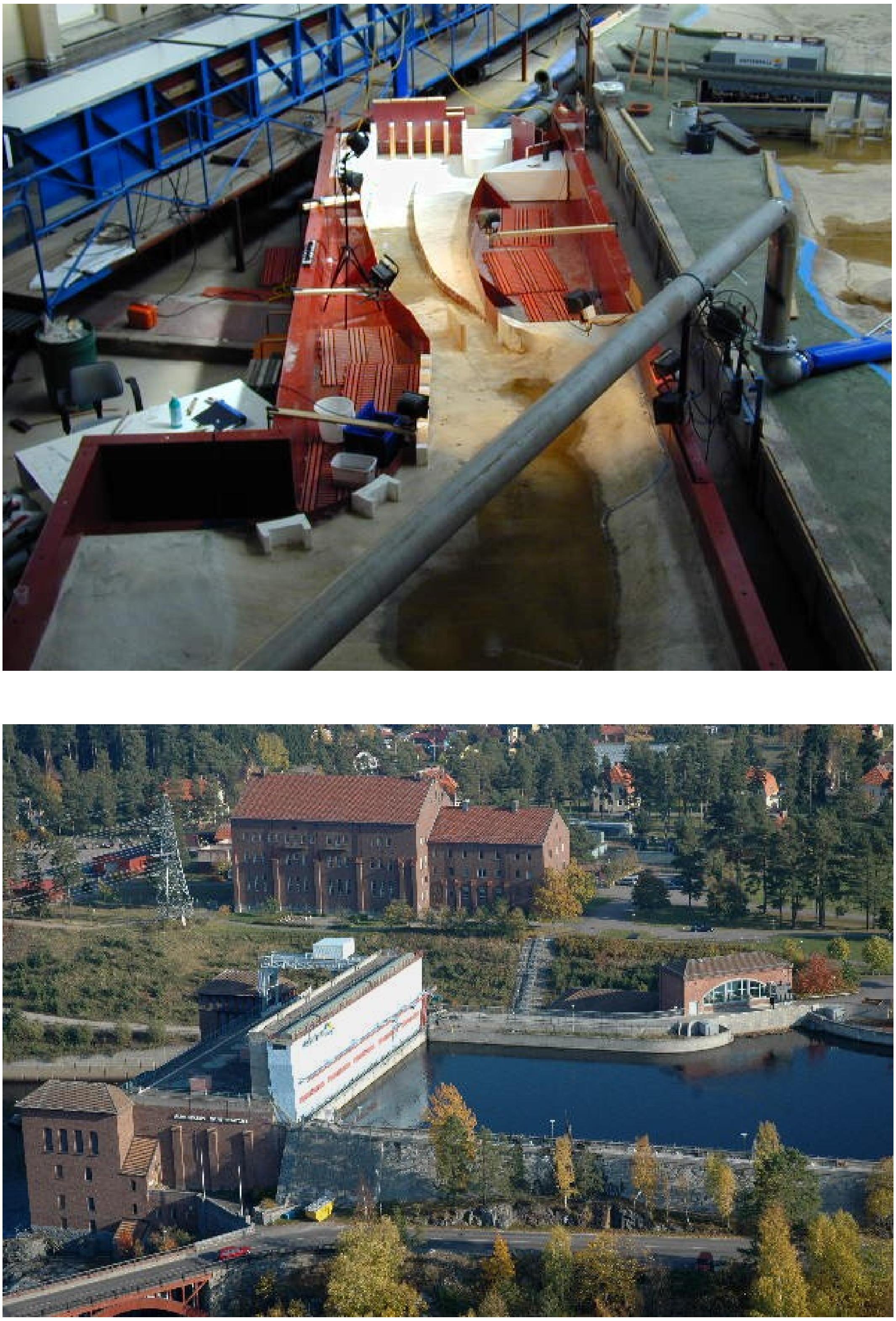


Figure 20.3 Model of Älvkarleby intake canal and sediment transport (photo below by Alf Linderheim)

Figure 20.4 Model (above) and prototype (below) of Vatnsfell flood discharge structures

21 Concluding remarks

Without making any effort of covering everything, a few remarks are made below to conclude the report.

21.1 Model construction

To provide the rebuilding of a dam with reliable basis, the physical model must be built with sufficient accuracy, especially the quality of spillway, spillway channel and stilling basin should be guaranteed.

Spillway openings have to be built with millimeter accuracy, as the reservoir level determination is directly affected. Certain dimensional deviations are inevitable when a model is produced. However, they should be corrected afterwards against the true dimensions when the discharge capacity is determined.

Quality control should be made at frequent intervals during the construction of the model, not only after its final assembly. Model components made in Plexiglas and sheet metal are difficult to modify afterwards. The model should be laser scanned for quality check upon completion and before testing, possible discrepancy is corrected.

The control point for model leveling should be kept throughout the model construction and testing, which is of vital importance. Mistakes were made before when the control point couldn't be located for positioning of spillway thresholds.

Many details in a model can be simplified without affecting modeling results. One should simplify things that can be simplified, as model costs can often be reduced and time saved.

21.2 Measurements

Depending on modeling purposes, conventional measurements in a model study may consist of following categories.

- Flow rate measurement
- Flow Velocity (profile) measurement (in 1D, 2D or 3D)
- Water depth/surface measurement
- Water pressure (force) measurement
- Wave motion measurement

Model tests can also involve measurements of air entrainment (air-water distribution in open channel flow) and air supply.

It is vital that all the measuring instruments and devices are calibrated regularly. The flow rate, for example, is often measured with magnetic flow meter, its accuracy affects directly the relationship between spillway discharge and reservoir water stage.

For large dam projects or projects of considerable economical concern, it is even advisable to carry out numerical modeling of spillway discharge capacity to double-check the physical test results.

21.3 Model testing

Physical model tests of any category are not standard laboratory testing like concrete sample compression or frost resistance, but research and development that require experiences and problem insight.

Successful model testing requires good teamwork between the client and the modeler. The client and its representative should actively participate in the tests. More minds are always better than one or two.

When time and money allows, the modeler should be given some freedom to make certain trial-and-error tests, for the client’s best. However, if the modeler was given and told to follow a menu of tests prescribed by a consultant, risk would, often in the long run, exist that some necessary tests might be missing, which was the case for Ajaure where the risk for erosion in the dam slope close to the spillway was not requested and therefore ignored.

The modeler should have a solid academic background, a good grasp of physical phenomena and actively accumulate experiences through projects. He or she should, through exchange or international activities, get acquainted with different issues, and very often details, of model tests, and know about how e.g. a similar test would be conducted by other hydraulic laboratories.

When doing model testing and interpreting measured results, it is vital that the modeler should have a sixth sense and understand scale effects, i.e. the differences between the model and the prototype. The ultimate aim of model tests is prototype design optimization. Being easily said, this requires, however, years of project experiences. The modeler should participate in prototype flood tests when possible, especially in connection with ongoing or planned model tests.

Finally, as at the end of most fairy tales, it should be said that the need and importance of model testing couldn’t be over-emphasized. One should also keep in mind that the modeling costs often constitute a negligible portion of the total costs in a dam rebuilding project or new hydro scheme. To correct an unsuitable design or a hidden mistake in a scale model always costs much less than in the prototype.

22 Other references

The following articles and publications touch in one way or another upon the dams examined in this report.

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