Final thesis

Feasibility study for stabilising the Plavinas dam, Latvia

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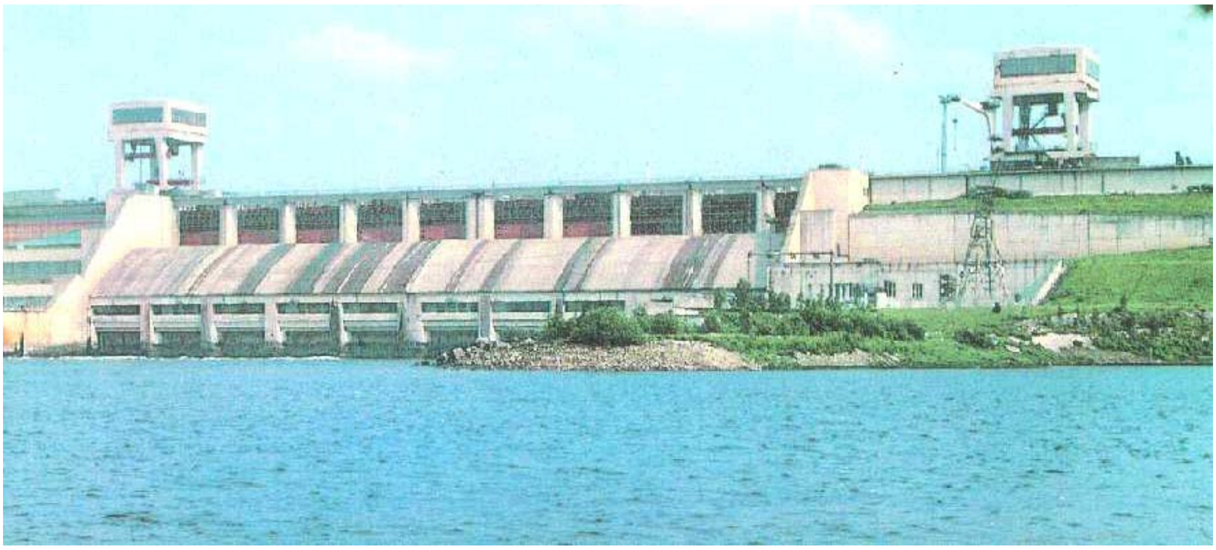

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PREFACE

The last part of the 5-year Master of Science course of Civil Engineering at Delft University of Technology, the Netherlands consists of a final thesis. This thesis is developed in co-operation with Witteveen+Bos.
The subject of the thesis is the Plavinas dam, Latvia, which is feared to become unstable. Providing reasonable solutions for stabilising the structure was the main objective of the project.
Many thanks to Witteveen+Bos for letting me perform this study at their office and providing me with information and feedback.

Verena Friedrich
Rotterdam, 5 June, 2002

SUMMARY

The stabilisation of the Plavinas dam, Latvia was the subject of this final thesis. The objective was to investigate the weaknesses of the Plavinas dam and to present possible stabilising treatments. This includes a rough estimate on costs and construction methods.

Plavinas hydroelectric power station is the most important dam in Latvia, supplying 30% of the country's power. The Plavinas blocks the runoff of the Daugava River, resulting in the formation of a reservoir. The power plant is a composite type structure consisting of a concrete power house and a spillway, which is located on top of the power house. The water from the reservoir can either flow through ten generating units located in the concrete power house, which is approximately 200 m long, or flow over the power house, the spillway.
The head difference between the reservoir and the tailrace measures 40 m. Several kilometres of hydraulically filled embankment dams extend to both sides of the concrete power house. The dam is founded on glacial till overlying a sandstone layer.

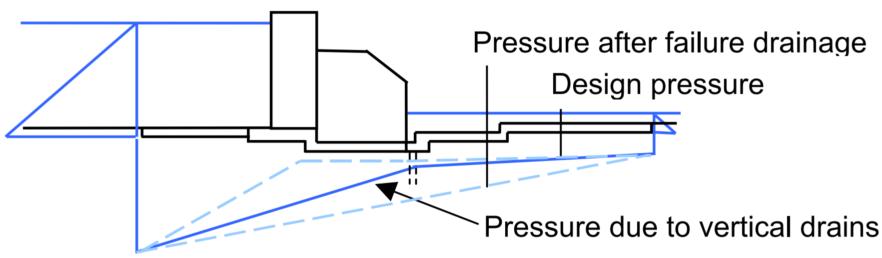
Drainage wells were incorporated in the foundation of the power house. This drainage system reduces the large uplift water pressures from the head difference between the reservoir level and the tailrace. This results in a larger effective weight of the structure, which is favourable for the bearing capacity of the structure against sliding and toppling. There are indications that the drainage wells, which can not be replaced under the concrete structure, do not function according to specifications:

- an increase of uplift water pressures is monitored. This could endanger the stability of the power house.
- transport of fine soil particles with the groundwater flow in the regional aquifer is occurring towards the drainage galleries near the right embankment as well as the drainage system underneath the structure.
- settlements of the power house have been observed, which seems to be caused by collapsing seepage channels.

The power house structure and the aprons were schematised and the normative loading cases were determined. The stability of the structure was calculated for the following situations:

- original design assumptions in which the uplift water pressure is reduced by approximately 90%
- rising water pressures due to continuously less effective original drainage system
- complete failure of drainage wells without compensating treatments
- different treatments, increasing the stability of the power house in case the original drainage fails entirely, either by adding more weight to compensate for the larger uplift water pressure or by reducing the uplift pressure by an extension of the seepage path or new drains.

The different stabilising treatments were analysed in a multi criteria analysis taking into account amongst others the problem elimination, durability and feasibility of execution.
The alternative of new drains in the downstream apron close to the power house provided the best alternative.



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LIST OF SYMBOLS

B = width of the foundation slab (m)
B_{ef} = effective width of the foundation (m)
C_c = compression index (-)
$c'_{e;d}$ = design value of the cohesion in (kPa)
e = void ratio (-)
e_B = eccentricity of the resulting vertical load (m)
$F_{s;v;d}$ = design value of the vertical load, perpendicular to the plane of the foundation (kN)
$F_{s;h;d}$ = design value of the horizontal load, in the plane of the foundation (kN)
g = gravity (m/s^2)
h = water depth (m)
ΔH = head difference (m)
H = horizontal load (kN)
i = gradient (m/m)
i_c = reduction factor for the angle of the load for the cohesion (-)
i_q = reduction factor for the angle of the load for the soil present next to the structure (-)
i_γ = reduction factor for the angle of the load for the effective specific weight of the soil under the structure (-)
k = coefficient of permeability (m/s)
L = length power house across valley (m)
L_{ef} = effective length of the foundation (m)
n = safety factor (-)
N_c = bearing capacity factor for the cohesion (-)
N_q = bearing capacity factor for the soil present next to the structure (-)
N_γ = bearing capacity factor for the effective specific weight of the soil under the structure (-)
s_c = shape factor for the cohesion (-)
s_q = shape factor for the soil present next to the structure (-)
s_γ = shape factor for the effective specific weight of the soil under the structure (-)
T = moment (kNm)
V = vertical load (kN)
w = moisture content (%)
w_P = plastic limit (%)
w_L = liquid limit (%)

δ = design value of the effective angle of internal friction between structure and soil (°)
ϕ'_d = design value for the effective angle of internal friction of the foundation soil (°)
ϕ' = representative value for the effective angle of internal friction of the foundation soil (°)
γ = total unit weight (kN/m^3)
γ_d = dry unit weight (kN/m^3)
γ'_d = design value of the effective specific weight of the soil under the structure (kN/m^3)
γ_s = unit weight of solid material (kN/m^3)
γ_{sat} = unit weight of material saturated with water (kN/m^3)
ρ = specific weight (kg/m^3)
$\sigma'_{max;d}$ = design value of the maximum stress on the effective foundation (kPa)
$\sigma'_{v;z;o;d}$ = design value of the original effective vertical stress in the foundation soil directly under the structure (kPa)

1 INTRODUCTION

The hydroelectric potential of the Daugava river is developed in a cascade of three hydropower plants which utilises the head from elevation 72 m down to sea level: Plavinas, Kegums and Riga. Plavinas Hydroelectric power station is the most important power plant in Latvia, supplying 30% of the country's electric energy. In terms of capacity this is the largest hydroelectric station in Latvia and it is considered to be the third level of the Daugavas hydroelectric cascade. The power house of 180 m length is combined with a spillway. The entire building complex is extremely compact. There are ten generating units installed at the hydroelectric station with a planned capacity of 825 MW. In 1996, the capacity of the plant was increased by 30 MW.

The Plavinas plant is situated at a distance of about 90 km from Latvia's capital city Riga and is unique in terms of its construction. For the first time in the history of hydropower construction practice, a hydroelectric power station was built on clay-sand and sand-clay foundations with a maximum water difference of 40 m. The foundation on this difficult soil is the origin of some small displacements of the structure and seepage through the abutment dam and underneath the structure. This thesis investigates methods for stabilising the Plavinas dam.

The chapters of this report treat the following subjects. Chapter 2 gives a project description, chapter 3 provides a short problem analysis. In chapter 4 a general risk analysis is carried out. The boundary conditions of the structure of the Plavinas dam including schematisation of the foundation soil, the power house structure and the different loads on the structure are described in chapter 5. A detailed analysis of the stability of the power house in case of the original drainage, rising water pressures and an entire failure of drainage is performed in chapter 6. Chapter 7 analyses different stabilising possibilities and in chapter 8, a selection of stabilising treatment is made. Chapter 9 provides some general information about the implementation of the selected treatment. The conclusions and recommendations are attached in chapter 10.

2 PROJECT DESCRIPTION

2.1 GENERAL[1 2]

Plavinas hydroelectric power station is the most important dam in Latvia, supplying 30% of the country's electric energy. In terms of capacity this is the largest hydroelectric station in Latvia. The Plavinas plant is located in close vicinity of the village Aizkraukle, about 90 km south-east of Riga, Latvia, see Figure 1 and Figure 2. The power plant is supplied by water of the Daugava river which is the largest river in Latvia having a catchment area of 88 000 km^2. The river has its headwaters in Russia and flows through Belarus before it enters into Latvia. The discharge[3] of the River varies seasonally from 95 m^3/s (winter low-flow) – 105 m^3/s (summer low-flow) to 8000 m^3/s (spring snowmelt period), while the mean annual discharge is ca. 600 m^3/s. Half of the runoff occurs in the months of snowmelt in March and April.

Figure 1: Map of Latvia

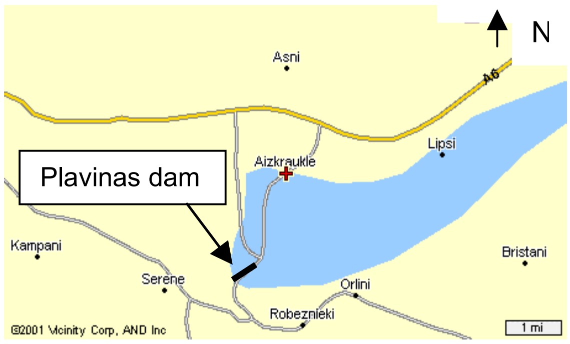


Figure 2: Map of region

The hydroelectric potential of the Daugava is developed in a cascade of three hydropower plants (abbreviated as HPP) which utilises the head from elevation 72 mASL (meters above sea level) down to the sea level: Plavians, Kegums and Riga, see Table 1. Plavinas is considered to be the third level of the Daugavas hydroelectric cascade. Kegums HPP, the second level of the cascade was built at 70 km distance from the firth of Daugava (where the Daugava River reaches the sea). Riga HPP is the newest of the Daugava hydroelectric stations. This is the first level of the cascade, 35 km distant from the firth of Daugava. In total, Latvia has 1517 MW of hydroelectric capacity, which is generated by the cascade.

Table 1: Daugava Hydropower Plants[4]

HPP	Headwater level mASL	Head m	Power Capacity MW	Year of Commissioning
Plavinas	72,0	40,0	6*82.5 +4*90=855	1965-66 1991-93
Kegums 1	32,0	14,0	4*17=68	1939
Kegums 2	32,0	14,0	3*64=192	1979
Riga	18,0	18,0	6*67=402	1974-76

The plants are all owned and operated by the State Joint Stock Company Latvenergo. The Plavinas power plant was designed and constructed by the Russian company Hydroproject.

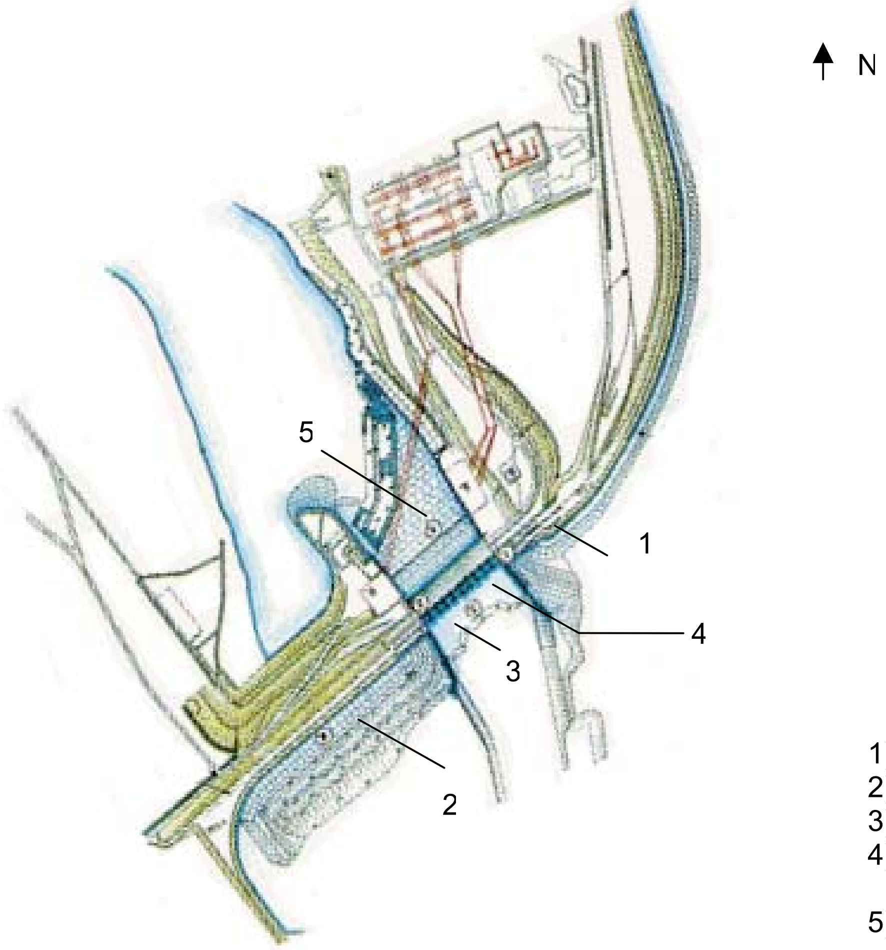
[1] Joint Stock company Latvenergo, *Tender Documents, Scope of Services*. Aizkraukle: April 2001
[2] http://www.energo.lv/en/latvenergo/3_2_15.php
[3] http://www.paic.lv/English/2_4.html
[4] Norwegian Geotechnical Institute, *Daugava River, Latvia Dam Safety Improvement Studies, 970009-3, Stability Evaluation of the Plavinas HPP, Latvia.* Oslo: October 1997

The construction started in the early 1960's and the power house was completed in the late 1960's. Since then, the plant has been used by Latvenergo.

2.2 DESCRIPTION OF PLAVINAS DAM[5]

2.2.1 Concrete Power house

The Plavinas dam blocks the runoff of the Daugava River by a concrete power house/spillway and a hydraulic fill embankment dam. This leads to the formation of a reservoir[6] with an area of 34,9 km^2 and a full capacity of 509 hm^3. Looking from the reservoir in downstream direction, the embankment dam on the right hand side will be termed ”right” embankment dam in this report.
The power house is of a rather unusual design in the sense that the spillway is located on top of the concrete power house and discharges over the power house. The water from the reservoir can either flow through the generating units located in the concrete power house, or flow over the spillway in case of a high water level. The total installed capacity of the power plant is 855 MW, generated by 10 generating units. Plavinas is the power plant in the cascade using the head from 72 mASL to 32 mASL. The prime dimensions of the structure are: length along the stream 65 m, length across the stream 183 m and height from toe to top of piers 58 m. A plan view can be found below in Figure 3 and Figure 4. A detailed plan view including contour lines is attached in Appendix 7.



1) right embankment
2) left embankment
3) upstream apron
4) concrete power house/spillway
5) downstream apron

Figure 3: Plan view of Plavinas dam

[5] Joint Stock company Latvenergo, *Tender Documents, Scope of Services*. Aizkraukle: April 2001
[6] http://www.fao.org/docrep/W6240E/w6240e13.htm

Figure 4: Location Plavinas dam near Aizkraukle[7]

By using the head difference between the reservoir and the tailrace water table, electric power is generated by ten turbines inside the power house. Ten pits for the turbine/generator sets and ten spillway gates are joined into two large rigid blocks, each about 90 m long. The blocks are separated by bitumen filled joints. Inside the concrete power house access

[7]http://www.adc.lv/aizkraukle/karte.htm

galleries can be found, see Appendix 1 and Appendix 6. At the upstream side the blocks are connected to caisson type retaining walls, downstream to L-shaped retaining walls. A cross section of the power house / spillway structure is shown in Appendix 1.

The concrete power house abuts on the embankment dam in a wing wall concrete structure. The embankment dams behind the wing walls support the power house/spillway structure by the presence of friction forces between the earth fill and the concrete surface. This provides a larger bearing capacity against the large horizontal water pressure from the reservoir.

2.2.2 Foundation of Plavinas Power house

The power house blocks are founded on drainage layers placed on an excavation in glacial till (loam and sandy loam) across the pre-glacial valley of an estimated depth of 80 m.
From upstream to downstream, there are three shallow cut-off trenches reaching below the base of the foundation slab.
The rather unusual hydro-geological situation makes the stability of the power house structure a critical issue. The designer has incorporated a complex system of drainage blankets and wells to control the pressure under the power house, see Figure 5. The stability of the structure is thus dependent on the effective drainage underneath to keep uplift pressures within the design assumptions. Outside the power house, pressure control is achieved by a large number of relief wells on the right bank (north).

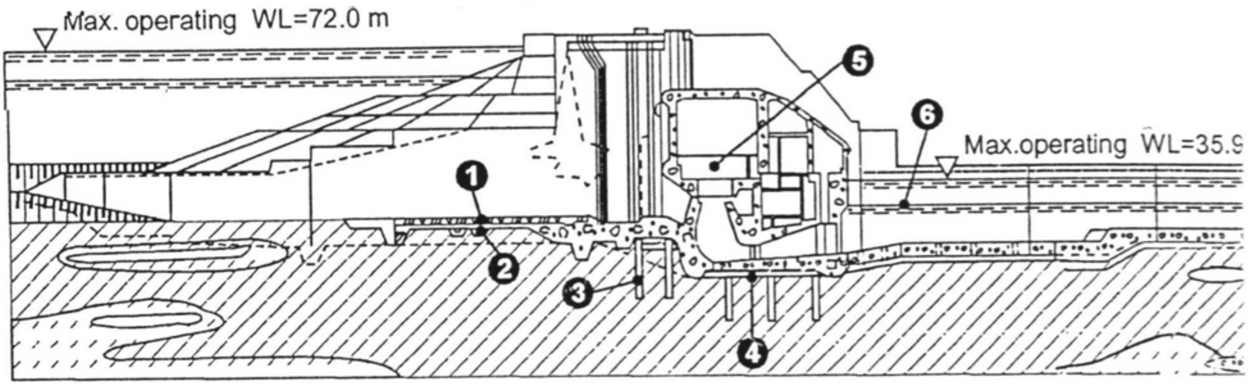


Figure 5: Cross section of Plavinas dam including embankment dam and drainage system

1) Upstream apron
2) Drainage blanket under upstream apron
3) Upper line of power plant drainage wells
4) Lower line of power plant drainage wells
5) Power plant
6) Stilling basin

There is an upstream apron slab placed on the glacial till and a downstream apron cast on drainage layers. The upstream apron reduces the uplift pressure under the power plant by lengthening the seepage path and prevents erosion due to acceleration of the water towards the turbine inlet. The downstream apron offers bed protection in the downstream area and provides a stilling basin, where the water can slow down due to turbulence.

The uplift pressures underneath the foundation slab of the plant are controlled by an extensive system of cut-off and drainage measures:

- A concrete cut-off apron upstream of the foundation slab. The apron is combined with an underdrain and two groups of drainage wells under the right bank retaining walls.

- An upstream drainage blanket combined with 10 m deep drainage wells divided into ten isolated segments. The drainage water was originally discharged into the tailwater, but is now discharged into a control tank defining the exit head at 32 mASL.
- A downstream drainage blanket combined with 10 m deep drainage wells within the limits of the draft tubes. The drainage water is discharged into the tailwater.
- A concrete tailwater apron covering the stilling basin. The apron is combined with an underdrain discharging through weep holes at the end of the downstream apron (inverted filter).

2.2.3 Embankment dams

Several kilometres of hydraulically filled (consisting of loamy sand of the alluvium in the river) embankment dams extend on both sides of the power house. The interior of the embankment dams is assumed to be relatively impervious. A slope protection was applied at the reservoir slope of the embankment dams. Figure 3 in section 2.2.1 shows a layer on the embankment dams, which continues to the end of the upstream apron. Therefore it is assumed that the function of this layer is not only to protect the reservoir slope, but also to provide an impervious layer connected to the upstream apron. This prolongs the seepage path and introduces the large uplift water pressure at the upstream end of the upstream apron, not under the power house directly. The embankment dams are provided with grout curtains[8], which presumably reach into the dolomite to provide an impermeable screen.
Outlines of the embankment dam are attached in Appendix 4 and Appendix 5. They were created by the use of the topographic maps from Appendix 7 and Appendix 8.
The embankment dams on either side of the power house/spillway are built on rock foundation (weakly cemented sandstone with sand and interlayers of clay).
The water pressure in the underlying aquifer in the Amata and Gauja formations is controlled on the right bank by relief wells, combined in two drainage galleries as well as isolated deep relief wells.

[8]Norwegian Geotechnical Institute, *Daugava River, Latvia: Dam Safety Improvement Studies, 970009-3, Stability Evaluation of the Plavinas HPP*, Latvia. Oslo: October 1997

3 PROBLEM ANALYSIS

3.1 OBSERVED PROBLEMS AT THE PLAVINAS DAM

At Plavinas, several problems have been observed. These are described below.

3.1.1 Drainage system under the power house

The power house is dependent on an underlying drainage system to reduce large uplift water pressures caused by the large head difference between the reservoir and the tailwater. Reducing the uplift water pressure beneath the power house by using relief wells increases the friction forces due to a larger effective weight to keep the structure from sliding and toppling. Until 1979 the pore pressures in the ground and uplift of the power plant were stable and very much as predicted in design. Starting in 1979 the pore pressures underneath the upstream apron started to increase with 0,2 m water column/year.
Using a drainage system underneath the structure increases the gradient in the till material, which promotes the loss of soil. The reduction of uplift water pressures becomes less effective in time, probably due to clogging of the drains under the concrete structure.

3.1.2 Internal erosion of foundation

The regional aquifer in the Amata and Gauja formations, underlying the Plavinas site is causing a considerable groundwater flow towards the buried channel and the right abutment of the dam. This groundwater flow is one of the most important safety aspects related to the Daugava valley dams. Transport of fine soil particles with the groundwater flow is occurring towards the drainage galleries near the right bank (see Figure 22, section 6.1.3) as well as the drainage system underneath the power house. This could cause washout of material in the embankment dam, possibly leading to failure. The erosion under the concrete power house may lead to a reduced shear resistance in the foundation soil.

3.1.3 Settlements

The mentioned transport of soil particles implies that seepage channels are being formed and that they are collapsing from time to time. The collapse of such seepage channels may lead to settlements of the overlying structures. Although the rate of settlement has remained small during about the last ten years, any reduction in the pore water pressures below the foundation may lead to further settlements.
The settlement of the power plant blocks are not uniform, the centre and left side have settled about 100 mm, but the right corner has settled about 300 mm. More serious are the differential movements between the power plant blocks and the adjacent wing walls and embankment structures. At the right abutment, the wing wall has settled 300 mm more than the power plant block and the embankment dam even more
A major collapse of such a seepage channel may also cause sinkholes to penetrate into the embankment dam sections or to the loss of ground support underneath the concrete structures. This process is one of the most serious dam safety concerns.

3.1.4 Movements

Repeated movements of an anchor of an inverted plumb line next to the right abutment indicate that some processes are taking place below the foundation slab of the right hand power house block. The recorded movement is one of the subsoil in downstream direction and towards the left bank.

3.2 PROBLEM DEFINITION

An increase in uplift water pressures under the concrete structure and washout of particles induced by seepage endanger the stability of the Plavinas dam.

3.3 OBJECTIVE

The objective of this thesis is to investigate the weaknesses of the Plavinas dam and to present possible stabilising treatments. This includes rough estimates of costs and construction methods.

4 GENERAL RISK ANALYSIS

4.1 INTRODUCTION

In the following section, an overview of important historic dam failures is given. After this, a general analysis of possible failure modes is performed. As the available data is limited, only a qualitative analysis is possible, followed by the consequences of failure of the Plavinas dam.

4.2 HISTORIC DAM FAILURES[9]

Dam failures are of particular concern because they have the potential to cause more death and destruction than the failure of any other man-made structure. This is caused by the destructive power of the flood wave that would be released by the sudden collapse of a large dam.
Many dams, both large and small, have failed but only a few have had a significant impact on the practice of dam design and engineering geology. The most common causes of dam failures are:

- Overtopping of embankment dams due to incidental high waves or inadequate spillway discharge capacity to pass floodwaters. This is one of the most common causes of dam failures and has nothing to do with the geology of the dam site. Any embankment dam will fail if the spillway is too small and flood waters rise high enough to flow over the top of the dam wall. The estimate of the size of the maximum flood a dam will have to survive during its life is a science, which has undergone continuing evolution over the last century. The result is that many dams built decades ago may now be judged to have inadequate spillways even though the spillways were designed to standards of safety which were accepted as adequate at the time of construction of the dam. Many millions of dollars has been spent upgrading the flood handling capacities of many existing dams, both embankment and concrete dams, as a result.

- Faults in construction methods such as inadequate compaction of fill or use of the wrong type of construction materials (eg silt) may lead to internal erosion or piping failures of embankment dams. An example is the failure of the Teton Dam in Idaho, USA in 1976.

- Geological problems with the dam foundation. The failure of the St. Francis Dam falls into this category. After the failure it was found that some of the foundation rock, a conglomerate, had disintegrated when the rock was immersed in water, and had lost all its strength when saturated. This is exactly what happened when the newly completed dam was filled with water for the first time and the dam failed shortly afterwards. Another example of a dam break due to foundation failure is the Malpasset Dam in France, which failed in 1959. This was the first collapse of a concrete arch dam.

- Landslides, which fall into the storage reservoir, sending a wave of water over the top of the dam may cause a dam to fail. The dam may survive if made of concrete but a destructive flood may still devastate the river valley downstream as happened at the Vaiont Dam in Italy in 1963 when over 1900 people were killed.

- Earthquakes can certainly cause damage to dams but complete failure of a large dam due to earthquake damage appears to be very rare. The Lower San Fernando Dam in California, USA did fail during an earthquake in 1971, which caused the fill in the dam

[9] http://homepages.tig.com.au/~richardw/

body to liquefy resulting in the collapse of the upstream part of the dam. A disastrous flood was only prevented because the reservoir level happened to be low at the time of the earthquake and no water escaped downstream.

- Dams are likely to exist, perhaps for hundreds of years, even after they are no longer required for their original purpose. During these years, dangerous alterations to the operation of the dam and/or its structure may lead to failure. An example is the South Fork Dam, Johnstown, U.S.A., which failed in 1889. Incorrect operation of a dam at any time can result in overtopping and failure as in the case of Euclides da Cunha Dam, Brazil, which failed in 1977.

4.3 FAILURE MODES AT PLAVINAS

In this section possible threats leading to failure of the water retaining structures are investigated. Failure of the Plavinas dam can occur in different ways. A general impression of possible failure modes is given in Figure 6. Slides of upstream and downstream slopes of the hydraulic fill embankment dam are shown. A horizontal slide of an earthen dam is presented even though this is not known to have occurred. Piping for both, the embankment dam and the power house are considered. Also the failure of the concrete power house by toppling, horizontal and circular slides are illustrated in Figure 6.

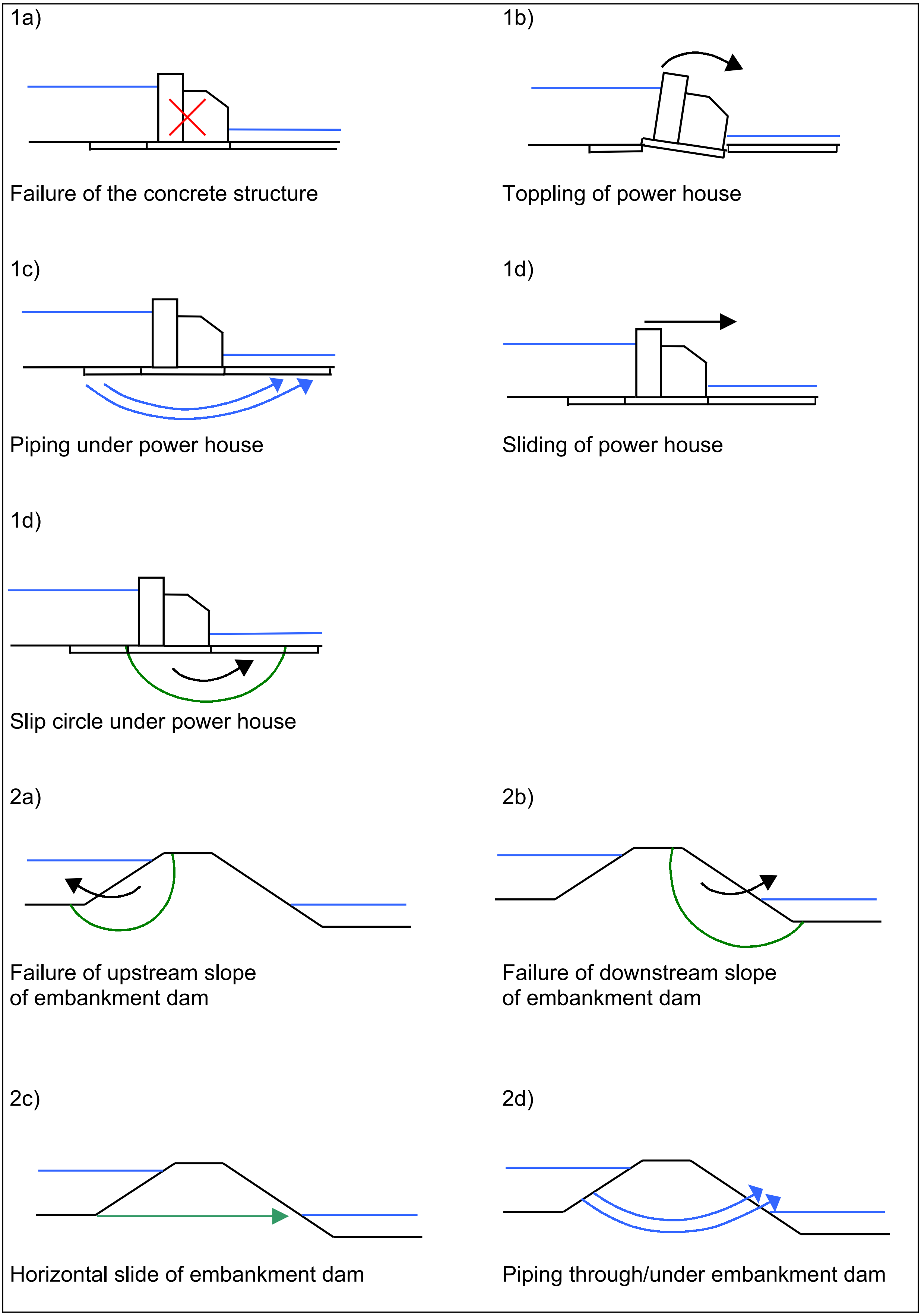


Figure 6: Failure modes for embankment dam and power house structures

A schematic overview presenting possible causes of failure is attached in Appendix 12, which will be explained below.

1) Failure of the concrete structure
   a) A damaged concrete structure can cause the dam to lose its ability to retain the water in the reservoir. This can be the result of:
      i) Incidental large forces such as induced by an earthquake
      ii) Differential settlements caused by long term loss of volume in the soil or different soil/load distribution along the structure. Background of this loss of soil can be groundwater flow under the concrete structure washing out the fine fraction of the soil, leading to the formation of cavities. In case of relatively homogeneous soil, these cavities may collapse if the surrounding soil is no longer able to support the structure. In case of inhomogeneous soil with large gravel, the fines will be washed out, but settlements are less likely to occur as the large fraction can support the structure longer. In the foundation soil under the left part of the power house structure, gravel can be found, whereas the right side is founded on more homogeneous soil. The observed settlements of 100 mm and 300 mm at the left and right side respectively seem to confirm this.
      The difference in soil/load distribution can be induced by an increased use of relief wells at the right hand side of the dam compared to the drainage under the left side or strong vibrations of some turbines, causing liquefaction.

   b) Toppling will cause the structure to move around a rotation point, thereby causing damage to the sensitive drainage system underneath the concrete structure and allowing water to escape downstream. Causes can be:
      i) A larger arm of force, due to extremely high water levels caused by a high river discharge, an earthquake or a landslide into the reservoir.
      ii) Incidental large forces induced by waves (from an earthquake or a landslide) can favour toppling. Also an earthquake itself can cause large additional vertical and horizontal forces on the structure.
      iii) Reduced vertical forces in downward direction can be the result of a reduction in the use of the relief wells, a malfunction of the drains or the removal of weight such as the temporary removal of turbine / generator parts during maintenance / replacement.

   c) Piping may lead to unacceptable leakage along/under the power house. This occurs in case of :
      i) Piping along wing walls of the power house, which abut the embankment dam may appear in case the power house loses its connection with the slope protection, which is assumed impermeable. Such a rupture may develop as a result of incidental large forces induced by an earthquake or differential settlements. These may again be caused by long term loss of volume in the soil or different soil/load distribution along the structure. Background of the loss of soil can be groundwater flow under the concrete structure washing out the fine fraction of the soil, leading to the formation of cavities. In case of relatively homogeneous soil, these cavities may collapse if the surrounding soil is no longer able to support the structure. A difference in soil/load distribution can be induced by an increased use of relief wells at the right hand side of the dam or strong vibration of some turbines, causing liquefaction.
      ii) Groundwater flow under the concrete structure, washing out the fine fraction in the soil, thereby increasingly facilitating the outflow of water and internal erosion. Especially the drainage system under the power house increases the gradient

considerably, which favours the washout of particles if the drains do not fulfil their filter function properly.

d) A slide of the power house on a horizontal or circular surface in the soil can be caused by:
   i) Insufficiently large friction forces to resist the horizontal loads. The loads may originate from large horizontal forces caused by waves from an earthquake / landslide or an earthquake itself.
   ii) Reduced vertical forces in downward direction reduce the friction forces, which help to resist sliding. This can be the result of a malfunction of the relief wells under the power house or the removal of weight such as the temporary removal of turbines during maintenance/replacement and more importantly an earthquake with large uplift forces.

2) Failure of the embankment dam can be the result of:
   a) Failure of reservoir side slope of the embankment dam caused by
      i) Surface erosion when extreme rainfall coincides with an inadequate slope protection, which is pushed off the dam, and the slope is left unprotected. Then, erosion of fill material will occur due to rainfall. Other causes for surface erosion can be overtopping of the dam by waves from an earthquake / landslide or wave attack during a storm in combination with inadequate slope protection.
      ii) A slip circle can form in case of malfunction of drains (leading to higher water pressure and lower effective stress in the soil) or large horizontal forces occur, caused by waves from an earthquake / landslide or an earthquake itself.
      Rapid draw down of the reservoir level due to reservoir operation or emergency emptying to prevent failure of the dam (in case of an emergency situation such as war or an expected extra-ordinary flood) could also initiate sliding.

   b) Failure of downstream slope of the embankment dam can occur in case of
      i) Surface erosion when extreme rainfall and a malfunction of drains lead to flow off with transport of material of the dam. Another cause for surface erosion can be overtopping of the dam by waves from an earthquake or a landslide. In case of a relative rise of water level (by settlement of the embankment dam or a high river discharge) and a malfunction of the spillway gates, the water level will rise until it flows over the embankment dam and (partially) washes away the hydraulic fill dam.
      ii) A slip circle can form in case of malfunction of drains (leading to higher water pressure and lower effective stress in the soil) or large horizontal forces occur, caused by waves from an earthquake / landslide or an earthquake itself.

   c) Sliding of a major part of the embankment dam could occur as a result of
      i) Liquefaction due to an earthquake.

   d) Piping can lead to unacceptable leakage through/under the embankment dam. This can occur in case of presence of:
      i) Karsted rock, which has inhomogeneous characteristics, allowing water to leak through the less dense / fractured zones bordering the fill dam foundation.
      ii) Groundwater flow under the embankment dam, washing out the fine fraction in the soil, thereby increasingly facilitating the outflow of water and again internal erosion. The high relief wells in the right embankment dam probably have facilitated this flow.

4.4 CONSEQUENCES OF FAILURE

The failure of the Plavinas dam, either in the embankment or the concrete power house, will have severe consequences for the area. Latvia is a very flat country, the highest elevation is at 312mASL. In the area of the Plavinas dam and downstream towards Riga, hardly any hills are encountered, see Figure 7. As a consequence the sudden failure of the Plavinas dam would result in the propagation of a dam break wave of the reservoir water causing devastation in the surrounding area due to the impact of the water. Also pollution of the water is an issue. In Latvia air and water pollution are caused by a lack of waste treatment equipment. The Gulf of Riga and the Daugava River are heavily polluted. Contamination of soil and groundwater with chemicals and petroleum products at military bases are common[10].

Figure 7: Map of Latvia showing topography[11]

4.4.1 Towns and villages[12]

The population of Latvia is 2,4 million. A large number of people live in the capital city Riga (890 000) and the major cities Daugavpils (127 000), Liepaja (108 000), Jelgava (72 000) and Jurmala (60 000). The remaining population is spread across several smaller towns and villages.

The areas of concern are the Aizkraukle district (2557 km^2, Population 45 000), Ogre district (1836 km^2, population 65 000), Riga district (3067 km^2, population 149 000) and Riga (307 km^2, population 890 000). In case of a gradual dam failure, mainly the area along the Daugava river is threatened.

Downstream of the Plavinas dam, there are several small villages, see Figure 8. They are located around the valley, only two (Lejasvekteri and Jaunjelgava) are situated along the roads following the river. Devastation of these villages even in case of a gradual dam failure may occur due to the close vicinity of the dam.

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10 http://www.odci.gov/cia/publications/factbook/geos/lg.html

11 http://www.mapquest.com

12 http://www.eunet.lv/VT/general/

If however the dam suddenly loses its ability to retain the reservoir water, a wider downstream area is threatened.

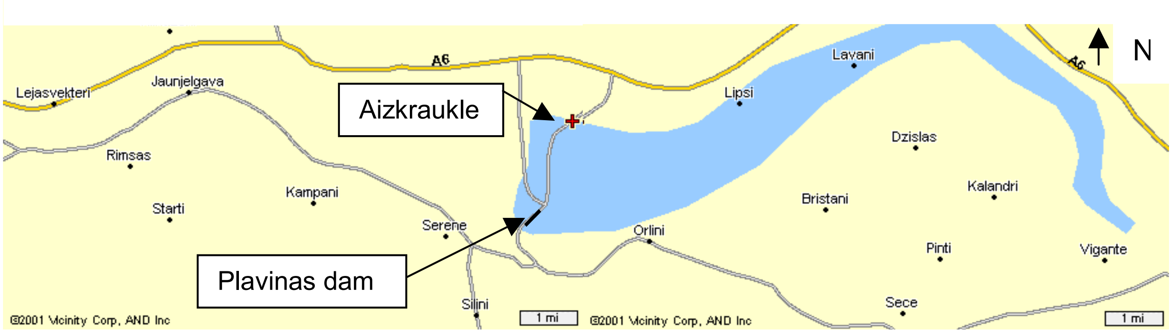


Figure 8: The reservoir of the Plavinas dam and surrounding villages

If the Plavinas dam failed or overtopped, the Daugava River would receive a part of the additional water, leading to a wave heading downstream towards the next level of the Daugava cascade, Kegums hydroelectric power plant, see Figure 9. The impact of the incoming wave might cause the failure (or overtopping) of the Kegums plant. This again may lead to material and human losses in that area. Again, part of the additional water will continue on the Daugava, causing another wave in the downstream direction.

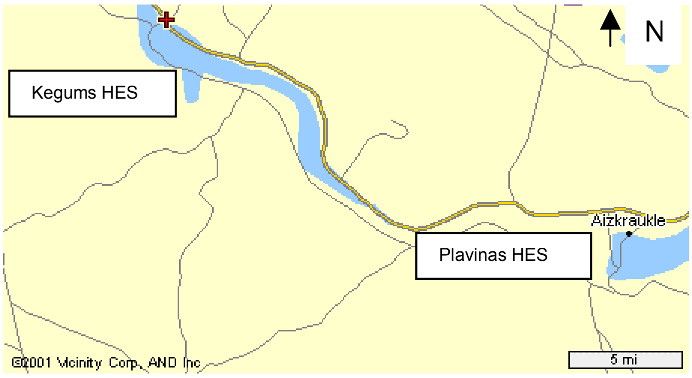


Figure 9: Location of Plavinas and Kegums

The town of Ogre would be likely face the consequences of such a flood. Continuing further downstream towards the last step of the cascade, the wave reaches the Riga plant, see Figure 10 and Figure 11. If this scheme failed (or overtopped) many lives would be in danger. Even though the original head difference is smaller at this site, the danger of a large number of casualties here is considerable due to the higher population density.

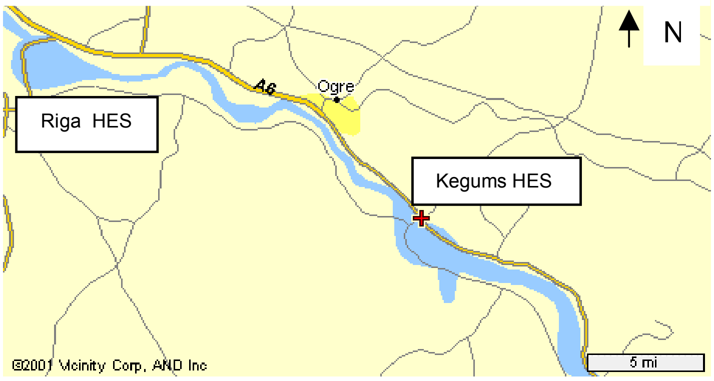


Figure 10: Location of Kegums

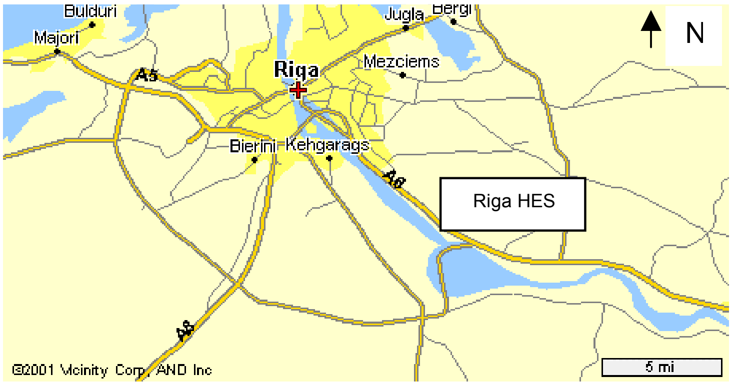


Figure 11: Location of Riga and Kegums

4.4.2 Additional consequences

Power generation

In case of a dam failure, a large part of Latvia's power production would cease to exist. The attempts of gaining more independence by importing less energy would be rendered futile. The statistics of "clean" energy would deteriorate dramatically as the Plavinas dam and the rest of the Daugava cascade generate a major part of the renewable energy. This may even have some political effects as Latvia is seeking acceptance into the European Union and is improving energy and environmental policies toward that end[13].

Infrastructure

The highway and railway from Riga to the city of Daugavpils in the south-eastern part of Latvia go through the towns Ogre, Kegums and Lielvarde, on the banks of the Daugava River. A dam failure or overtopping could cause flooding of this highway and railway, possibly pushing cars and/or trains off the track, and inflicting human and material damage. Also the highway and railway infrastructure may suffer damage.

Industry

The economic and geographical location of the Ogre district and the nearness of Riga, are advantageous for developing various sorts of entrepreneurial activities. Therefore, the industry is not only focused on the city of Riga. Some industry has settled near Ogre. As these areas are in the close vicinity of the Daugava River, damage by a flood is almost certain.

Agriculture

A dam failure (or merely overtopping) would flood the local area, including farmland. The water will destroy the crops and cause losses among the livestock. On top of that, the water of the Daugava River is heavily polluted. This may prohibit further agricultural use of the land in the future.

Environment

The main concern for the wildlife in case of a dam failure/overtopping is the impact on the environment and thus the wildlife. It is not hard to imagine that a highly polluted flood causes losses among the flora and fauna. The natural environment of some species may severely be altered.

[13] http://www.fe.doe.gov/international/latvover.html

5 BOUNDARY CONDITIONS

The following sections treat the boundary conditions of the Plavinas dam. These are of interest in order to analyse the stability of the powerhouse. Section 5.1 provides information on the geology of the foundation soil. Section 5.2 shows the schematisation of the power house. Section 5.3 presents the average and extreme water levels. In section 5.4 the background of the uplift water pressures is explained. Resulting from these sections, the loads on the power plant are determined in section 5.5 (horizontal loads) and section 5.6 (vertical loads). Finally, the loads are summarised in Figure 21in section 5.7.

5.1 GEOLOGICAL DATA

5.1.1 Regional lithology[14]

The Plavinas plant is located in an area shaped by pre-quaternary, glacial and interglacial action as well as recent river valley developments. Over a geologically long period, this ancient valley served as regional ground water drainage. Thus, it can be assumed that both of the valley flanks were weakened by scour due to the groundwater, and/or by the shear and drag caused by glacial action later. Furthermore, it can be assumed that valley flanks have additionally been fractured and weakened by stress relief during inter-glacial phases.
During and after glaciation, the valley in which the dam is located, was filled with glacial deposits covering bedrock, creating the Daugava 'buried valley' of today (estimated depth of 80 m). These deposits consist predominantly of till. The glacial till material has been investigated both in the field and in the laboratory. Following Russian soil classification practice, the designers of the power plant distinguished two kinds of glacial till with a gentle transition between these materials, the so-called loam and sandy loam. The difference between the two is small and lies in the clay content. The sandy loam has a lower amount of clay and is generally at the bottom of the valley fill and above bedrock.

Left hand side of the power house founded predominantly on loam, whereas the right hand side of the power house is founded on predominantly sandy loam (less dense than loam). No significant amount of boulders are found in the till, however, stone and coarse gravel sized particles (20-200 mm) are encountered regularly.

Appendix 3 presents a schematic longitudinal section through the power house and the embankment dams. It is based on Figure 12 and Figure 13, which display a longitudinal section across the valley taken along the axis of the power house including the geology under the structure. Note that Figure 12 is a mirrored image, with turbine pit 1 belonging to the right hand side of the dam viewed from upstream. In Figure 13, the vertical scale is 5 times the horizontal scale and the depth and shape of the lower part of the valley are assumed. In the remaining report, the geology will be assumed according to Appendix 3.
The embankment dams consist of hydraulic fill, which usually involves sandy material. Figure 13 indicates that loamy sand is present in the area, so the use of this material is assumed for the embankment dams. The left part of the power house is founded on loam with gravel and cobbles, whereas the right part is founded on morainal sandy loam (slightly stiffer).

[14] Norplan A.S. of Norway, Latvenergo, European Bank for Reconstruction and Development, *Feasibility study and project preparation for rehabilitation of the Daugava river hydropower schemes, Latvia,* Final report: February 1995

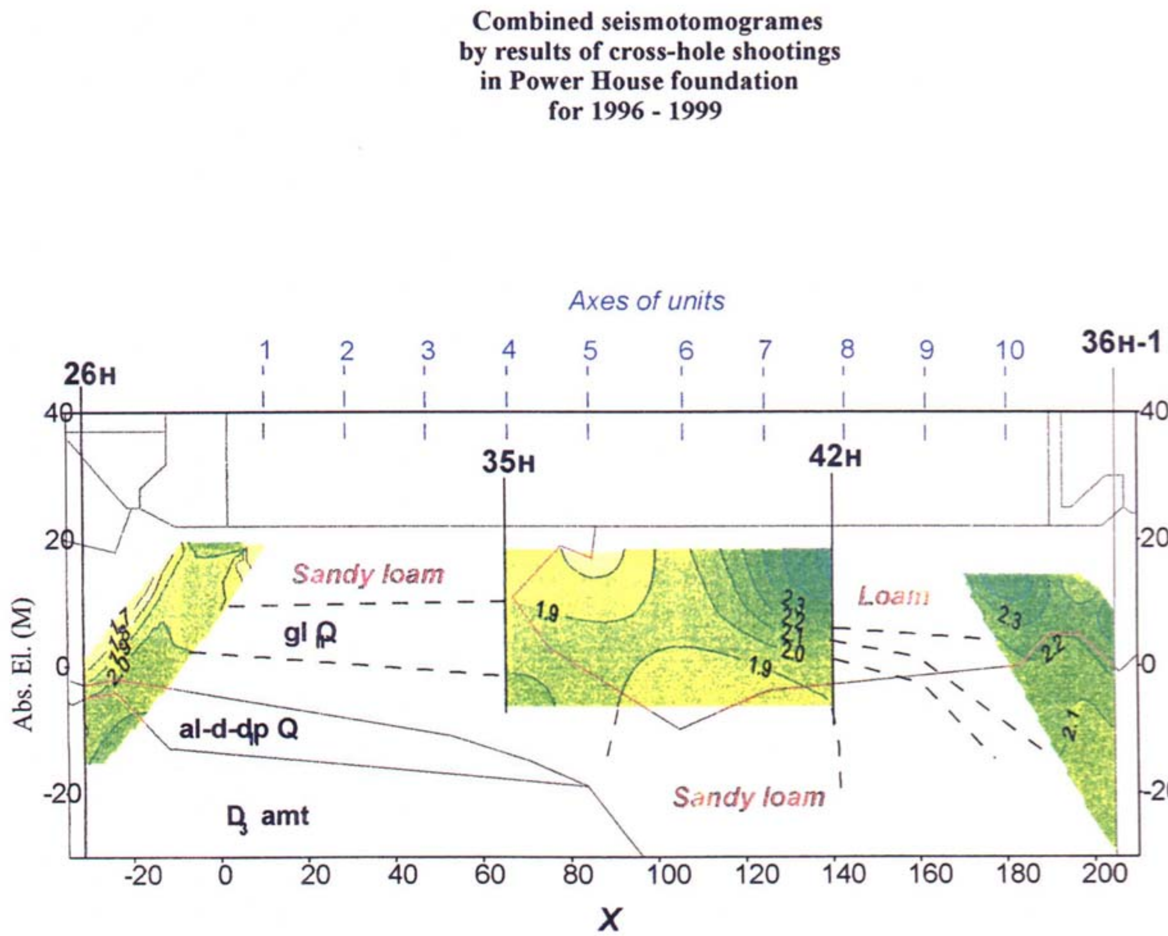


Figure 12: Longitudinal section foundation soil

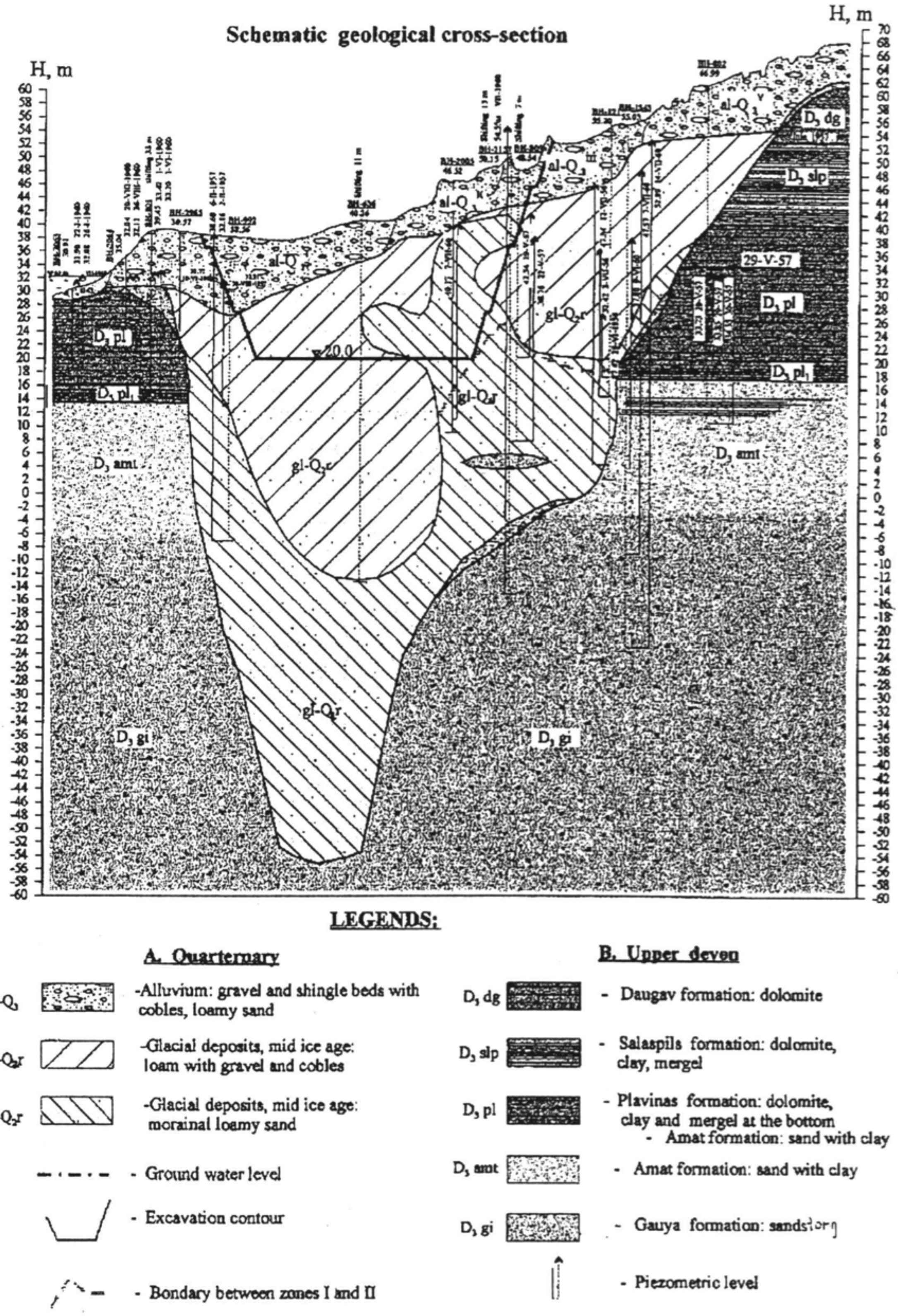


Figure 13: Geological section (longitudinal) across the valley along the axis of the power house[15]

[15] Joint Stock company Latvenergo, *Tender Documents, Scope of Services*. Aizkraukle: April 2001

5.1.2 Schematisation of subsoil

Below, a description of the present formations is given. Note that the Amata formation is named weakly cemented sandstone and fine sand with clay interlayers, whereas Figure 13 describes this layer to consist of sand with clay. However, the Amata formation was mentioned[16] to be a rock formation, which coincides with the contents of Table 2. Therefore the presence of weakly cemented sandstone (with sand and interlayers of clay) is assumed.

Table 2: Existing formations[17]

Formation	Denomination	Lithology	Thickness
Daugava	D_3 dg	Fine cristalline dolomite, pelite and marlciaous, fissured cavernous, karstic with dolomite flour	15 m
Salaspils	D_3 slp	Compact marlacious clay with interlayers of marl and dolomite	25 m
Plavinas	D_3 pl	Fine cristalline dolomite, fissured cavernous with subordinate interlayers of marl and plastic clay	25 m
Amata	D_3 amt	Weakly cemented sandstone and fine sand with clay interlayers	25 m
Gauja	D_3 gj	Weakly cemented sandstone and sand with rare clay interlayers	> 100 m

Table 3 presents typical values of physical and engineering properties that were found from laboratory tests of the glacial till material[18]. When consulting the classification diagram for soils in Appendix 9, these values coincide with soil parameters of loam. As can be seen in the below table, gravel is encountered in this layer.

Table 3: Soil parameters

Moisture content	w	9,9 – 10,7 %
Total unit weight	γ	22,6 – 22,9 kN/m^3
Unit weight of solid material	γ_s	26,1 – 26,35 kN/m^3
Plastic limit	w_P	11,6 – 12,3 %
Liquid limit	w_L	20,3 – 21,4 %
Content of clay sized fraction (<2μ)		14,8 – 16,9 %
Content of silt sized fraction		29,1 – 30,7 %
Content of sand sized fraction		43,3 – 48,0 %
Content of gravel sized fraction		6,8 – 11,1 %
Void ratio	e	0,269 – 0,299
Dry unit weight	γ_d	20,4 – 20,9 kN/m^3
Compression index (oedometer)	C_c	0,052 – 0,079
Friction angle (from CU triaxial tests)	tan ϕ‘	0,738 $\pm$ 0,048
Mineralogical composition of clay fraction		Illite 70%, kaolinite 25%, chlorite 5%

The value of tanϕ‘ in Table 3 corresponds to 34,6°< ϕ’ < 38,2°, which coincides with frequently assumed values of loam in the Dutch standards NEN 6740[19] with a value of

[16] Institute Hydroproject, *Main findings of geophysical explorations in the area of Plavinas HPP*. 1472-T.18. Moscow: August 2000.

[17] Norplan A.S. of Norway, Latvenergo, European Bank for Reconstruction and Development, *Feasibility study and project preparation for rehabilitation of the Daugava river hydropower schemes, Latvia,* Final report*:* February 1995

[18] Joint Stock company Latvenergo, *Tender Documents, Scope of Services*. Aizkraukle: April 2001

[19] Nederlands Normalisatie- Instituut, *NEN 6740, Geotechnics TGB1990-Basic requirements and loads*. December 1991

ϕ'=35°. This matches the values observed by NGI, see Appendix 10, where c'=10 kPa and ϕ'=34° (up to ϕ'=37°) are used.
The original design parameters (by Hydroproject) of the till material were c'=25 kPa and ϕ'=24,2°, claiming that a weak layer runs under the entire power house. Soil samples from NGI do not confirm these values. However, a layer of sandy loam covers half the foundation between generators 1-5, see Figure 12. Parameters of this layer were not provided. They are assumed to be lower than the values for loam. The sandy loam was mentioned to be less dense than loam and as there are no data on this material it will be assumed to posses the lowest values of ϕ'=27,5°-35° and c'=0-2 kPa, which can be found in the Dutch standard NEN 6740. The sandy loam material lies beneath the foundation of one of the two blocks of the power house, see Figure 12 and Figure 14. The values used in the future are summarised in Table 4.

CUR 162[20] gives information on deposits from glacial origin, containing clay, silt, sand, gravel and stones. Most of this material has a high sand fraction. The permeability of loam is mentioned to be approximately $5*10^{-9}$. The till material at the right side of the valley consists of sandy loam, which is slightly more permeable. The hydraulic fill material of the embankment dams is assumed to consist of loamy sand material, which can be found in the alluvium of the river. The permeability of sandy loam is assumed to lie between the value of fine sand (10^{-5}- 10^{-6}) and loam ($5*10^{-9}$), resulting in the values below.

Table 4: Parameters of till material

Soil	**ϕ' (°)**	**c' (kN/ m^2)**	**γ (kN/m^3)**	**γ_{sat} ((kN/m^3)**	**k (m/s)**
Loam (with gravel)	35	5 – 7,5	22,6 – 22,9	26,1 - 26,4	10^{-8} - 10^{-10}
Sandy Loam	27,5	0 - 2	19 – 20	19 –20	10^{-7}- 10^{-8}

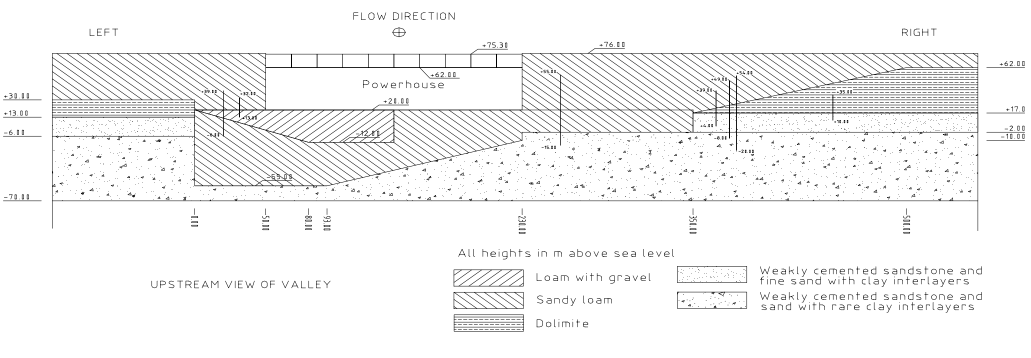


Figure 14: Schematisation of foundation soil at Plavinas in the direction of the dam axis

5.1.3 Regional hydrogeology

Along the Daugava valley, the groundwater conditions can be characterised by an open aquifer in the quaternary deposits and the Daugava formation and a closed aquifer in the Plavinas, Amata and Gauja Formations. The Gauja and Amata aquifer covers most of eastern Latvia. The pressure is artesian under the valley. At Plavinas the pressure is rising

[20] CUR-publicatie 162, Construeren met grond – Grondconstructies op en in sterk samendrukbare en weinig draagkrachtige grond, CUR, Gouda. April 1992

from 55 mASL in the valley to 120 mASL in the watershed. The aquifer is believed to be discharging into the reservoir in a window where the pre-glacial buried valley intersects into the Amata formation.

The range of permeability[21] of dolomite and sandstone is $10^{-5} – 10^{-9}$, respectively $10^{-5} – 10^{-10}$. In order to gain more precise values, additional pumping tests should be performed. However, the sandstone layer was mentioned to be a closed aquifer, meaning that the layer is relatively impermeable, therefore hardly penetrating the sandy loam layer above. Therefore they do not interfere with the processes in the above layers. Only the area of the right bank is known to be more permeable (see Appendix 11). Here, relief wells are placed to prevent large uplift forces on the overlying soil and downstream apron. This area of reduced density is assumed to be restricted to the area of the right bank up to the end of the downstream apron.

The Latvian plain is dominated by Devonian sedimentary rocks, overlying Silurian and Cambrian sedimentary rocks. The depth to basement is 800 to 1000 m in the Daugava valley. The Plavinas plant is situated in the upper Devonian rocks, mainly dolomite, sandstone marl and plastic clay.

5.1.4 Regional seismicity[22]

Latvia is located on the border of two seismic relatively non-active platforms represented by the East-European platform to the east and the Baltic shield to the west. In general the earth's crust in Latvia can be considered as a continental crust of stable platforms. The seismicity of the Daugava valley is modest. However, as recently as 1976, an earthquake of magnitude 4,8 on the Richter scale occurred just north of the Latvian/Estonian border, which can be translated to an earthquake with a peak acceleration of approximately 0,15 g.
An estimate for a design earthquake is 5 to 5,5 Richter magnitude causing a peak ground acceleration of not more than 0,2 g. The stability of the dam under earthquake loading has not been checked.

[21] Verhoef, drs, P.N.W., *Ingenieursgeologie*. Delft University of Technology, Subfaculty of Geosciences. Forth edition: October 1994
[22] Norplan A.S. of Norway, Latvenergo, European Bank for Reconstruction and Development, *Feasibility study and project preparation for rehabilitation of the Daugava river hydropower schemes, Latvia,* Final report: February 1995

5.2 SCHEMATISATION POWER HOUSE

In order to perform a more detailed analysis of the Plavinas dam, a schematisation of the concrete power house including the foundation slab is required. Using the schematised cross section and longitudinal section in Appendix 1 and Appendix 2, a calculation of the weight of the structure was performed in Appendix 15 section 1. The different elements contributing to the weight of the power house are listed below.

1. Cross section

By using the cross section of the power house the area of concrete was measured and subsequently multiplied by the length of the powerhouse across the valley. The areas of the cross section are numbered and can be found in Appendix 15 section 1.1.

2. Longitudinal section

In longitudinal direction the areas in Appendix 15 section 1.2 contribute to the weight of the structure. The length of these areas is assumed to coincide with the length of the main room where the generators and turbines are located.

3. Spillway pillars

There are ten large spillway pillars at the upstream side of the power house, which are assumed to consist of solid concrete. The dimensions of these pillars can be found in Appendix 15 section 1.3.

4. Draft tube pillars

Ten smaller pillars are found at the downstream side of the power house. They continue from the downstream end of the foundation slab to the right side of the inner room where the turbines are located. The dimensions of these pillars can be found in Appendix 15 section 1.4.

5. Weight turbines

The generator and turbine are combined in the schematisation. The radius and height of the generator were measured and the volume of this cylinder was determined. The generator is assumed to consist of a very high percentage steel (weight of turbine itself is integrated in this assumption), see Appendix 15 section 1.5.

6. Weight sliding gates on spillway

On top of the concrete spillway, sliding gates of steel are located between the upstream pillars. The thickness, height and width of the barriers were measured, leading to a volume of steel, which had to be reduced as the barriers are not solid, see Appendix 15 section 1.6.
The same weight is expected in the intake gates of the turbines. The intake can be blocked in case maintenance works are necessary. Still, this additional weight has not been taken into account, as these gates are not constantly positioned in front of the intakes.

7. Weight water near intake

At the upstream part of the power house foundation slab the reservoir water contributes an important part of the loads on the foundation. The volume of water is determined by measuring the width of the slab on which the water can be found, multiplying this by the length across the valley and the depth of the water. The volume of upstream pillars is subtracted see Appendix 15 section 1.7.

8. Weight water in draft tubes

The weight of the water inside the draft tubes (which are located inside the power house behind the turbines) is calculated by multiplying the depth of the water with the length and width of the tubes. This is subsequently multiplied by the number of draft tubes and the specific weight of water, see Appendix 15 section 1.8.

5.3 WATER LEVELS

The maximum and average water levels are summarised below. No minimum water level is given for the reservoir. Still, the operator will attempt to stay close to the maximum water level in order to use a large head difference for the production of electricity. For variations in water level see Appendix 26.

Table 5: Extreme and average water levels at the Plavinas dam

Water level in:	Maximum water level	Minimum water level	Average water level
Reservoir	+72 mASL	-	+71 mASL
Tailrace	+36 mASL	+32 mASL	+33 mASL

5.4 UPLIFT WATER PRESSURES

5.4.1 Uplift pressures at end of aprons

The uplift water pressures originate in the head difference between the reservoir and the tailrace. The boundary conditions for the uplift water pressures at the end of the upstream and downstream apron are the average water level in the reservoir and the tailrace. Variations in water level take time to influence the pressures and therefore the average pressure is close to reality.

Both, the upstream and the downstream apron seal the foundation soil from the above water. The embankment dams possess a slope protection, which continues down to the end of the upstream apron. This layer seals the area around the apron. As a result, the uplift water pressures under the aprons and power house have their boundary condition at the upstream end of the apron and not closer to the power house. The latter would be the case if water could find its way from the sides.

The average depth of water in the reservoir is +71 m – 32 m = 39 m. The apron is 2,5 m thick. The uplift water pressure under the upstream end of the apron is (39 m + 2,5 m)*10 k/Nm^3 = 415 kN/m^2, see Figure 15.

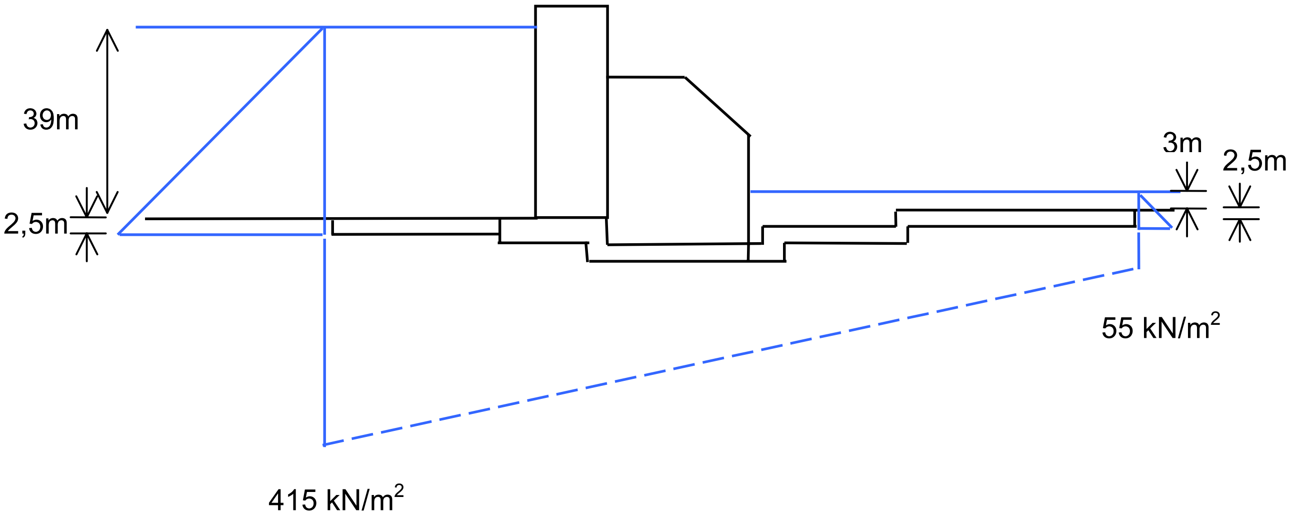


Figure 15: Uplift pressure at the end of the aprons

This is the boundary condition for the uplift pressure at the end of the upstream apron. The boundary condition at the end of the downstream apron can be found the same way. The average depth in the tailrace is +33m-30m=3m. The apron is 2,5 m high. The uplift water pressure at the end of the downstream apron is (3 m+2,5 m)*10 k/Nm^3 = 55 kN/m^2, see Figure 15.

5.4.2 Uplift pressure under the power house without drainage wells under power house

Without drainage system the uplift water pressures are calculated in the following way. The gradient below the structure is:

$$i = \frac{\Delta H}{L_{hor} + L_{vert}}$$

$$i = \frac{71m - 33m}{282{,}2m + 17m} = 0{,}127$$

with
i = gradient (m/m)
ΔH = head difference (m)
L_{hor} = horizontal seepage path (m)
L_{vert} = vertical seepage path (m)

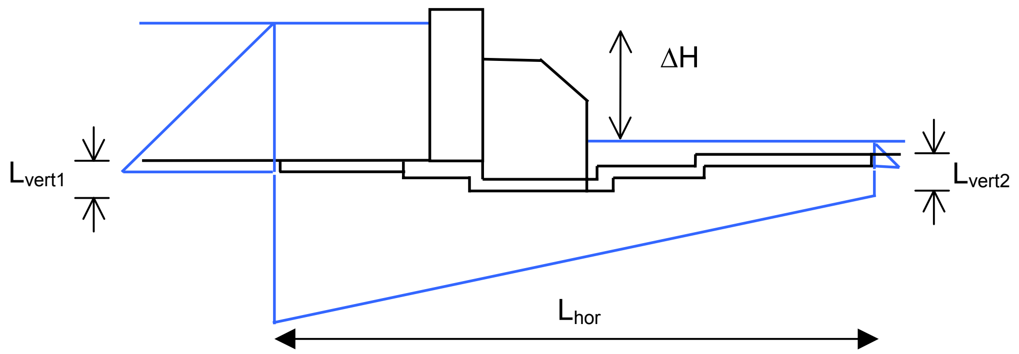


Figure 16: Shape of uplift water pressure distribution under the power house without drainage wells

The gradient provides information about the change of piezometric level per length. When translating this into changes of pressure under the plant, the gradient is multiplied by ρ*g. This corresponds to i=1,27 kN/m^2 per m.
The uplift pressures under the structure are assumed to be reduced linearly between the given pressures at the boundaries, see Figure 16. Still, some parts of the power house and apron are located at a lower elevation than others, which results in jumps in the uplift water pressures (the deeper the reference level the higher the uplift pressures). When calculating the pressures under the structure, the gradient is multiplied by the distance from the previous known point and subtracted from the pressure in the previous point. If the pressure is calculated in a place of lower elevation than the previous, the pressure will be larger by the vertical distance in meters water column minus the vertical distance multiplied by the gradient.
The uplift loads under the natural gradient under the power house (no drainage wells operational) are approximately 350-400 kN/m^2, see Appendix 14 and Appendix 16 section 3.

5.4.3 Uplift pressure under the power house with drainage wells under power house

The drainage system under the power house reduces the uplift water pressure originating from the large head difference between the reservoir and the tailrace, see Figure 17. In the design of the power house the uplift pressures were supposed to be reduced by 90%[23]. This implies a reduction of uplift pressure from about 350-400 kN/m^3 to approximately 40 kN/m^3 near the drainage wells under the power house, see Appendix 14.

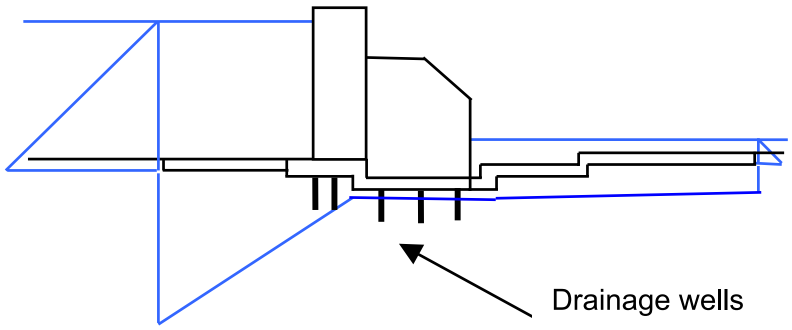


Figure 17: Uplift water pressures due to drainage wells under power house

The pressures due to the drainage are calculated by using the boundary conditions at the end of the aprons in Appendix 15 section 3. The uplift water pressures are calculated at a certain elevation. Some parts of the power house and apron are located at a lower elevation than others, which results in jumps in the uplift water pressures (the deeper the reference level the higher the uplift pressures). This can be seen in Appendix 13 for drainage according to design assumptions. When calculating the pressures under the structure, the gradient is multiplied by the distance from the previous known point and subtracted from the pressure in the previous point. If the pressure is calculated in a place of lower elevation than the previous, the pressure will be larger by the vertical distance in meters water column minus the vertical distance multiplied by the gradient.

[23] Norplan A.S. of Norway, Latvenergo, European Bank for Reconstruction and Development, *Feasibility study and project preparation for rehabilitation of the Daugava river hydropower schemes, Latvia,* Final report*:* February 1995

5.5 HORIZONTAL LOADS

In this section the existing horizontal loads on the power house structure are described. They are summarised in the stability analyses of Appendix 15 to Appendix 24, section 7.

5.5.1 Horizontal loads due to head difference

The most important loads on the Plavinas dam originate from the large head difference between the reservoir and the tailwater level. For the normative case the most extreme water levels from Table 5 are used, see Table below:

Table 6: Water levels in reservoir and tailrace

	Water table	**Depth**
Reservoir	+72 mASL	52m (40m water above apron + 12m to lowest part foundation)
Tailwater	+32 mASL	12,5m (9m water above apron + 3,5m to lowest part foundation)

The maximum water level in the reservoir gives the most unfavourable loading case. The horizontal hydrostatic load is calculated by

$$H = \frac{1}{2}\rho_w g h^2 L$$

with

H = horizontal load (N)
ρ_w = specific weight water (kg/m^3)
g = gravity (m/s^2)
h = depth water (m)
L = length across valley (m)

The weight of the water on the structure is calculated by:

$$W = \rho_w g h L$$

The depth of water has a larger influence on the horizontal load than the weight has on the structure. For the horizontal load, a larger water depth than the reservoir depth is used, see Table 6. This is done because the structure reaches several meters under ground where the water contributes to the horizontal loads. The weight of the water however is only found on top of the foundation slab. Furthermore, the depth of water is of quadratic influence for the horizontal load and of linear influence for the weight.

The horizontal load from the reservoir water is named H1, the load in opposite direction from the tailwater level is named H2, see section 5.7. The values of these forces are given in Appendix 15, section 7.

5.5.2 Horizontal loads due to ice pressure

A rough estimate of ice loads is done by assuming ice shelf formation on the reservoir up to a thickness of 0,5 m. The reservoir and discharge are fairly small so that no ice accumulation on the power house is assumed. Still, thermal expansion can cause horizontal forces. If assuming the same loading case on sheet piles as has been adapted in CUR 166[24] at a rise

[24] CUR-publicatie 166, *Damwandconstructies*, CUR, Gouda. March 1997

of ice temperature of 6°C a day, a pressure of 150 kN/m can be expected. This load will be used later on though it is very small. The arm of this force is assumed to be located at the maximum water level.
The load due to the ice pressure is named H3 in Appendix 15, section 7.

5.5.3 Horizontal loads due to soil pressure

Active soil pressure
At the upstream side of the power house foundation slab, the soil pressure favours a slide of the plant in downstream direction. The pressure is determined by taking into account the existing stress from the reservoir water load of 40 m water. An additional soil pressure from 12,5 m soil, from the top of the apron at +32 m to the bottom of the power house foundation slab at +19,5 m is assumed. The pressure is transformed into an active soil pressure by using the approximate active soil pressure coefficient of K=1/3. The arm of force was calculated in proportion of the vertical weight and the pressure in the soil. For this load, no load factor was used as a safety was already implemented by using the design values of the angle of internal friction.
The load due to active soil pressure is named H4 in Appendix 15, section 7.

Passive soil pressure
At the downstream side of the power house foundation slab, the soil pressure resists a slide of the plant in downstream direction. The pressure is determined by taking into account the existing stress from the tailwater load of 3 m water. An additional soil pressure from 10,5 m soil, from the top of the apron at +30 m to the bottom of the power house foundation slab at +19,5 m is assumed. The pressure is transformed into a passive soil pressure by using the approximate passive soil pressure coefficient of K=3. The arm of force was calculated in proportion of the vertical weight and the pressure in the soil. For this load, no load factor was used as a safety was already implemented by using the design values of the angle of internal friction.
The load due to passive soil pressure is named H5 in Appendix 15, section 7.

5.5.4 Horizontal load due to earthquake

When considering an earthquake load, the vertical and horizontal accelerations are assumed not to occur simultaneously.
The design earthquake of 0,2 g is taken into account. It is simulated by introducing 20% of the downward vertical loads on the foundation slab as an additional horizontal load in downstream direction. This facilitates sliding and toppling.
This load is named H7 and has an arm of force of half the distance between the top of the upstream apron and the lowest part of the concrete structure.
The earthquake load of the downstream apron takes into account 20% of the vertical loads on the apron and was named H8.

In case an earthquake is of interest in a calculation, the appendices can be composed of different parts depending on the normative loading case:

- a for the standard loading case, no earthquake
- b when taking into account horizontal loads due to an earthquake
- c when taking into account horizontal loads due to an earthquake

Generally, the horizontal earthquake load has a more severe impact on the stability of the power house. In case the stability calculation determines that the vertical load is normative, this is mentioned.

5.5.5 Support by upstream apron

The upstream apron is a concrete slab, which protects the intake area against erosion. The apron runs from the left embankment to the right embankment dam with a constant length of 54 m, see Appendix 13. It is attached to the power house by a water stop. Transfer of tension forces is limited through this connection. However, the power plant has survived an earthquake in 1976, which was assumed to have peak accelerations of 0,15g. Therefore, the transfer of the difference of 15% of the downward load on the upstream apron and 15% of the downward load of the power house is assumed to be possible in this joint. The value of this load is calculated in Appendix 15b, section 6 and is named H9 in section 7.

5.5.6 Support by downstream apron

The downstream apron consists of a concrete slab, which protects the tailwater area against erosion. The length of this apron is approximately 165 m, see Appendix 13. The friction forces under the apron support the power house. This load depends on the effective downward loads on the soil under the slab and is calculated in Appendix 15 section 6 under the name H6.

5.5.7 Horizontal loads due to navigation

There is no navigational channel from the sea to the Plavinas dam, no locks or other means allowing navigation are present. Therefore, no forces originating from the collision of a ship against the dam are taken into account.

5.5.8 Support from embankment dams

Friction forces between the wing walls and the embankment dams support the power house. The area in which these forces can develop is assumed to measure from elevation +20 mASL to +76 mASL with a width of approximately 30 m, see Figure 18.
The specific weight of the hydraulic fill material is assumed to be 20 kN/m^3 and the neutral soil pressure coefficient is K=0,5. The representative value of the friction angle is assumed to be 27,5° for sandy loam. The friction develops between the fill material and the concrete structure. Therefore the friction coefficient is reduced by 2/3 compared to friction between soil-soil. The arm of force of the resulting friction force equals 1/3 of the height of the friction area, as the soil pressure in the lower part of the friction area is larger (triangular shape of the pressure line). The value of this load is calculated in Appendix 15, section 6. This load is named H10 in section 7.

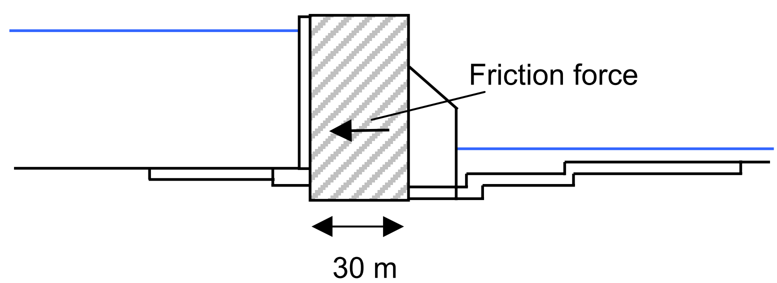


Figure 18: Cross section with support from embankment dams

5.6 VERTICAL LOADS

The vertical loads on the power house foundation slab are summarised in the stability analyses of Appendix 15 to Appendix 24, section 8. The following sections describe the different loads on the structure.

5.6.1 Downward vertical loads

The vertical loads in downward direction represent the weight of the structure, which is carried by the foundation slab.
The loads, which have been taken into account in the schematisation, originate from the weight of different parts of the structure, which have been treated in section 5.2. The loads are presented in section 5.7 and are listed below. The loads are quantified in Appendix 15, section 1and 8.

V1 weight due to cross section
V2 additional weight longitudinal section
V3 weight large pillars
V4 weight small pillars
V5 weight large pillars
V6 weight turbines/generator
V7 weight water near pillars
V8 weight water in draft tubes

5.6.2 Vertical loads due to earthquake

The vertical load due to the design earthquake of 0,2g is simulated by subtracting 20% of the downward vertical forces as this makes the structure lighter and thus more susceptible to sliding (horizontal and non linear) and toppling. The used arm of force equals the position of the resulting vertical force without earthquake. However, this vertical load is more favourable to the stability than the horizontal earthquake load, see Table 7,Table 8 and Table 9 as example.

5.6.3 Uplift loads

The drainage system reduces the uplift water pressures under the structure. The uplift water pressures were the subjects of section 5.4.
The uplift loads due to the water pressure are calculated by multiplying the lowest pressure (a) in the area of the same horizontal reference level (bottom of concrete structure) with the length of this area (c) and the width across the valley. The higher pressure at the end of the area minus the lower pressure gives an additional pressure (b) under the structure with a triangular shape. This difference in pressure is multiplied by the length of the area (c) and the width across the valley and subsequently divided by two (due to the triangular shape of the line of pressure). A more detailed schematisation of the pressures is attached in Appendix 13. The values of the uplift pressures were calculated in the above way and are shown in Appendix 15, section 3. The uplift loads on the structure are given in Appendix 15, section 8.

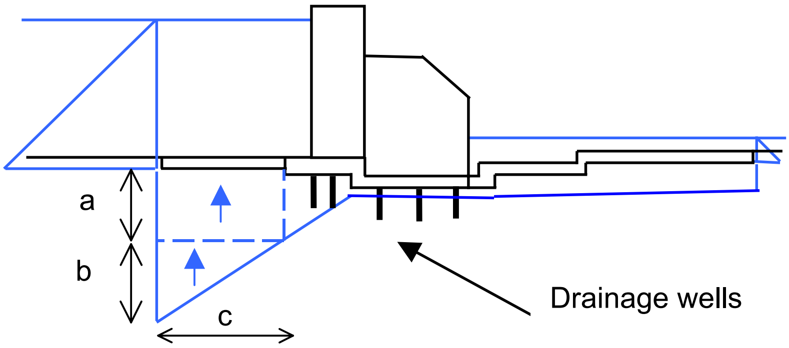


Figure 19: Uplift loads

5.6.4 Pressure under power house

The soil stress under the power house is calculated by using the following formula. The resulting vertical load V composed of the vertical downward loads and the uplift water pressures. Therefore, the stress in the soil under the power plant calculated with the formula below is an effective stress of the soil under the foundation slab.

$$\sigma'_{max,min} = \frac{V}{B_{ef}} \pm \frac{M}{\frac{1}{6} B_{ef}^{\ 2}}$$

with

V = total vertical force (kN)

B_{ef} = effective width foundation (m)

M = total moment (kNm)

$\sigma'_{max,min}$ = effective maximum and minimum stress in the foundation soil (kPa)

A schematic of the minimum and maximum pressure under a foundation slab under vertical and horizontal loads is given below.

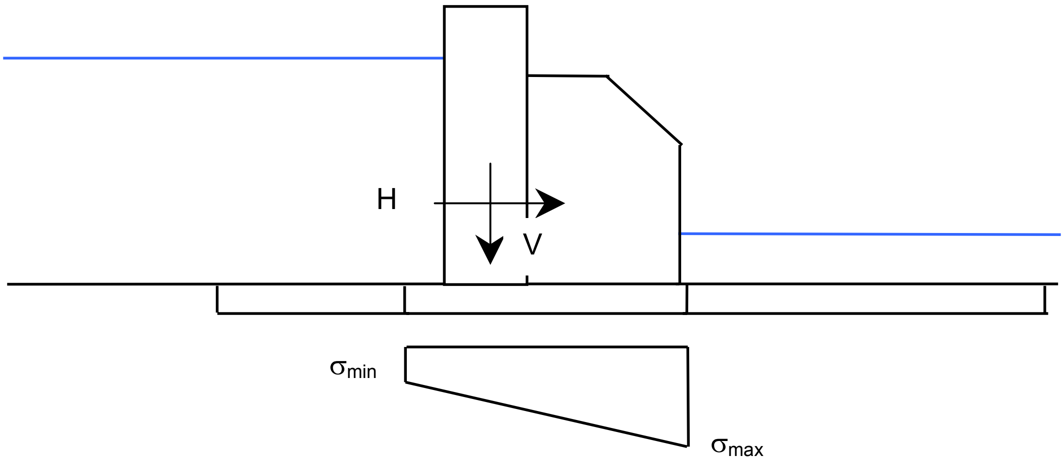


Figure 20: Minimum and maximum pressure under foundation slab

5.7 SUMMARY LOADING CASE

A summary of the loads from the previous sections is given in Figure 21, which unites the horizontal and vertical loads from the previous sections.
For the standard loading case, partial load factors were used, see Appendix 15 sections 7, 8 and 9. In case of a horizontal and vertical earthquake no load factors were used when checking the stability because this is an exceptional loading case which hardly ever occurs.

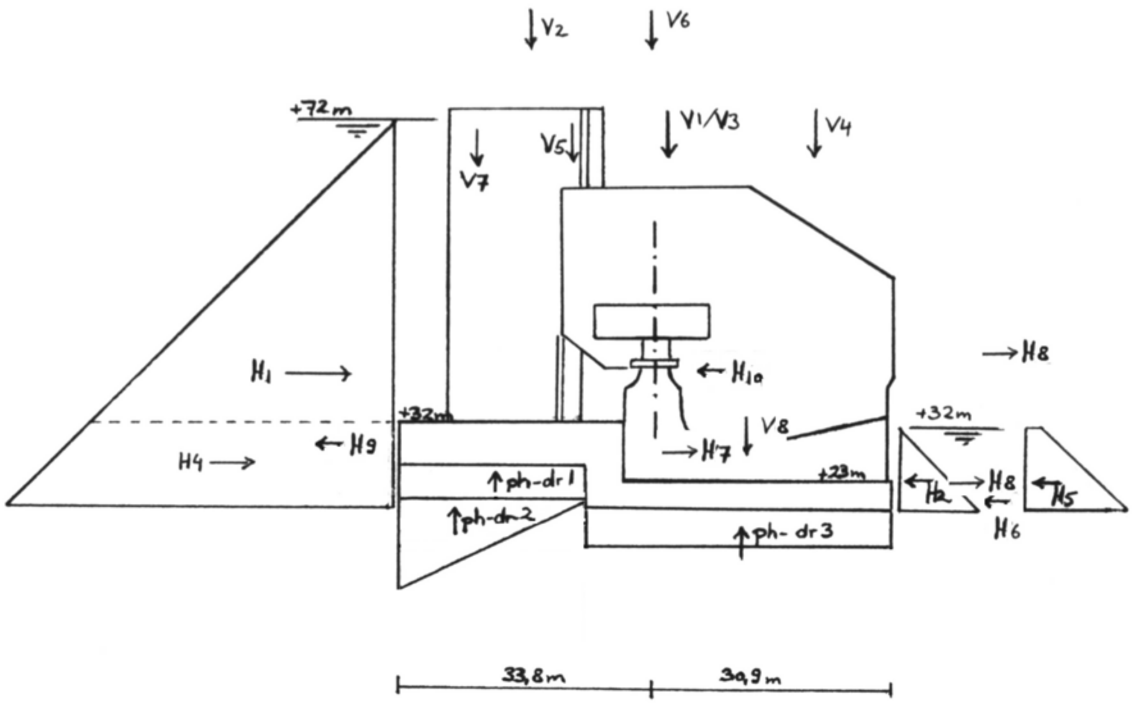


Figure 21: Loads on the power house

6 STABILITY ANALYSIS OF DAM

In this chapter, a problem analysis is performed. For this, the same failure modes from the last chapter (Figure 6) will be discussed. Still, the stability of the upstream and downstream slopes of the embankment dam are treated together in one section and horizontal sliding of an embankment dam is not known to have occurred in the past, therefore this is not discussed any further. Concerning the power house, the failure of the concrete structure is not analysed here, as there is no data available and the other problems have been observed, which appear far more serious. A short reflection on the downstream apron is added as it can cause concern if the uplift water pressures rise without introducing compensating measures.

Several incidents have been recorded by the monitoring systems of the Plavinas dam. In the past, some preliminary analyses have been carried out, see Appendix 10. Figure 22 presents areas of concern.

6.1 EMBANKMENT DAMS

6.1.1 Stability of right embankment dam

No detailed information on the geometry of the embankment dams is given. Still, an indication of their geometry can be obtained by using the plan view of Appendix 7. The embankment dams have survived an earthquake in 1976 and are still in place. The main concern in the right embankment dam is the encountered seepage path and a reduced density as a result of washout of particles.

The relief wells in the right embankment dam reduce the pressure in the sandstone layer in order to keep the pore pressure in the overlying till above low (feed of groundwater due to anomaly). This increases the effective pressure of the soil to gain more bearing capacity against sliding of the right bank. In case sliding occurs of the right bank, the reservoir water will flow towards the tailwater area, leading a flood in downstream direction and washout of soil, initiating failure of the embankment dam. Without relief wells, the uplift forces in the sandstone layer may lift particles in the sandy loam layer, allowing large seepage paths to form.

6.1.2 Stability of the left embankment dam

No data is provided on the piezometric levels and possible drainage systems. Therefore the left embankment dam appears to not be an object of concern and no further investigations are conducted.

6.1.3 Piping through the right embankment dam

The embankment dams are provided with grout curtains, reaching through the dolomite layers, probably extending some meters in the sandstone layer. In close vicinity of the powerhouse, no dolomite layer is found. Here, the embankment dam consists of sandy loam material on top of the Amata and the Gauya sandstone. The longitudinal section along the dam in Appendix 3 is assumed to continue unchanged in downstream direction. The grout curtain is assumed to possess a connection with the power house and to follow the embankment dam.

The right bank lies in an area with an anomaly, where the sandstone layer feeds into the overlying material which is made possible due to flow along fissures in the rock formation.

The fissures may originate from a weak and eroded boundary of the sandstone due to past glacial activities.

A concentrated seepage path was detected in the right embankment dam. This seepage path appears to originate from a crack in the Amata sandstone. The seepage path is assumed to have originated in loose areas of the interface between the Amata sandstone with the Plavinas dolomite (possibly due to dissolving of dolomite) leading to continuous washout of fines towards the interface of sandstone and sandy loam more to the left. The presence of a high gradient of 40m/240m=0,167 at the location of the seepage path may indicate that a slow process takes place where the weakly cemented fines in the sandstone / dolomite and of overlying loamy material are washed out. The rules of Bligh and Lane for the calculation of the required length of the seepage path are not applicable for cemented soil.

The installation of relief wells may have increased the hydraulic gradient toward fissures in the Amata sandstone and promoted internal erosion. A zone of concentrated seepage is detected on the downstream reach at the right flank area, see Appendix 7 and Figure 22. Piping in the Amata horizon rock occurs, not only due to mechanical removal of fines from the near-filter zones of the wells, but also due to continuous suffosional removal (transport of fine soil particles with the groundwater flow) of material through the concentrated seepage paths. The way of concentrated seepage is dated for a crack. Increased fracturing of the rock mass in the zone of influence of the crack has caused increased karstings. Greatest karstings are marked in a zone of the near surface and in a zone of influence of a crack.

It is assumed that the high relief wells at the right bank intercept water from the fissures of the sandstone to prevent uncontrolled discharge with possible development of sand boils and to reduce the pore pressures in the fill material in the area of the anomaly. The discharge of the relief wells has been monitored starting in 1972. The cumulative discharge of particles over the last thirty years is estimated to be 173 m^3 of fines. This is an enormous quantity of washed out particles, causing widely spread areas of reduced density, see Appendix 11. The transport of particles through the pipes of the relief wells must be stopped if the strength of the foundation is not to reach a critical point or intrusion of reservoir water occurs through the seepage path.

A zone of reduced density is assumed to be found in the interface of the sandstone with the till material. Movements in the till material were indicated by an inverted pendulum in combination with the presence of relief wells. The soil underneath the power house appears to be lost into fissures of the rock, thereby moving the inverted pendulum. This loss of material may be the reason for settlements in the right bank area of retaining walls and the right part of the power house structure. Settlement of the structure has caused minor movements between the two blocks of the power plant. The settlement of the power plant blocks are not uniform, the centre and left side have settled about 100 mm, but the right corner has settled about 300 mm. More serious are the differential movements between the power plant blocks and the adjacent wing walls and embankment structures. At the right abutment, the wing wall has settled 300 mm more than the power plant block and the embankment dam even more. All of this can cause damage, but will not directly endanger the overall stability.

A segment of wall close to the high relief wells of the downstream side of the right embankment dam (settlement area, Figure 22) started settling at a considerably increased rate and continues to do so. The fact that the rate of settlement of a segment of wall has not decreased since the early 1990s should be considered. In this particular case, it may mean that the conditions required for the development of arching in the soil (such as ratio of width of area of ground loss to height of soil cover, relative density of soil, effective stress) no longer exist. Therefore settlement as a result of loss of soil volume takes place continuously, instead of sporadically as would be the case if soil arches were to develop or collapse.

Figure 22: Plavinas dam including locations of concern

In order to obtain more detailed information about the extension and location of the crack and areas of less dense material additional explorations are necessary. Geophysical methods (like seismic refraction, which measure sound velocities) should be used to show the extension of less dense zones in the right bank area.

6.1.4 Piping through the left embankment dam

Some anomalies have been observed about 300 m to the left of the concrete structure in the left embankment dam. It is unclear what anomalies these are, but as no further investigations are mentioned, it is assumed that this is negligible. No concentrated seepage and settlements have been reported. The absence of high relief wells may be the reason for the absence of leakage and washout of fines through the embankment dam.

6.2 CONCRETE POWER HOUSE WITH WORKING DRAINAGE WELLS ACCORDING TO DESIGN

In this section the stability of the power house is investigated. Appendix 15 shows the analysis with drainage wells working according to the design assumption that the uplift pressures are reduced to approximately 40kN/m^3. Appendix 15 a, b and c treat the situation without earthquake, with horizontal and vertical earthquake respectively.
The weight of the structure was determined in Appendix 15 section 1. The used uplift water pressures were treated in section 5.4.3.

6.2.1 Horizontal slide of concrete power house

In this section, the stability of the power house will be examined taking into account the unusual drainage system underneath the structure to reduce the uplift forces. Appendix 15 section 10 shows the stability analysis of the power house with horizontal sliding.

The friction between the structure and the foundation soil must be larger than the sum of the horizontal forces (H). The friction is determined by the product of the friction coefficient (f) and the sum of the vertical forces (V):

$$n \cdot H < f \cdot V$$

with

$$f = \tan\delta$$
$$\delta = \frac{2}{3}\phi \; \textit{slide surface soil} - \textit{concrete}$$
$$\textit{or}$$
$$\delta = \phi \quad \textit{slide surface soil} - \textit{soil}$$
$$n : \textit{safety factor}$$

The weight of the power house is spread over foundation slab beneath the power house. The width of the slab is 64,7 m and the effective width over which the loads are transferred to the underground are slightly smaller (see Figure 23) with:

$$B_{ef} = B - 2 \cdot e_B$$

where

B = the entire width of the foundation slab
e_B = the eccentricity of the resulting vertical force

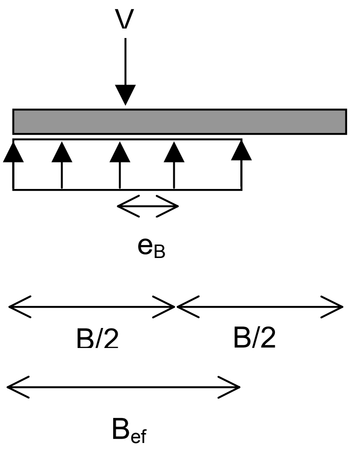


Figure 23: Effective width foundation slab

Additional friction forces from aprons are taken into account. Furthermore, it is assumed that a horizontal slide will not pass along the concrete, but through the subsoil. The reason for this is the irregular shape of the base of foundation.

The safety coefficients against sliding in case of a working drainage are summarised below.

Table 7: Safety factors power house in case of working drainage wells

Situation	no earthquake (*with* load factors)	0,2g hor. earthquake (*no* load factors)	0,2g vert. earthquake (*no* load factors)
Safety factor against horizontal sliding	2,96	1,70	7,16

6.2.2 Failure foundation power house

The shear stresses in the foundation soil can exceed the bearing capacity of the foundation soil, which leads to circular or non circular slip planes through the soil, squeezing the weak layers aside. The Dutch norm TGB 1990 suggests the method Brinch Hansen to determine the maximum strength of the foundation soil. This method is used to determine the bearing capacity of the soil and is explained below.
To determine the strength of the foundation soil under the power house, the drained situation is assumed as the structure has been in place for over thirty years. The bearing capacity of the foundation soil can be approximated by:

$$\sigma'_{\max;d} = c'_{e;d}\, N_c s_c i_c + \sigma'_{v;z;o;d}\, N_q s_q i_q + 0{,}5\gamma'_{e;d}\, B_{ef} N_\gamma s_\gamma i_\gamma$$

with:

$$N_c = (N_q - 1)\cot\phi'_d$$
$$N_\gamma = 2(N_q - 1)\tan\phi'_d$$
$$N_q = e^{\Pi \tan\phi'd}(\tan(45° + 0{,}5\phi'_d))^2$$

$$s_c = \frac{s_q N_q - 1}{N_q - 1}$$

$$s_q = 1 + \frac{B_{ef}}{L_{ef}} \sin\phi'_d$$

$$s_\gamma = 1 - 0{,}3 \frac{B_{ef}}{L_{ef}}$$

For $F_{s;h;d}$ parallel to B_{ef}:

$$i_c = \frac{i_q N_q - 1}{N_q - 1}$$

$$i_q = (1 - \frac{0{,}70 F_{s;h;d}}{F_{s;h;d} + A_{ef} c'_{ef} \cot\phi'_d})^3$$

$$i_\gamma = (1 - \frac{F_{s;h;d}}{F_{s;h;d} + A_{ef} c'_{ef} \cot\phi'_d})^3$$

and:

$\sigma'_{max;d}$ = design value of the maximum stress on the effective foundation
B_{ef} = effective width of the foundation, determined according to section 6.2.1
L_{ef} = effective length of the foundation
$F_{s;v;d}$ = design value of the vertical load, perpendicular to the plane of the foundation in kN
$F_{s;h;d}$ = design value of the horizontal load, in the plane of the foundation in kN
c'_d = design value of the cohesion in kPa
$\sigma'_{v;z;o;d}$ = design value of the original effective vertical stress in the foundation soil directly under the structure in kPa
γ'_d = design value of the effective specific weight of the soil under the structure in kN/m^3
N_c = bearing capacity factor for the cohesion
N_q = bearing capacity factor for the soil present next to the structure
N_γ = bearing capacity factor for the effective specific weight of the soil under the structure
ϕ'_d = design value for the effective angle of internal friction in °.
s_c = shape factor for the cohesion
s_q = shape factor for the soil present next to the structure
s_γ = shape factor for the effective specific weight of the soil under the structure
i_c = reduction factor for the angle of the load for the cohesion
i_q = reduction factor for the angle of the load for the soil present next to the structure
i_γ = reduction factor for the angle of the load for the effective specific weight of the soil under the structure

A schematisation of the foundation soil under the power house is given in Figure 14 and Appendix 3. One of the two power house blocks is founded on sandy loam, which is assumed to posses hardly any cohesion (0-2 kPa), see 5.1.2. Therefore, the first term in the formula of Brinch Hansen is negligible.
The calculation in Appendix 15 section 10.3 was simplified by assuming only one layer of sandy loam under the powerhouse. The depth of influence is larger than the thickness of the layer of sandy loam. The presence of the sandstone layer and presence of loam at the left side of the plant (with a higher angle of internal friction) were neglected, but are assumed to have a favourable influence on the stability.

In the calculation the aprons were assumed not to prevent a slip circle as the connections with the foundation of the power house appear not to be rigid. The effective length of the foundation was assumed to be the entire length across the stream. The calculation was performed for the standard loading case for the power house including load factors. Appendix 15 b and c analyse the situation for the design earthquake without using load factors, as this is an exceptional situation.

The calculation in Appendix 15 section 10.3 has provided maximum values, which are allowed according to Brinch Hansen in order to prevent sliding through the foundation soil. The results are summarised in Table 8.

Table 8: Allowed and existing pressure under power house

Situation	no earthquake with load factors (kN/m^2)	hor. Earthquake 0,2g, no load factors (kN/m^2)	vert. 0,2g earthquake no load factors (kN/m^2)
$\sigma'_{max\ allowed}$	2003	1834	3955
$\sigma'_{existing\ under\ power\ house}$	988	1078	842

6.2.3 Toppling

In order to prevent toppling (turning over) the resulting load should be located close to the centre of the foundation.
The following formula demands that the arm of the total resulting load (T/V) should remain within the core of the foundation slab to keep the foundation under pressure ($1/6*B_{ef}$):

$$\frac{T}{V} \leq \frac{1}{6} B_{ef}$$

with:
T = total moment on foundation slab (kNm)
V = resulting vertical load on the foundation slab (kN)
B_{ef} = effective width foundation slab (m)

In case of a working original drainage, the power house is not endangered by toppling, see Table 9. The resulting force stays well within one sixth of the effective slab width, even in case of a design earthquake (no use of load factors).
Support from the sides of the power house is taken into account, see Appendix 15 section 5.

Table 9: Allowed and actual distance resulting force from middle effective width

Situation	no earthquake with load factors	hor. earthquake 0,2g, no load factors	vert. 0,2g earthquake no load factors
allowed: $1/6B_{ef}$ (m)	10,23	10,18	10,18
existing: T/V (m)	5,63	4,84	4,77

6.2.4 Piping under the power house

The left side of the power house is founded on loam with gravel, whereas the right side is founded on sandy loam. The permeability of sandy loam on the right side is higher than the foundation soil consisting of loam on the right side. Still, the soil apparently is of very low permeability as only the aprons provide the existing length of the seepage path for the present gradient.

Discharge of water from beneath the structure is possible through weep holes in the downstream apron, which are assumed to be located at the end of the downstream apron in a large filter layer, see Appendix 13. Still, sand discharge was observed half way between the power house and the end of the downstream apron. It is assumed that the sand was discharged with drainage water from the drains beneath the power house. However, the downstream area should be closely monitored in order to find out whether the drainage wells are the only source of particle washout.

Inverted pendulums were used to monitor relative lateral displacement between the power house and the foundation soil. Movements of the subsoil were detected (not of the structure). It can be said with reasonable certainty that ground is being lost in downstream direction and possibly to the left of the pendulum, which is located at the right side of the concrete structure, see Figure 22.
Reducing the uplift water pressure beneath the power house by using relief wells (current situation) helps to increase the friction forces due to a larger effective weight to keep the structure from sliding and toppling. Still, using drainage wells underneath the structure has the disadvantage, that the gradient towards the power house is in the till material is increased significantly, which promotes the loss of soil.
The observations indicate that piping takes place under the right side of the power house. This appears to be confirmed by settlements in this area, which become less intense to the left of the structure. The origin of this internal erosion is believed to be the drainage system under the power house. A rise in uplift water pressure, which has been observed under the upstream apron, seems to confirm this thesis, as it could be the result of blocked drains.

6.2.5 Downstream apron

The effective weight of the downstream apron is strongly dependent upon the drainage wells under the concrete power house. If the pressure rises above the design pressure of 40 kN/m^3, the effective weight drops quickly and lifting of the downstream apron followed by failure of the entire dam due to washout and lack of support may occur. Therefore the rise in uplift water pressure mentioned in section 3.1.1 needs to be compensated in order to keep the apron in place.

6.3 CONCRETE POWER HOUSE AFTER FAILURE DRAINAGE WELLS

The stability of the power house is investigated for the situation when the drainage wells fail to function and higher uplift water pressures develop under the power house and the aprons. This situation is of interest as an increase of uplift water pressures was reported with 0,2 m water column/year[25]. Still it is not certain that this process continues slowly in time.
The rise of water pressure was detected beneath the upstream apron, indicating that the drainage system under the power plant is losing effectiveness, probably due to partial blockage of drains. The increase in pressure is not assumed to come from the deeper sandstone layer. First, the layer is a closed aquifer, second the piezometric level is not high enough to be of a large influence in the upstream area. Here, the uplift pressures are dominated by the uplift water pressures from the reservoir.
A rise in uplift pressures reduces the safety of the structure against sliding (horizontal or deeper slip plane) through the foundation soil. This threat can not be eliminated easily. The drains can not be reached, maintained or replaced when blocked because the vertical drains are incorporated into the foundation slab under the concrete power house, see Appendix 1.
Appendix 16 shows the stability analysis in case of a complete failure of the drainage wells, whereas Appendix 17 shows the stability analysis in case of rising uplift pressures. The uplift water pressures in case drainage wells are no longer operational are shown in Appendix 14.
The weight of the structure remains the same as in section 6.2. The uplift water pressures for the situation without drainage wells under the power house were treated in section 5.4.2.
This section does not treat piping under the power house again, as this has been done in section 6.2.4.

In this section, the following situations are analysed:

- an entirely failed drainage system (Appendix 16)
- the uplift water pressures have risen starting in 1976 to an estimated value in the year 2002 of:
  $0{,}2 m / year \cdot 10 kN/m^3 \cdot 26 years = 52 kN/m^2$.
  The total uplift pressure under the power house is thereby assumed to be 40 kN/m^3 + 52 kN/m^3 = 92 kN/m^3 (Appendix 17)

6.3.1 Horizontal slide of concrete power house

The capacity of the power house to withstand horizontal sliding is analysed in Appendix 16, section 10.1 for large uplift pressures in case of complete failure of the drainage system. Appendix 17, section 10.1 analyses horizontal sliding in case of the observed rising uplift pressures.
The method to calculate the safety against horizontal sliding was described in section 6.2.1.

The safety coefficients against sliding are shown in Table 10. In case the drainage wells have failed entirely only the safety without earthquake loading is presented, as already for the standard loading case the structure is unstable. The case in which the drainage wells work less effectively than in the design assumption, the horizontal earthquake load is taken into account.

[25] Norwegian Geotechnical Institute, *Daugava River, Latvia Dam Safety Improvement Studies, 970009-3, Stability Evaluation of the Plavinas HPP, Latvia.* Oslo: October 1997

Table 10: Safety factors power house in case of working drainage wells

	Entire failure drainage wells	Rise of uplift pressure to 92 kN/m^3	Rise of uplift pressure to 92 kN/m^3, hor. Earthquake
Safety factor against horizontal sliding	0,43	1,98	1,38

6.3.2 Failure foundation power house

The bearing capacity of the foundation soil in case the drainage system loses its efficiency is determined in this section. The method of calculation was presented in section 6.2.2.

The calculation in Appendix 16 and Appendix 17 section 10.3 provide the maximum allowed values for the effective soil stresses for which no sliding of the structure through the soil occurs. The results are summarised in Table 11.

Table 11: Allowed and existing pressure under power house

	Entire failure drainage wells	Rise of uplift pressure to 92 kN/m^3	Rise of uplift pressure to 92 kN/m^3, hor. earthquake
$\sigma'_{max\ allowed}$ (kN/m^2)	0	1548	1377
$\sigma'_{existing\ under\ power\ house}$ (kN/m^2)	773	950	1047

6.3.3 Toppling

In case the uplift water pressures continue to rise, the power house is endangered by toppling. However, the resulting load apparently still remains within one sixth of the effective slab width (assuming that the current pressure is approximately 92 kN/m^2), even in case of a design earthquake (without load factors).
Support from the sides of the power house is taken into account, see Appendix 16 and Appendix 17, section 5.

Table 12: Allowed and actual distance of resulting force from middle effective width

	Entire failure drainage wells	Rise of uplift pressure to 92 kN/m^3	Rise of uplift pressure to 92 kN/m^3, hor. earthquake
allowed: 1/6B_{ef} (m)	9,03	10,10	10,09
existing: T/V (m)	14,59	6,34	5,29

6.3.4 Downstream apron

The effective weight of the downstream apron is strongly dependent upon the drainage wells under the concrete power house.
In this analysis, the uplift pressures under the downstream apron are much higher than in case the drainage wells under the power house work according to the design assumption. If the drainage system fails entirely, the resulting force on the downstream apron is negative, implying that it is lifted in this loading case, see Appendix 16 section 9.

6.4 CONCLUSION

In the previous sections, the following has become clear.

- Internal erosion through the right embankment dam causes considerable washout of particles. This process should be stopped to prevent further development of seepage paths, which may threaten the stability of the embankment dam.
- Internal erosion under the power house apparently originates in the drainage wells underneath the concrete structure. Replacement of these drains appears not to be possible as they are incorporated into the foundation of the power house slab. Therefore other means than the existing drainage wells should be found to keep the structure in place with as little washout of particles as possible.
- In case the drainage system continues to become less effective and large uplift pressures develop under the plant, the downstream apron will be lifted. The apron will need more effective weight to stay in place.
- In case the drainage system continues to become less effective and large uplift pressures develop under the plant, the structure will lose its safety against sliding and toppling. The power house will need more effective weight to remain stable.

7 ANALYSIS OF STABILISING ALTERNATIVES

When discussing stabilising possibilities for the Plavinas dam, one should bear in mind that there are several constraints named by Latvenergo, which must be taken into account when designing the stabilising program:

- The power plant cannot be completely shut down for the purpose of rehabilitation work. At least half the number of the turbines must remain in operation at any time. This means that the work has to be carried out under a pressure head of 40 m. During flood periods the entire power station must be in operation and possibly also the spillway, which is located over the power house.
- The drainage layers below the power plant and upstream apron are of vital importance for the stability of the plant. Any disturbance of these layers may endanger the overall stability. Disturbed layers cannot be restored. The drainage system is somewhat different under the left and right power house blocks. Any type of grouting activity must not impair the proper functioning of these drainage layers and wells.
- There are considerable artesian pore water pressures in the deeper soil and rock layers beneath the power house. Due to this situation there may exist some soil layers where the effective soil pressures are nearly zero. These lenses may easily liquefy. Methods causing strong vibrations should therefore be given very careful consideration.
- The working space available in the basement of the power house is very limited. Access to the inside of the power house is through galleries.

In this chapter, an analysis of stabilising alternatives is performed for the right embankment dam and the power house (the left embankment dam is not considered, see section 6.1.2 and 6.1.4).

7.1 TREATMENTS FOR THE RIGHT EMBANKMENT DAM

Below, some general suggestions for treatments for the right embankment dam are given. More detailed suggestions are not performed due to lack of information on the dam.

7.1.1 Prevent further internal erosion

It is assumed that the high relief wells at the right bank intercept water from the fissures of the sandstone to prevent uncontrolled discharge with possible development of sand boils and to reduce the pore pressures in the fill material in the area of the anomaly.
Using drains in the direct area of the power house should be abandoned as it increases internal erosion. If required, other options should be considered:

- Pumping at some distance from the right bank should be considered for reducing the uplift water pressure in the area of anomaly on the right bank.
- Detection of a more precise location of fissures and blocking these by using of grout injection or placement of vertical curtains.
- The use of new drains in the loamy sand and sandstone material of the fill dam may provide another solution.

7.1.2 Prevent further settlements

The settlements of the right part of the power house and the retaining walls of the right bank originate in the loss of material through to the nearby relief wells. If the loss of material in this area can be minimised, the basis for future settlements is eliminated. After reducing the washout to a minimum, it is likely that some small settlements continue until the soil has regained its arching capacity. If these settlements are unacceptable, filling of the cavities or should be considered at some locations to provide support.

7.1.3 Conclusion treatment for the right bank

Abandoning the drains on the right embankment reduces the risk of development of further cavities in the dam. The cavities should be sealed to prevent future seepage or settlements. Before the cavities are sealed, the groundwater flow should be diverted to allow the grout to harden.
However, due to insufficient quantitative and spatial information being available, a more detailed solution to the problems in the embankment dam is not pursued.

7.2 STABILISING TREATMENTS POWER HOUSE

It is unclear what lifespan Hydroproject assumed in the design. In general a lifespan of one hundred years is not uncommon for hydroelectric power plants. For the Plavinas dam this time will expire in 2066. This still leaves about 65 years of operation ahead. The uplift water pressures under the plant should be closely monitored, checking the predicted rise of 0,2 m water column/year. However, the pressure build up may suddenly start to increase even more if the drainage system gets blocked entirely. An alternative way to keep the structure in place should be implemented on time, assuming large economic interest in prolonging the lifespan of the dam.
Several treatments can be considered to ensure the stability of the power house when abandoning the original drainage wells under the structure:

- Adding weight to the power house
- Adding weight to the downstream apron
- Adding anchors to the downstream apron
- Extending the upstream apron
- Extending the downstream apron
- Adding vertical curtains
- Lowering the reservoir table
- Installing new drains in upstream apron
- Installing new drains in downstream apron
- Connecting the upstream apron with the power house slab

These treatments are analysed in the following sections.

7.2.1 Adding weight to power house

General

The safety against sliding toppling can be improved if the effective weight (resulting downward load) of the structure is increased. This can be accomplished by adding weight to the structure. There is hardly any space in the power house to store additional encumbrance. The upstream pillars could be extended towards the reservoir, but the foundation slab of the power house does not offer a lot of space for storing additional weight. Also, additional weight due to larger upstream pillars can not be accomplished by wider pillars as this would hinder the flow of the water towards the turbines. The spillway can be used to place additional weight on the power house. Still, the water must find its way from the reservoir to the tailrace whenever the reservoir level reaches the maximum level. Walls on the spillway add weight to the structure and guide the water downstream. A roof on top of the walls would add even more weight see Figure 24.
Adding walls on top of the power house accompanied by a roof on the horizontal part of the spillway increases the downward load on the foundation soil and provides more bearing capacity against sliding (horizontal and deep slip planes) and toppling.

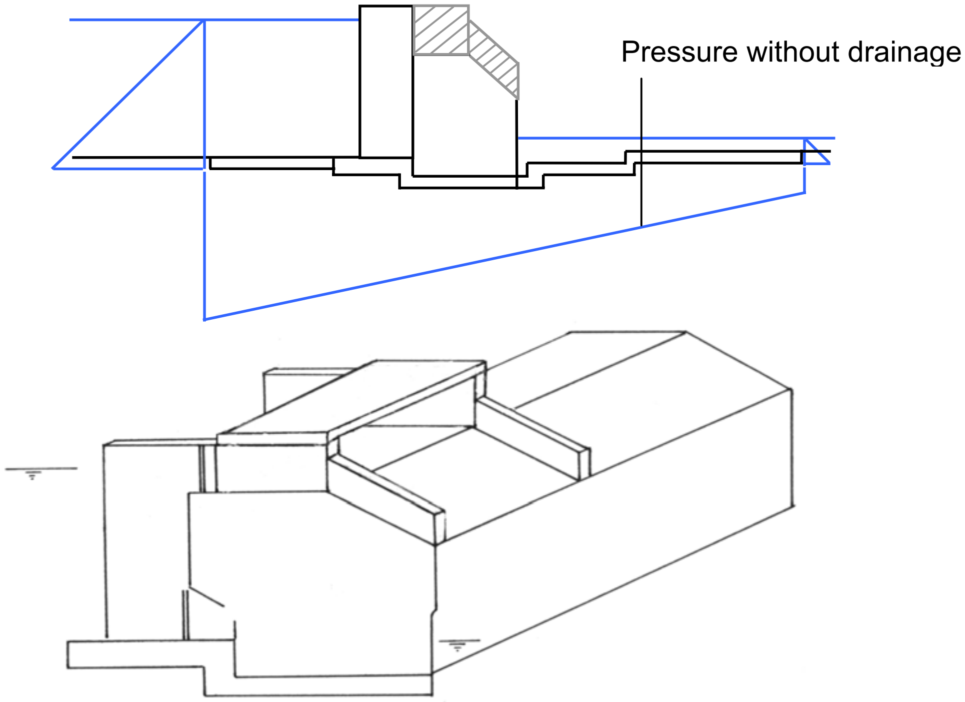


Figure 24: Walls on the spillway

Stability
The stability of the power house with additional weight is analysed in Appendix 18. Walls 6 m wide and 15 m and 10 m high on the horizontal respectively sloping part of the spillway combined with a roof of 5 m on top of the walls on the horizontal part of the spillway were implemented in the calculation.
Table 13 shows the results of the stability analysis for additional walls and provides a comparison with the situation in case the drainage system under the power house has failed entirely and the case in which the original design drainage is functional. This alternative does not provide sufficient stability for the power house. The results for an additional earthquake load will be even less favourable. The weight on top of the spillway can not compensate sufficiently for the large uplift water pressure. Also, the additional weight on the power house does not solve the problem of uplift of the downstream apron.

Table 13: Stability for additional weight on powerhouse, no earthquake

Situation	n hor. slide (-)	1/6B_{ef} (m)	T/V (m)	$\sigma'_{max\ allowed}$ (kN/m^2)	σ'_{max} existing under power house (kN/m^2)
Additional weight on power house, no drainage, no earthquake	0,55	11,05	10,46	74	706
No treatment, no drainage no earthquake	0,43	9,03	14,59	0	773
Drainage wells working according to design assumption, no earthquake	2,96	10,23	5,63	2003	988

Implementation
When placing concrete walls on top of the spillway, a connection is necessary between the existing concrete structure and the new walls. Pins of steel may be drilled into the concrete so that the walls will have a strong connection with the spillway. This is especially the case for the descending part of the spillway. The walls could consist of reinforced concrete, filled with sand to minimise the amount of expensive concrete. When building the walls, only part of the spillway will be left to discharge water downstream. The area over which the water can be spilled after completion will be reduced to 183m-10*6m=123m.

Remark
This option hardly changes the stability of the power house compared to the situation in which the drainage wells under the plant have failed. Even when combining this alternative with another, its contribution will be very small. Therefore, this alternative is abandoned

7.2.2 Adding weight to downstream apron

General
Adding weight to the downstream apron by placing concrete walls along the apron, see Figure 25 increases the effective downward load. This allows the apron to stay in place for rising uplift pressures. The additional weight compensates for the uplift forces under the power house and the effective weight on the apron results in friction forces, which point in the upstream direction and increase the stability of the structure.
The aprons will be provided with ten large walls, while the other loads remain unchanged.

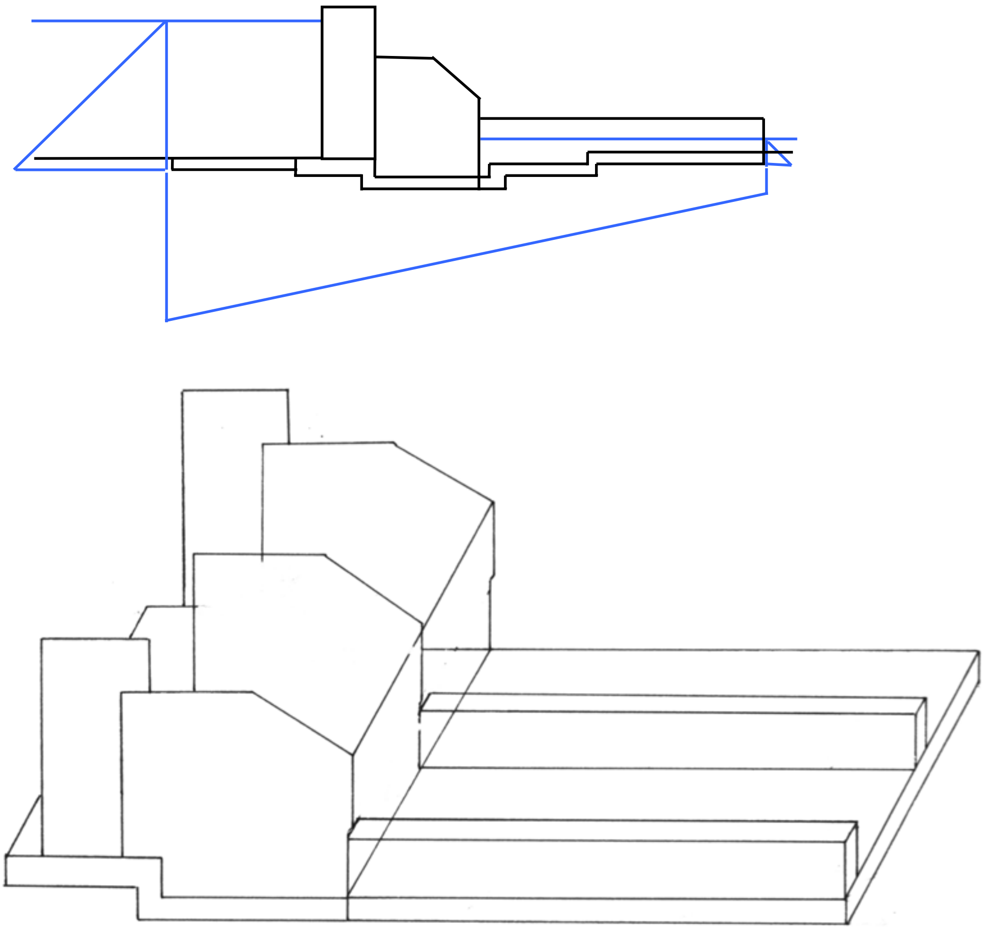


Figure 25: Walls on the downstream apron

Stability

The walls can significantly increase the resistance of the power house to horizontal sliding and slipping through the foundation soil. The additional weight mobilises friction forces under the apron, which results in support of the power house foundation slab. Appendix 19 shows the placement of additional walls 9 m wide walls of 25 m respectively 21 m height (jump in downstream apron causes difference in height wall) in section 6 and section 10 analyses the stability.

Table 14 shows the results of the stability analysis and provides a comparison with the situation in case the drainage system under the power house has failed entirely and the case in which the original design drainage is functional. This alternative does not provide sufficient stability for the power house when no earthquake is taking place. The results for an additional earthquake load will only be less favourable. Toppling of the power house remains a problem as the supporting load under the apron concentrates near the lowest point of the power house.

Table 14: Stability for additional weight to downstream apron

Situation	n hor. slide (-)	$1/6B_{ef}$ (m)	T/V (m)	$\sigma'_{max\ allowed}$ (kN/m^2)	σ'_{max} existing under power house (kN/m^2)
Additional weight on downstream apron, no drainage, no earthquake	1,44	9,03	13,40	1011	734
No treatment, no drainage no earthquake	0,43	9,03	14,59	0	773
Drainage wells working according to design assumption, no earthquake	2,96	10,23	5,63	2003	988

Implementation

The distance between the draft tubes measures approximately 18 m. The concrete walls were assumed to be 9 m wide, filled with earth or gravel and closed on top with a concrete slab, leaving half of the downstream area free for the river discharge. Still, the walls are gigantic and may cause problems due to the considerable weight on the downstream apron. Settlements or cracks in the underlying concrete slab may occur.

In order to place to walls on the downstream apron, prefabricated elements are an option. Construction works while half of the turbines are in operation will make the execution complicated. Prefabricated elements could imply caisson type walls. These would need support to float them through the shallow water of the downstream apron. Another option the transport of T-shaped elements towards the apron.

Remark

The additional weight on the apron clearly increases the stability of the structure. As the effect is not entirely satisfactory, this alternative may be combined with another treatment.

7.2.3 Adding anchors to downstream apron

In order to keep the downstream apron in place, anchors or piles may be installed, see Figure 26.

The subsoil consisting of sandy loam has a very low cohesion of no more than 2 kPa, whereas the effectiveness of anchors and piles originates in the cohesion of the soil. This means that the downstream apron would need an enormous number of anchors.

When installing the anchors, a large number of holes in the concrete slab are required, through which piles would need to be formed. This would also mean perforating the apron and possibly reducing the seepage path of the water under the plant.

This option is abandoned due to lack of effectiveness.

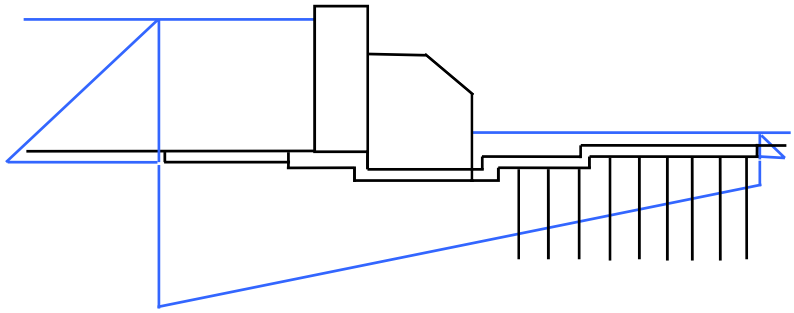


Figure 26: Anchoring the downstream apron

7.2.4 Extending upstream apron

General

Extending the upstream apron provides a significantly longer seepage path. This reduces the gradient and brings the boundary condition of large uplift loads further away from the power house. Therefore the uplift pressure under the power plant will be reduced, leading to a larger effective weight of the structure. An extension also reduces the pressure under the downstream apron, which needs to be kept in place. This extension needs to consist of an impermeable layer to place the boundary condition of large uplift forces further from the power house. This implies that also the area at the sides of the apron needs to be impermeable, see Figure 27.

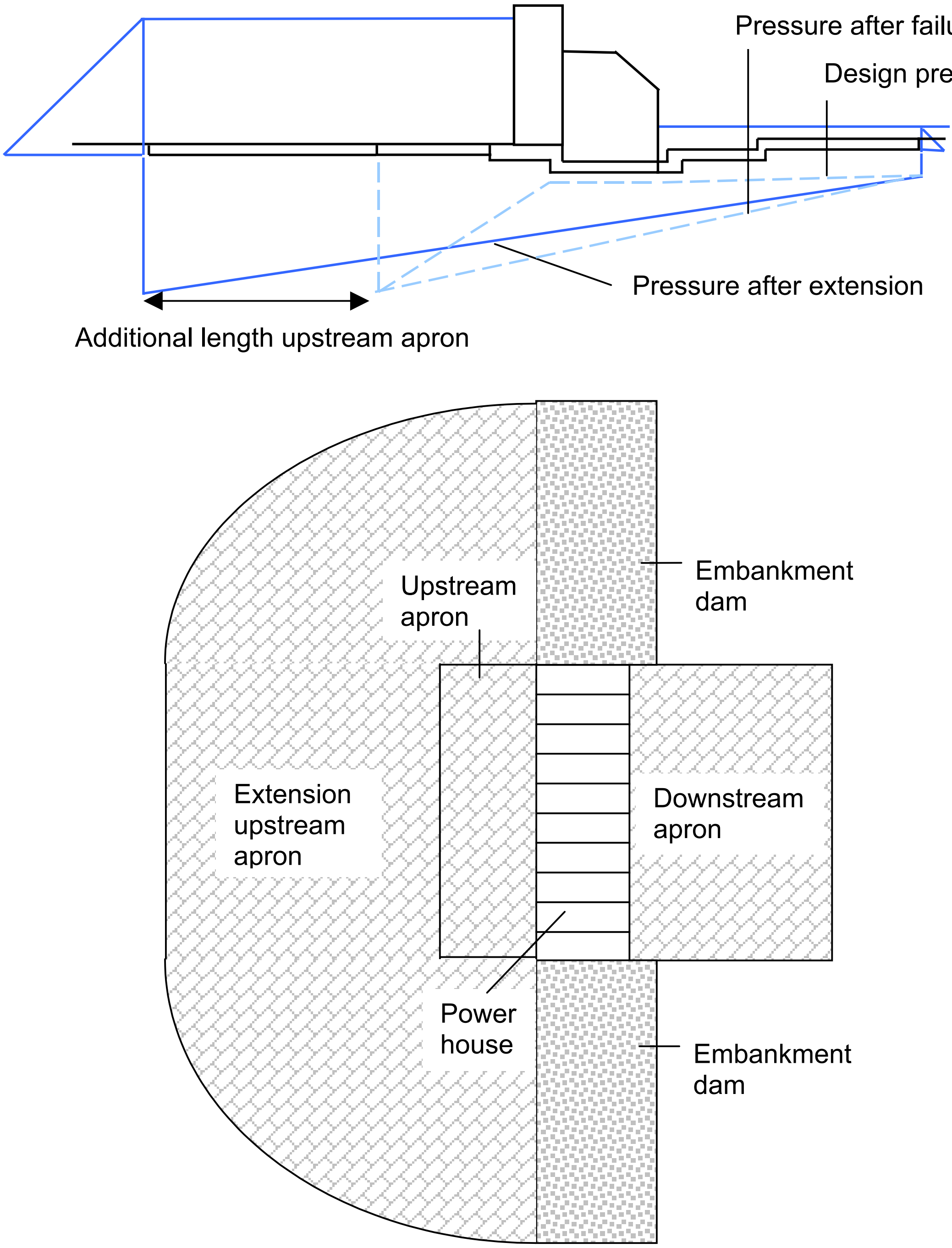


Figure 27: Pressure without drainage, with extension of the upstream apron

Stability
In order to provide sufficient stability for the structure, the upstream apron needs to be extended for hundreds of meters in the upstream direction. Appendix 20 treats the situation of an extension to 1000 m, see section 4 and 10. The bearing capacity of the foundation soil is not large enough. In order to achieve a better resistance, the upstream apron needs to be extended even further.

Table 15: Stability for extension upstream apron

Situation	n hor. slide (-)	1/6B_{ef} (m)	T/V (m)	$\sigma'_{max\ allowed}$ (kN/m^2)	$\sigma'_{max\ existing\ under\ power\ house}$ (kN/m^2)
Extension of upstream apron, no drainage, no earthquake	1,16	9,47	7,43	816	958
No treatment, no drainage no earthquake	0,43	9,03	14,59	0	773
Drainage wells working according to design assumption, no earthquake	2,96	10,23	5,63	2003	988

Implementation
The upstream apron could be extended by placing rock material on the bottom of the reservoir and subsequently penetrating this with asphalt or concrete. Another option rolling impermeable membranes on the bottom and allowing overlap. This is a complicated task as the reservoir is approximately 40 m deep and the equipment needs long arms to either roll the membrane in place or pump the asphalt or concrete at the bottom. The result should be checked by sending divers to the bottom of the reservoir. This is difficult due to the large water depth of the reservoir. Lowering the water level only facilitates the execution if this is done drastically. This can only be done slowly to prevent landslides and allow the other power plants further downstream to cope with the additional amount of water. Also, the operator of the power plant does not wish to lower the water table in order to continue with the production of electricity.

Remark
This option increases the stability of the structure. As the effect is not entirely satisfactory and the placement of a membrane in a large area is expected to be extremely expensive. This alternative may be combined with another treatment.

7.2.5 Extending downstream apron

Extending the downstream apron will reduce the gradient under the power house, but the uplift water pressure is not reduced. This implies that an extension of the downstream apron does not add effective weight to the power house. Therefore this option presents no alternative to increase the stability of the concrete structure and it will not be investigated further.

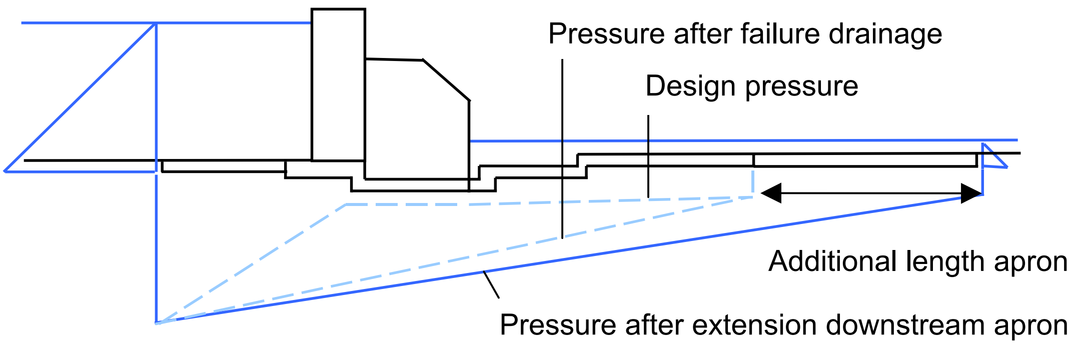


Figure 28: Extension of downstream apron

7.2.6 Adding vertical curtains

General

Adding vertical curtains into an impermeable layer under the structure to create low pressures from the downstream side under the structure is not possible. The present sandstone layer at the bottom of the old glacial valley lies extremely deep. Furthermore the impermeability of this layer is questionable. Placing vertical curtains in the foundation soil (Figure 29) will reduce the gradient and the uplift pressures. Lengthening the vertical seepage path causes an additional distance for the water to travel of twice the length of the curtain.

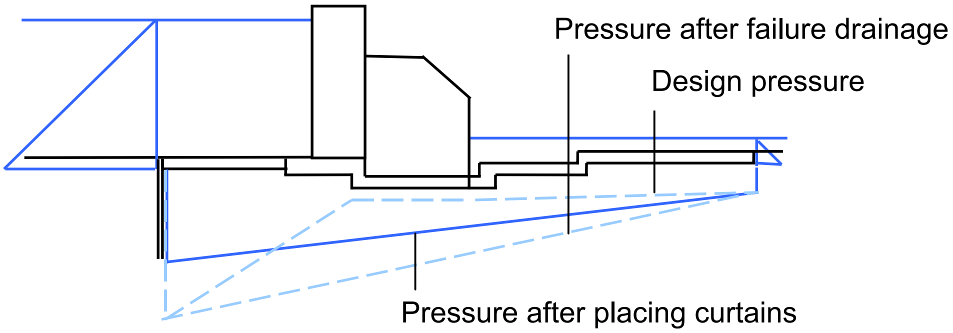


Figure 29: Curtains at the end of the upstream apron

Stability

In Appendix 21 the stability analysis was performed for a vertical curtain of 30 m at the upstream end of the upstream apron. Table 16 shows that vertical curtains do not improve the stability considerably. In order to reduce the uplift pressures significantly, very deep curtains would be necessary, preferably more than one. This implies that the upstream apron may require an extension.

Table 16: Stability for curtains at end upstream apron

Situation	n hor. slide (-)	$1/6B_{ef}$ (m)	T/V (m)	$\sigma'_{max\ allowed}$ (kN/m^2)	σ'_{max} existing under power house (kN/m^2)
Adding vertical curtains no drainage, no earthquake	0,55	9,18	12,04	67	805
No treatment, no drainage no earthquake	0,43	9,03	14,59	0	773
Drainage wells working according to design assumption, no earthquake	2,96	10,23	5,63	2003	988

Implementation

When placing impermeable curtains at the end of the upstream apron, sheet piles are no option due to the presence of boulders in the till material, leaving the possibility of a curtain formed in the soil. It is a complicated task to create a curtain, which can be considered impermeable, especially under a large water depth.

Remarks

This treatment is not very effective while the execution is difficult. Therefore, this alternative is abandoned.

7.2.7 Lowering of reservoir water table

Lowering the water table in the reservoir, see Figure 30 reduces the horizontal loads on the structure and therefore has a positive effect on horizontal, non horizontal sliding and toppling. Still, in order to improve the stability of the structure, the head difference would need to be reduced significantly. As the head difference is the source of energy for the hydroelectric power plant it should not be reduced. Therefore, this aspect will not be discussed any further.

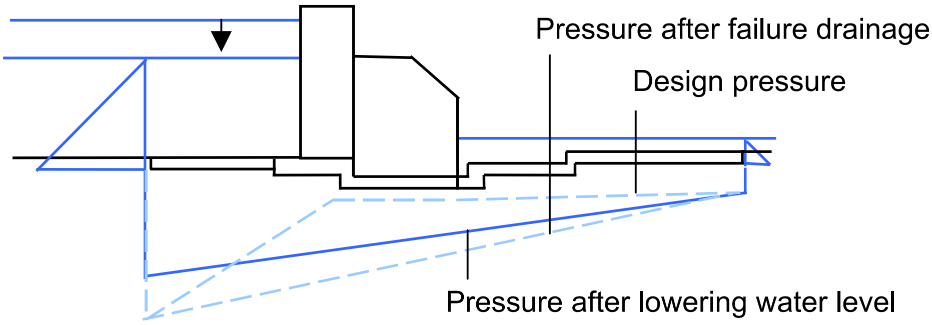


Figure 30: Lowering of reservoir level

7.2.8 Installing new drains in upstream apron

General

The installation of new drains can be considered in order to reduce the uplift water pressure and thus increasing the resulting downward load to obtain a satisfactory safety for the structure. Such drains could be installed in the upstream apron.

New drains in the upstream apron require pumps or a connection to the lower tailwater level in order to achieve a reduction of the pressures. When placing drains in the upstream apron, see Figure 31, the gradient will be larger than nowadays because the horizontal distance over which the pressure is reduced is smaller. This may cause washout of particles, but this would only be the case in the area under the upstream apron and not under the power house. After all, the gradient under the power house will be very small due to the reduction at the upstream side. Some washout under the upstream apron is not expected to cause significant problems for the power plant. Possible seepage paths are not formed under the power house, therefore additional settlements are not expected.

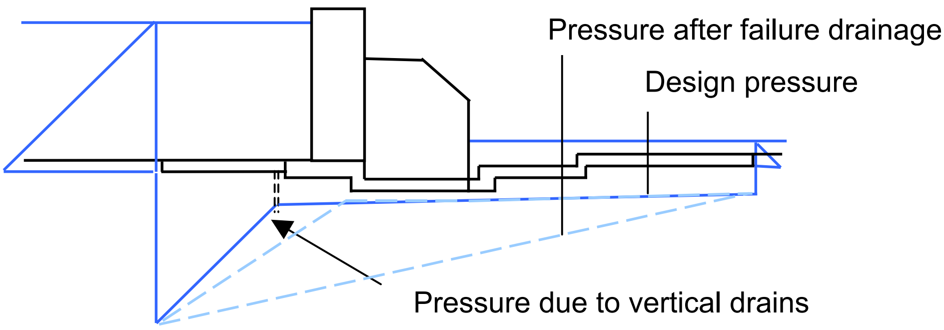


Figure 31: New drains in the upstream apron

Stability
The stability of this alternative is analysed in Appendix 22, section 10. The below table summarises the results for a reduction of uplift pressures under the upstream apron near the power house foundation slab to a value of 40 kN/m^2. This new reduction would not cause larger effective stresses in the subsoil as this alternative does not reduce the uplift water pressures more than the original drainage wells. These reduced the uplift water pressures to 40 kN/m^2 under the power house. The reduction to 40 kN/m^2 under the apron results in uplift water pressures under the power house (which is located 9,5 m deeper than the apron) of approximately 40 kN/m^2 + 9,5 m * 10 kN/m^3 = 135 kN/m^2. The pressures in the foundation soil are slightly larger compared to the design assumption.
Table 17 shows the results of the stability analysis for the new drains for the standard loading case and a horizontal earthquake load compared to the results of no drainage system and the original design pressures.

Table 17: Stability for new drains in upstream apron

Situation	n hor. slide (-)	1/6B_{ef} (m)	T/V (m)	$\sigma'_{max\ allowed}$ (kN/m^2)	$\sigma'_{max\ existing\ under\ power\ house}$ (kN/m^2)
New drains in upstream apron, no earthquake	1,61	9,53	6,54	1248	1026
New drains in upstream apron, horizontal earthquake	1,26	9,68	5,39	1162	1099
No treatment, no drainage no earthquake	0,43	9,03	14,59	0	773
Drainage wells working according to design assumption, no earthquake	2,96	10,23	5,63	2003	988
Drainage wells working according to design assumption, horizontal earthquake	1,70	10,18	4,84	1834	1078

Implementation
In order to reduce the uplift water pressures under the upstream apron, holes need to be drilled through the concrete slab. This is a complicated task because of the large water depth in the reservoir. Even when reducing the reservoir level by half, the depth is considerable and drilling equipment on a boat / pontoon needs to reach the concrete. After drilling through the concrete, the hole needs to be extended for approximately another 10 to 15 m for the drainage pipes. These may be perforated plastic pipes connected to underwater pumps.

Remark
The drilling through the concrete and installation of drains under deep water complicates the realisation of this option, though the stability would be improved drastically. Replacing the drains or the pumps in case of problems is difficult and calls for expensive equipment.

7.2.9 Installing new drains in downstream apron

General

The installation of new drains in the downstream apron can be considered in order to reduce the uplift water pressure and thus increasing the resulting downward load to obtain a satisfactory safety for the structure. New drains in the downstream apron only help, if they are located near the power house as the reduction of pressure is required here.

The gradient caused by the new drains is smaller than in case of the original drainage system under the power house. Still, the groundwater flows from the end of the upstream apron to the downstream end of the power house foundation slab, which implies that if washout takes place this may lead to settlements of the power house structure. However, the smaller gradient suggests that the erosion should not be a bigger problem than for the original drainage system. The uplift pressures and location of the drains should be closely monitored to check for rising pressures of washout. If problems occur, the drains in the downstream apron can be replaced or cleaned (obviously unlike the present drainage wells, where problems apparently can not be compensated).

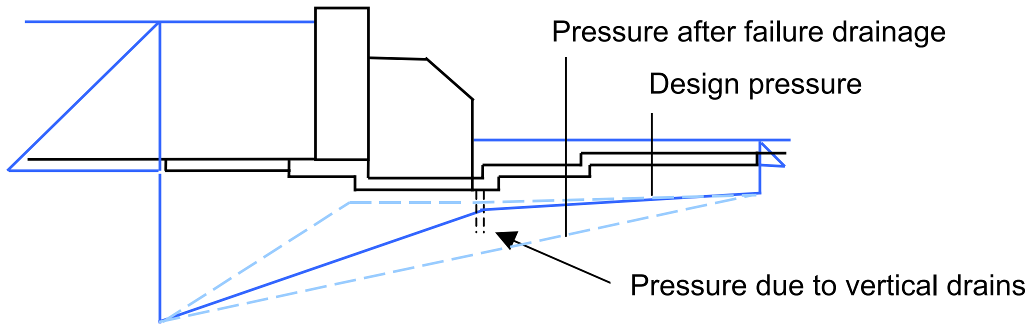


Figure 32: New drains near the power house

New drains close to the power house behind the draft tubes could replace the old drainage system under the power house. The uplift loads under the structure will be reduced towards the beginning of the downstream apron compared to the original situation where the drainage wells reduced the pressures at the upstream end of the foundation slab of the power house.

Stability

Drains at the beginning of the downstream apron increase the stability considerably, see table below. The stability analysis in Appendix 23 section 10 was performed for a reduction of uplift pressure to 100 kN/m^2. When installing drains without pumping system, the uplift water pressure will be reduced to approximately (10 m+3,5 m)*10 kN/m^3 = 135 kN/m^3. When taking into account the design earthquake, this uplift pressure is slightly too large to prevent failure of the foundation soil.

This alternative provides a good bearing capacity against sliding through the foundation soil. The reduction of uplift water pressures and the stresses in the foundation soil are smaller than in the design assumption, see Table 18.

Table 18 shows the results of the stability analysis for the new drains for the standard loading case and a horizontal earthquake load compared to the results of no drainage system and the original design pressures.

Table 18: Stability for new drains in downstream apron

Situation	n hor. slide (-)	$1/6B_{ef}$ (m)	T/V (m)	$\sigma'_{max\ allowed}$ (kN/m^2)	$\sigma'_{max\ existing\ under\ power\ house}$ (kN/m^2)
New drains in upstream apron, no earthquake	1,56	10,40	7,57	1256	811
New drains in upstream apron, horizontal earthquake	1,21	10,28	6,13	1099	933
No treatment, no drainage no earthquake	0,43	9,03	14,59	0	773
Drainage wells working according to design assumption, no earthquake	2,96	10,23	5,63	2003	988
Drainage wells working according to design assumption, horizontal earthquake	1,70	10,18	4,84	1834	1078

Implementation
Holes through the downstream apron are required to place drainage pipes. The connection with the apron needs to be watertight to force soil particles through the drainage pipes.
After drilling through the concrete, the holes need to be extended for approximately another 10 to 15 m for the drainage pipes. These may be perforated plastic pipes connected to underwater pumps. In the downstream area pontoons may be used for drilling through the slab and placing the drainage pipes. The under water pumps may be placed with the assistance of divers.

Remark
This alternative is easier to implement than drains under deep water. Also, the maintenance is facilitated due to the relatively shallow water of an average depth of 10m just behind the power house.

7.2.10 Connecting upstream apron with foundation slab power house

The safety against sliding may be improved if the joint between the upstream apron and the power house foundation slab could transfer considerable tension forces. This may be accomplished by anchoring steel profiles between the slabs and adding a layer of concrete on top, see Figure 33. This way, the friction forces under the upstream apron could be mobilised to resist sliding. The stability of the downstream apron is not improved by this alternative. Also, it is extremely difficult to install metal profiles in the apron under 40m of water. Therefore this alternative is abandoned.

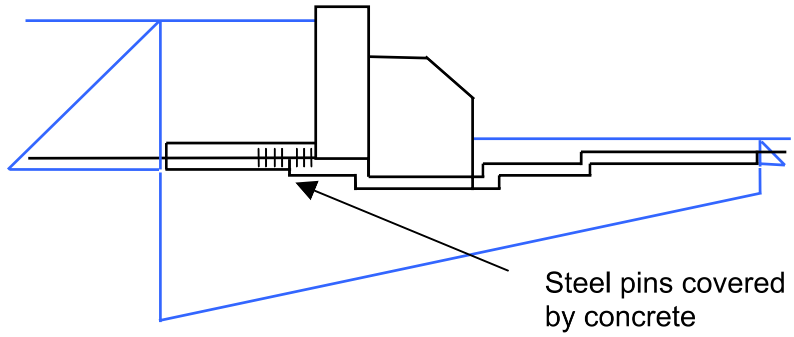


Figure 33: Connection between power house and upstream apron

7.2.11 Reflection

The most promising treatments from section 7.2 are summarised below.

1. Adding weight to the downstream apron by installing walls
2. Extending the impermeable upstream apron into the reservoir
3. Installing new drains in the upstream apron
4. Installing new drains in the downstream apron

Number 1. and 2. can not stabilise the power house sufficiently. A combination however is possible and is analysed in the next section (7.2.12).
Alternative 3. and 4. can stabilise the power house sufficiently.

7.2.12 Adding walls to downstream apron combined with an extension of the upstream apron

General

The combination of walls on the downstream with an extension of the upstream apron stabilises the power house considerably.

Stability

The extension of the upstream apron reduces the gradient and thus the pressures under the structure. Appendix 24 shows the stability analysis for an extension of the upstream apron to 1000 m combined with ten walls on the downstream apron of 12 m and 8 m (due to jump in apron) high and 8 m wide. The walls start approximately 30 m behind the spillway to allow the water to be spilled freely on the downstream apron.
Table 19 shows the results of the stability analysis for this alternative for the standard loading case and a horizontal earthquake load compared to the results of no drainage system and the original design pressures. This alternative stabilises the power house, while keeping the soil pressures slightly lower than in case of the original design assumption.

Table 19: Combination of walls on downstream apron with an extension of the upstream apron

Situation	n hor. slide (-)	$1/6B_{ef}$ (m)	T/V (m)	$\sigma'_{max\ allowed}$ (kN/m^2)	$\sigma'_{max\ existing\ under\ power\ house}$ (kN/m^2)
Walls on downstream apron plus extension upstream apron no drainage, no earthquake	1,80	9,47	7,17	1375	943
Walls on downstream apron plus extension upstream apron no drainage, horizontal earthquake	1,23	9,66	5,74	1102	1031
No treatment, no drainage no earthquake	0,43	9,03	14,59	0	773
Drainage wells working according to design assumption, no earthquake	2,96	10,23	5,63	2003	988
Drainage wells working according to design assumption, horizontal earthquake	1,70	10,18	4,84	1834	1078

Implementation

The placement of an impermeable membrane on the upstream apron is complicated due to the large water depth and the large area, which needs to be sealed. The placement of the walls on the downstream apron may be performed by using prefabricated concrete elements. Caisson type walls should be considered. Still, the height of the walls can be a problem. When transporting walls to the downstream apron, a large part of the caisson is located under the water line due to its weight. The water in the downstream apron is very shallow, so that the caissons will need more floating capacity, possibly provided by pontoons. The upper part of the caissons may also be placed after the structure is sunk in place. Placing other prefabricated concrete elements on the apron may cause difficulties, as crane pontoons can not bear large moments.

Remark

This alternative increases the stability of the structure and the seepage path is extended. Still, the enormous weight on the downstream apron may cause settlements or cracks and the alternative involves more implementation work than the others do.

8 SELECTION OF TREATMENT

In the last section several options for stabilising the Plavinas dam were analysed. In this section, the best alternatives are compared in order to select the alternative, which increases the stability of the power house while keeping the implementation simple and affordable.

The best options to stabilise the power house are:

- new drains in the upstream apron
- new drains in the downstream apron
- an extension of the upstream apron in combination with walls on the downstream apron

8.1 MULTI CRITERIA ANALYSIS

In order to find the most favourable solution for stabilising the power house of the Plavinas dam, the following criteria will be used when comparing solutions:

- Problem elimination
  The treatments needs to improve the stability of the power house, minimise further internal erosion and keep the downstream apron in place.
- Additional problems
  The alternative may cause new problems.
- Durability of the solution
  The treatment should last as long as possible in order to prevent additional stabilisation works during the rest of the lifespan of the structure.
- Feasibility of execution
  The treatment involves construction works, which may be more or less complicated to implement than others.
- Time of realisation
  The period in which the equipment is rented for the implementation of the treatment and personnel is required should be as short as possible.
- Costs of implementation
  The stabilising treatment should be as affordable as possible.
- Continuance of operation
  The operator of the plant wants to prevent the loss of energy production during construction works as this means loss of revenue.

The different alternatives receive scores for each criterion. The score lies between 5 and 0 for the most and least favourable effect, respectively, see Table 20.

Table 20: Multi criteria analysis of remaining alternatives

Alternative / Criteria	new drains in the upstream apron	new drains in the downstream apron	extension of upstream apron in combination with walls on the downstream apron
Problem elimination	• The stability of the structure is amply sufficient. • The maximum stress on the foundation soil is slightly larger than in the original	• The stability of the structure is amply sufficient. • The maximum stress on the foundation is slightly smaller than in the original case.	• The stability of the structure is amply sufficient. • The maximum stress on the foundation is slightly smaller than in the original case.

	case. • The reduction of uplift pressure is achieved under the upstream apron. Therefore no further washout under the power house is expected and future settlements are minimised ➢ Score: 5	• The reduction of uplift pressure is achieved under the upstream apron and the power house. The gradient is smaller than in the original case, therefore washout under the power house will be very limited. ➢ Score: 4	• A smaller gradient is achieved by extending the upstream apron. Therefore the washout will be negligible. ➢ Score: 5
Additional problems	• The underwater pumps need to withstand large vertical loads from the reservoir water. ➢ Score: 3	• The underwater pumps need to withstand the current behind the draft tubes. However, the velocities are not expected to be very large ➢ Score: 5	• The large additional weight on the downstream apron may cause cracks in the concrete if the slab is not sufficiently reinforced. • In case the uplift water pressures start to rise due to a rupture in the extended apron, it is very difficult to find the location and seal it. ➢ Score: 1
Durability of solution	• From time to time maintenance of the drains and pumps will be necessary. This is extremely difficult under the deep water of the reservoir. ➢ Score: 1	• From time to time maintenance of the drains and pumps will be necessary. This would take place more shallow water than in the reservoir. ➢ Score: 3	• This option is assumed to offer a durable solution with little if any maintenance works. ➢ Score: 5
Feasibility of execution	• The implementation of this alternative is complicated due to the large water depth. The precision of the drilling and the placement of the drains and pumps are questionable.	• The drilling and placement of drains can be achieved from pontoons with the guidance of divers. The water is significantly more shallow than in the reservoir.	• A very large area must be sealed by using prefabricated membranes, which are rolled on the bottom of the reservoir or applied by using rockfill, penetrated by asphalt of concrete.

	➢ Score: 2	➢ Score: 4	The large depth necessitates large arms on pontoons. • The walls on the downstream apron are large and difficult to bring in place. ➢ Score: 2
Time of realisation	• The process of drilling and placing the drains and pumps will be slower than in more shallow water. ➢ Score: 3	• The drilling process and the installation of drains and pumps is assumed to take several weeks to months. ➢ Score: 5	• The placement of an impermeable membrane in a large area will presumably take several months • Bringing large walls on the downstream apron will also take several months to construct and place on the apron. ➢ Score: 0
Imple-mentation costs	• Due to the water depth this option will be far more expensive than in more shallow water. ➢ Score: 3	• This option will be less expensive than in the reservoir due to less expensive drilling equipment and a quicker execution. ➢ Score: 5	• The extension of the apron will be an extremely expensive treatment due to the expensive equipment, material and long time of implementation • The construction of walls for the downstream apron takes a long time and an enormous quantity of concrete is involved, leading to high costs ➢ Score: 0
Continuance of operation	• When implementing this alternative, the turbines in close vicinity of the construction works should be closed for operation to prevent strong currents around the working place. ➢ Score: 4	• When implementing this alternative, the turbines in close vicinity of the construction works should be closed for operation to prevent strong currents around the working place. ➢ Score: 4	• When implementing this alternative, the turbines in close vicinity of the construction works should be closed for operation to prevent strong currents around the working place. ➢ Score: 4

When weighing all the criteria with the factor 1, the total score for the alternatives is shown in Table 21.

Table 21: Total scores of alternatives

Alternative	new drains in the upstream apron	new drains in the downstream apron	extension of upstream apron in combination with walls on the downstream apron
Total score	21	30	17

The order of the most favoured alternatives is:
1. installation of new drains in the downstream apron
2. installation of new drains in the upstream apron
3. extension of the upstream apron combined with walls on the downstream apron

8.2 SENSITIVITY ANALYSIS

In order to gain more insight into the sensitivity of the most favourable solution, the criteria are given different weighing factors. The criteria keep the weighing factor 1 except for the one on the same row of the below table.

Example for new drains in upstream apron:

Problem elimination is weighed with factor 2 whereas the other criteria such as durability, feasibility etc keep the factor 1. The total score is obtained in the same way as before:
2*5+1*3+1*1+1*2+1*3+1*3+1*4=26.

Table 22: Sensitivity analysis of alternatives

Alternative / Criteria	new drains in the upstream apron	new drains in the downstream apron	extension of upstream apron in combination with walls on the downstream apron
Problem elimination	26	34	22
Additional problems	24	35	18
Durability of solution	22	33	22
Feasibility of execution	23	34	19
Time of realisation	24	35	17
Implementation costs	24	35	17
Continuance of operation	25	34	21

The results of the weighed sums for the alternative of new drains in the downstream apron show higher values than the sums of the other alternatives.
Therefore, according to the sensitivity analysis, the order of the most favoured alternatives remains unchanged:

1. installation of new drains in the downstream apron
2. installation of new drains in the upstream apron
3. extension of the upstream apron combined with walls on the downstream apron

The sensitivity analysis shows that the order of preference is insensible to the investigated changes in weighing factor.

9 IMPLEMENTATION OF SELECTED ALTERNATIVE

9.1 GENERAL

The original drains under the power house are losing effectiveness and the stability of the structure must be improved by other means. From several possible alternatives, the selected treatment to stabilise the power house is the implementation of new drains at the beginning of the downstream apron. The uplift water pressures are reduced, though less under the upstream part of the structure than in case of the original drainage wells. Still, the stabilising capacity of this solution was demonstrated in section 7.2.9.

Figure 34 shows the location of the new drains and provides a general impression of the uplift water pressures. In reality, there are jumps in the line indicating the uplift pressures. The pressures are larger under the deeper parts of the structure. A detailed impression of the uplift pressures in case of the design assumption, after an entire failure of the drains and the situation for the new drains in the downstream apron are shown respectively in Appendix 13, Appendix 14, and Appendix 25.

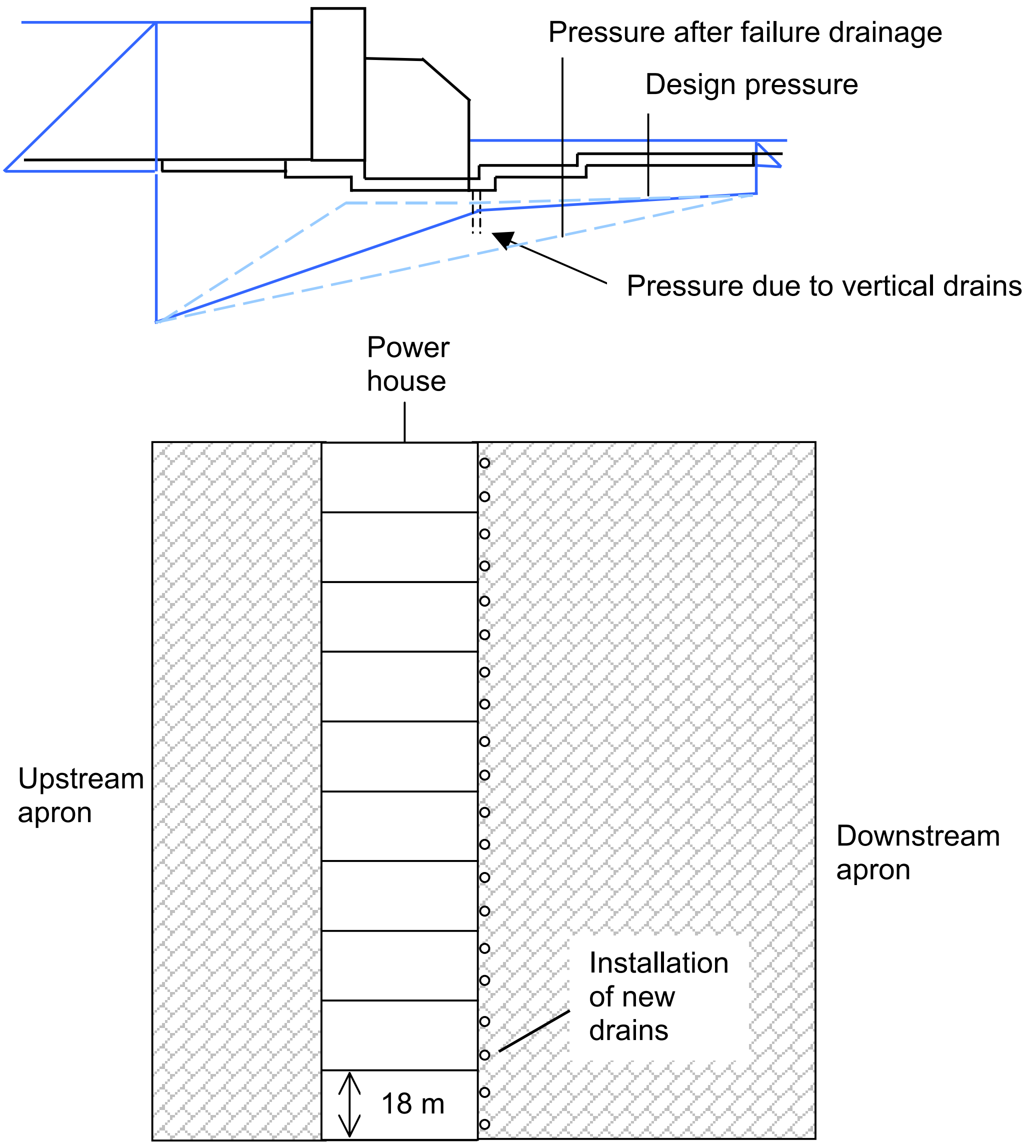
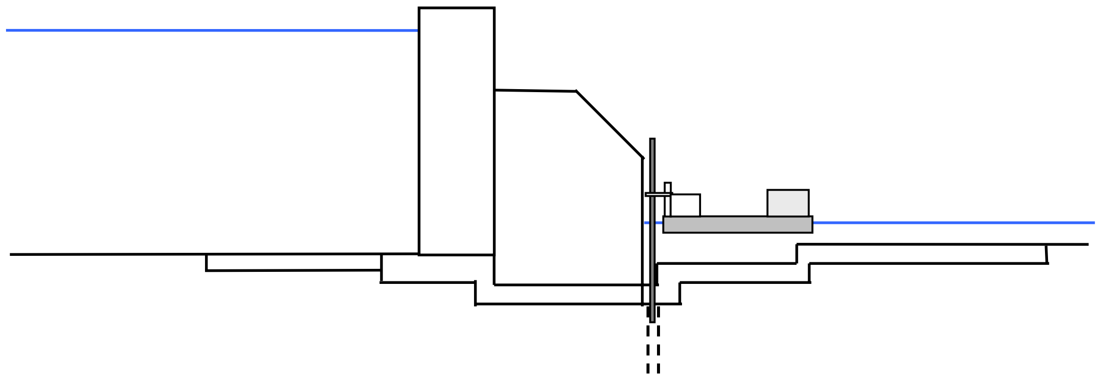


Figure 34: Location new drains in downstream apron

The only concern of this alternative is washout of particles. Nevertheless, the gradient by which the uplift pressures are reduced is much smaller than in case of the drains in the original design. The original drains have been operational for over thirty years, which indicates that in principle drains should be functional, especially in case the drains are accessible for maintenance. The uplift water pressures should always be monitored closely. If a rise in pressure occurs, the drains should be cleaned or replaced.

9.2 WORKS

Before the installation of the drains can be performed, means of transport for the drilling equipment to get close to the location of the future drains are required. There are no navigation channels towards the Plavinas dam. This implies that separate pontoon pieces may be transported over land and combined at the location. The drilling equipment then is installed on the pontoon. For the drilling through the concrete and subsequently through the till material, special rock drills may be used. The shafts should be approximately 15 m long with a diameter of at least 0,1 m. As an estimate, two drains are placed behind each draft tube, resulting in 20 drains.
After drilling is completed, prefabricated perforated plastic pipes can be placed in the shaft. Finally, pumps are connected to the pipes to lower the water level from the underlying drain.



9.3 COSTS

The costs for installing the drains depend strongly upon the working speed. The most expensive is the rent of the pontoon and drilling equipment including the labour hours.
In order to make a rough estimate of construction costs, the time to install 20 drainage pipes next to the power house is set to 20 weeks.
The following cost elements are considered:

- Renting a small pontoon for the time of execution and transport time (one week for the installation of each drain): 20 weeks of execution plus two weeks of transport
  22 weeks * 500 Euro / week [26] = 11 000 Euro
- The transport itself is assumed to cost half this amount
  6 500 Euro
- Drilling through concrete costs about 300 Euro per m on land including personnel and equipment. However, drilling under water will be more complicated and therefore a factor 10 is assumed.
  This results in 300 Euro / m * 10 * 18 m * 20 = 1 080 000 Euro

[26] *VGBouw,* Operating Cost Standards for Construction Equipment. 11th edition, Alphen aan den Rijn/Diemen: 1995

- The drainage pipes are assumed to cost approximately 200 Euro per m. When installing drains of 15 m length this results in
  200 Euro / m * 15 m * 20 = 60 000 Euro
- The underwater pumps are assumed to cost approximately 5000 Euro each (on land pumps cost approximately 1000 Euro and underwater pumps are assumed to be more expensive)
  5000 Euro * 20 = 100 000 Euro
- The installation of the drains and pumps is guided by divers. A rate of 500 Euro per hour is assumed for a total of 80 hours:
  500 Euro / hour * 80 hours = 40 000 Euro
- Closing the openings through which the water from the original drainage wells was discharged:
  10 000 Euro

The total cost come down to roughly 1 310 000 Euro.

10 CONCLUSIONS AND RECOMMENDATIONS

Below, conclusions and recommendations are given for both the embankment dam and the power house.

Embankment dams:
The available data on the embankment dams was too limited to provide detailed suggestions for stabilisation works.
Additional data on the fill material and cavities in underlying rock formations of the embankment dams is required to analyse the seepage through the dam and suggest detailed treatments in order to prevent further erosion.

The washout of soil particles through the relief wells on the right bank needs to be stopped. This may be done by blocking the way of the water by injecting cavities in the embankment dam with grout or possibly by replacing the old drains with new pipes of a finer mesh.

Power house:
The available data to perform this thesis was very limited. Assumptions concerning the dimensions of the power house structure and the parameters of the foundation soil were necessary to analyse the stability.
It is recommended to reanalyse the parameters of the foundation soil. If the foundation soil is stronger than assumed, the required stabilising treatments may suffice to be less intensive than proposed in this thesis.
The assumed geometry of the power house should be confirmed.

The alternative of new drains in the downstream apron next to the power house provides the best alternative to stabilise the power house. This was established in a multi criteria analysis.

Even after installing new drains the risk of particle washout is still present.
It is recommended to analyse data of the original drainage system and the permeability of the foundation soil in order to design the new drains.

The gradient under the power house is smaller than in the design assumption. Therefore the new drains will not cause washout of more particles than in the present situation.

Settlements of the power house structure are assumed not to be increased compared to the present situation. This assumption is valid as the gradient and the maximum soil stresses under the plant will be smaller due to the new drains.
The magnitude of (differential) settlements that the power house and adjacent walls can bear without the formation of cracks compromising the water retaining function needs to be investigated. This is recommended for both present and possible future situations.

Washout and settlements due to the new drains in the downstream apron may prove to be unacceptable. In this case the second best alternative, namely the installation of drains in the upstream apron should be considered. However, the implementation of that second alternative is more complicated and costly.

Any further pursue of these topics certainly requires more research in situ. Feedback and co-operation from both operators and designers of the Plavinas hydroelectric power plant are vital.

REFERENCES

Books:

- Civieltechnisch Centrum Uitvoering Research en Regelgeving, *Kansen in de civiele techniek, Deel 1: Probabilistisch ontwerpen in theorie*. Stichting CUR. Gouda: March 1997
- Duivendijk, van, ir. J., *Energiewaterbouwkunde*. Delft University of Technology, Subfaculty of Civil Engineering: march 2001
- Elling, R., Andeweg , B., Jong, J. de, Swankhuisen, C., *Rapportagetechniek*. Wolters-Noordhoff: 1994
- Fell, R. *Geotechnical engineering of embankment dams*. Rotterdam Balkema: 1992
- Herzog, Max A. M., *Practical dam analysis*. Thomas Telford Limited: 1999
- Novak, P., Moffat, A. I. B., Nalluri, C., Narayanan, R., *Hydraulic Structures*. Spon Press, Third edition: 2001
- Weaver, Kenneth D., *Dam foundation grouting*. American Society of Civil Engineers: 1991
- Nederlands Normalisatie-instituut, *NEN 6740, Geotechnics TGB1990-Basic requirements and loads.* December 1991
- CUR-publicatie 162, *Construeren met grond – Grondconstructies op en in sterk samendrukbare en weinig draagkrachtige grond*, CUR, Gouda. April 1992
- CUR-publicatie 166, *Damwandconstructies*, CUR, Gouda. March 1997
- Verhoef, drs, P.N.W., *Ingenieursgeologie*. Delft University of Technology, Subfaculty of Geosciences. Forth edition: October 1994
- Verruit, A., *Grondmechaninca*. Delftse Universitaire pers, 1997
- VGBouw*, Operating Cost Standards for Construction Equipment*. 11th edition, Alphen aan den Rijn/Diemen: 1995

Reports:

- Institute Hydroproject, *Main findings of geophysical explorations in the area of Plavinas HPP*. 1472-T.18. Moscow: August 2000.
- Norwegian Geotechnical Institute, *Daugava River, Latvia: Dam Safety Improvement Studies, 970009-3, Stability Evaluation of the Plavinas HPP, Latvia.* Oslo: October 1997
- Vaughan, Prof P.R. D Sc F Eng FICE, *Latvenergo – Plavinas Dam Preliminary Stability Analysis*. Suffolk: October 1996
- Norplan A.S. of Norway, Latvenergo, European Bank for Reconstruction and Development, *Feasibility study and project preparation for rehabilitation of the Daugava river hydropower schemes, Latvia,* Final report*:* February 1995
- Daugava hydro cascade, Plavinas HPP*, History of incidents in the power house foundation*: October 2000
- Joint Stock company Latvenergo, *Tender Documents, Scope of Services*. Aizkraukle: April 2001

Drawings:

- Plavinas Dam and HPS, Power house, right part plan elevation +27.26 (23.00), drawing no. 204, Norplan A.S., 30-10-1997.
- Plavinas Dam and HPS, Power house, left part plan elevation +27.26 (23.00), drawing no. 203, Norplan A.S., 30-10-1997.
- Plavinas Dam and HPS, Power house, cross section, drawing no. 110, Norplan A.S., 30-10-1997.
- Plavinas Dam and HPS, Power house, cross section including aprons, drawing no. 109, Norplan A.S., 30-10-1997.

- Plavinas Dam and HPS, Power house, longitudinal section & upstream view, drawing no. 108, Norplan A.S., 30-10-1997.
- Plavinas Dam and HPS, Power house, plan, drawing no. 108, Norplan A.S., 30-10-1997.

Internet:
- http://www.energo.lv
- http://homepages.tig.com.au/~richardw/
- http://www.fe.doe.gov/international/latvover.html
- http://www.eia.doe.gov/emeu/cabs/latvia.html
- http://www.projects500.com/projects/edit/eu22.htm
- http://ww2.abb-alstom-power.se/Alstom/Segen136.nsf/$/E401275C3FB313604125673600328F75
- http://www.orions-ck.lv/e_curprojects.htm
- http://www.lzp.lv
- http://www.vgd.gov.lv/geo/en/p5.html
- http://wall.vgd.gov.lv/geo/en/p6.html
- http://www.lvnet.lv/Riga/Map/
- http://www.odci.gov/cia/publications/factbook/geos/lg.html
- http://www.fe.doe.gov/international/latvia.html
- http://www.eunet.lv/VT/general/
- http://www.paic.lv/English/2_4.html
- http://www.fao.org/docrep/W6240E/w6240e13.htm
- http://www.modlab.lv/en_proj_cascade.html
- http://www.modlab.lv/publications/publ6.htm
- http://www.adc.lv/aizkraukle/karte.htm

APPENDICES
Final thesis

Feasibility study for stabilising the Plavinas dam, Latvia

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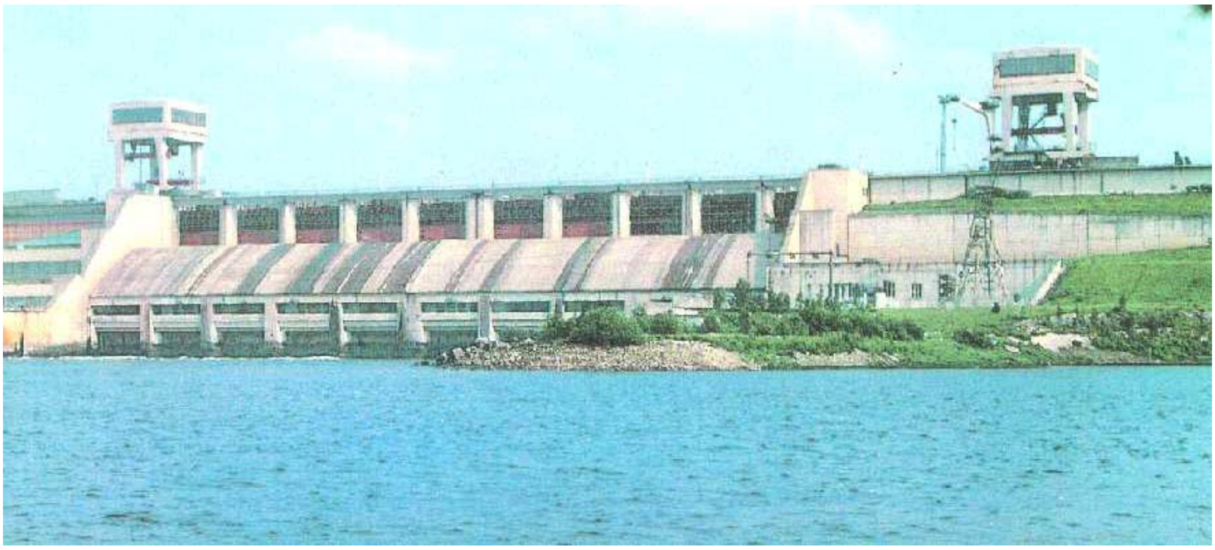
June 2002



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APPENDICES

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Appendix 22: Stability analysis power house for new drains in upstream apron
Appendix 23: Stability analysis power house for new drains in downstream apron
Appendix 24: Stability analysis power house for walls on downstream apron combined with an extension of the upstream apron
Appendix 25: Uplift water pressures for new drains in the downstream apron
Appendix 26: Water levels reservoir and tailrace
Appendix 27: Background information on dams
Appendix 28: Grouting techniques

Appendix 1: Cross section of concrete power house / spillway

Appendix 2: Longitudinal section of concrete power house / spillway

Appendix 3: Longitudinal section across valley and power house

Appendix 4: Cross section right embankment dam

Appendix 5: Cross section left embankment dam

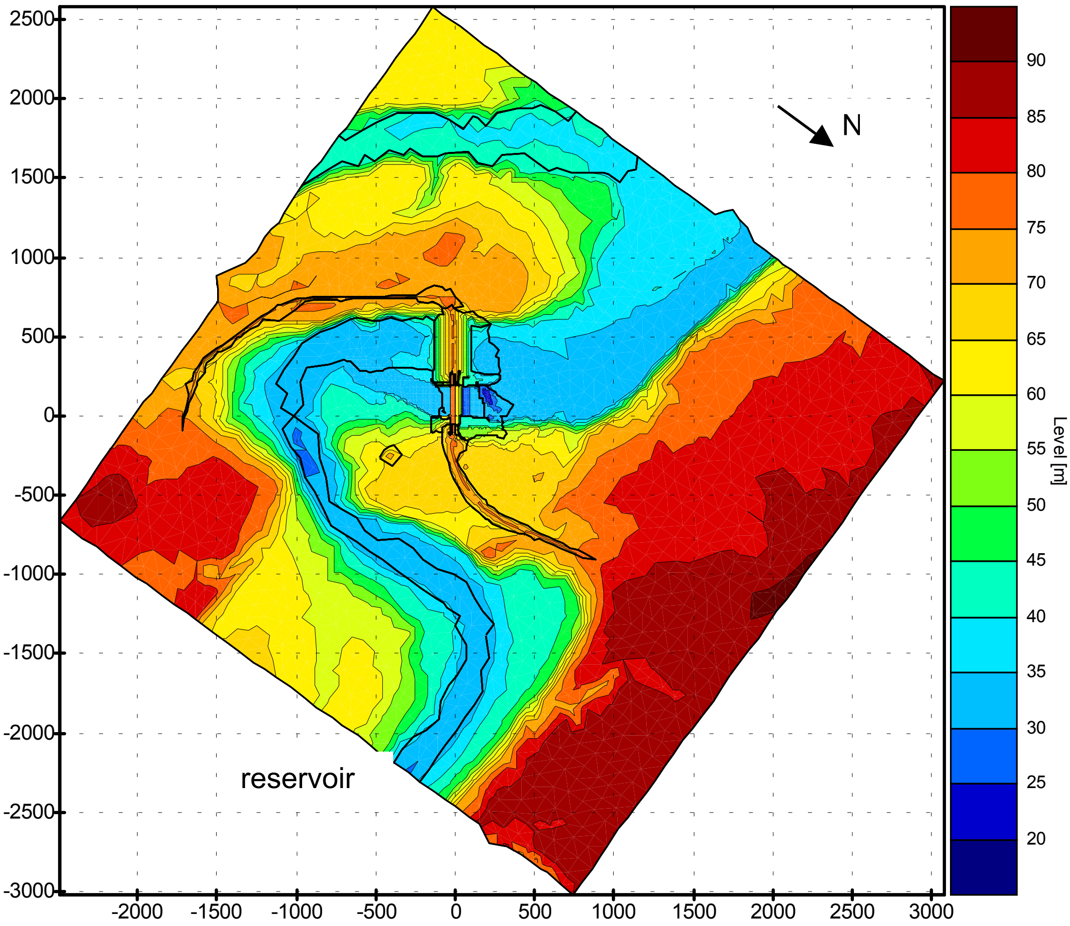
Appendix 6: Location of galleries

Location of galleries

Appendix 7: Plan view of Plavinas dam

Plan view of the Plavinas dam

Appendix 8: Topography of the area of the Plavinas dam



Topography of the area of the Plavinas dam

Note: the water table of the reservoir reaches the level of +72 mASL. Therefore, also the upstream areas marked with yellow shades are under water.

Appendix 9: Diagram of soil classification

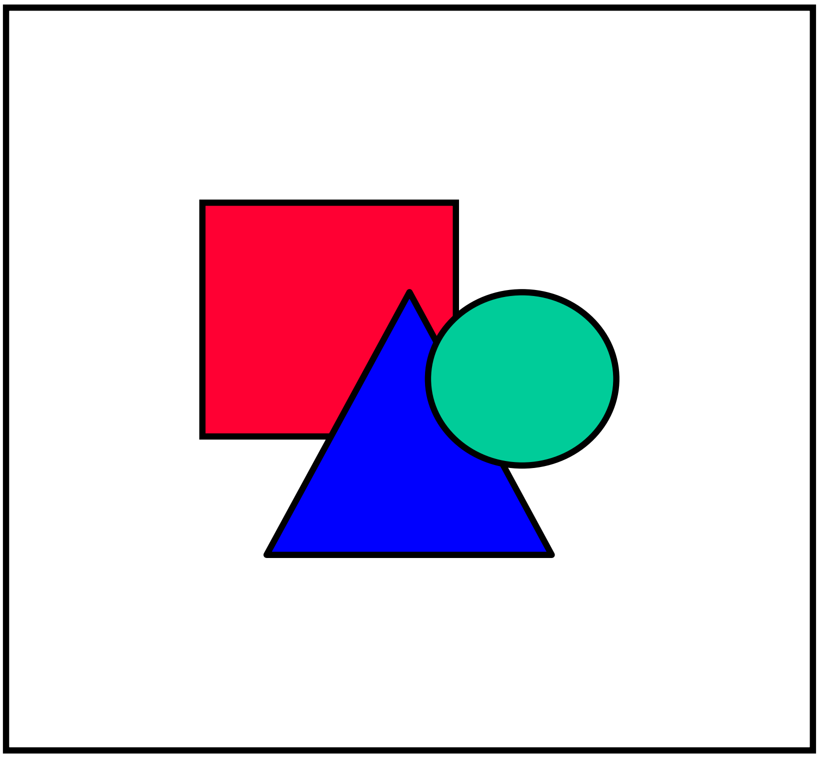


Diagram of soil classification

Appendix 10: Previous findings

PREVIOUS REPORTS

Several incidents have been recorded by the monitoring systems at the Plavinas dam site. In the past, some preliminary analyses have been carried out, see following sections.

Report prepared by Professor P.R. Vaughan, 1996[1]

Professor Vaughan has prepared a preliminary stability analysis of the Plavinas dam. He concludes the following:

The results of the calculations indicate that the structure is stable at present, but the stability depends on the rather unusual seepage control from the upstream apron, and on the drainage arrangements. Hydraulic failure of the joint between the apron and the intake does not lead to immediate failure of the structure. However, the structure loses a considerable part of its stability reserve. According to Professor Vaughan it might move unacceptably, and become vulnerable to penetration of the reservoir pressure below the intake structure. If reservoir pressures were to penetrate this way and choke the upstream drain, then failure would occur. A modest amount of weight added to the intake structure would give a significant reserve against failure in this way. It might be that an increase of safety to modern levels would involve the addition of weight and improved seepage seal provided by the upstream apron.

Both the apron and the power house/spillway are rigid structures sitting on compressible foundation. Both have been subject to movement due to consolidation settlement, and, on the right-hand side, due to loss of ground from the drainage wells.

Report prepared by the Norwegian Geotechnical Institute, 1997[2]

The Norwegian Geotechnical Institute (NGI) has prepared a stability analysis of the Plavinas dam concluding the following:

The analysis shows that in case the original design parameters prove to be valid, the stability of the dam is unacceptable and significant stabilisation works will be necessary. If on the other hand the parameters used by NGI prove to be valid, no stabilisation works will be needed. A final answer to this question can only be found after additional soil exploration and testing has been carried out.

NGI concludes that the stability of the power plant blocks depends on the effectiveness of the under-drainage system. Until 1979 the pore pressures in the ground and uplift of the power plant were stable and very much as predicted in design. Starting in 1979 the pore pressures underneath the upstream apron started to increase with 0,2 m/year.

The stability analysis has demonstrated that even with NGI shear strength parameters the factor of safety may drop to unacceptable levels if the pore pressures below the foundation slab continue to increase. According to NGI, an improved seepage control is needed to safeguard the structure.

Furthermore, NGI concludes that the new seepage control measures should also impede the loss of material from the foundation soil through the drainage wells and the downstream apron weep holes. The measures under consideration are the grouting of seepage paths and distant pumping from the aquifer to relief the pressure at some distance from the power plant structures.

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[1] Vaughan, Prof P.R. D Sc F Eng FICE, *Latvenergo – Plavinas Dam Preliminary Stability Analysis*. Suffolk: October 1996

[2] Norwegian Geotechnical Institute, *Daugava River, Latvia Dam Safety Improvement Studies, 970009-3, Stability Evaluation of the Plavinas HPP, Latvia.* Oslo: October 1997

The geotechnical properties of the glacial till filling the buried valley under the power plant have been re-examined on the base of NGI's experience with glacial soils. The design parameters from the original Hydroproject design were c'=25 kPa and ϕ=24.2° (Mohr-Coulomb shear strength parameters). This last value is according to NGI's experience extremely low for glacial till.
Generally, soil investigations and laboratory tests have shown a mean shear strength parameter of c'=10 kPa and ϕ=34° for glacial till. The mean internal friction angle for the triaxial tests alone was ϕ=37°.

Report by the Russian Joint-Stock company of power and electrification, 2000[3]
Geophysical researches executed at the Plavinas site have given additional information on geological and hydro-geological conditions.

- Identification of the paths of concentrated seepage, study of interrelation between the aquifers and estimation of intensity of seepage process development.

A detected zone of concentrated seepage crosses the right bank dike axis approximately 160 m from the right bank abutment of the power house. This zone is caused by the presence of lateral thrust cracks. It extends to downstream direction between existing relief wells at elevation 42 m and 52 m. The zone is 20-40 m wide and is shown in Figure 22.
In the given zone of rocks of the Amata and Plavinas horizons intensive water exchange takes place. The Amata horizon experiences additional groundwater feed due to overflow of artesian water from the overlying Plavinas horizon. Next to the mentioned zone focused along the paleo-valley, minor weakened zones of a cross direction, supposedly connecting the detected abnormal zone and section of relief wells of the lower row (elevation 42 m) are delineated in the bedrock mass by geophysical methods. In the revealed zone concentrated seepage was is in progress.
As based on the results of geophysical investigations at the left embankment dam, there are no paths of concentrated seepage from the reservoir. However, a zone with attributes of piping development caused by water outflow from the dam embankment into heavily disintegrated dolomites occurring in its foundation, is detected on the downstream slope of the dam.

- Study of heterogeneity of moraine soils by their elastic properties and state in the foundation of structures, assessment of changes in the properties and state and behaviours of moraine soils in the power house foundation with time.

As established by results of cross-hole shooting, moraine soils in the foundation of the structure are heterogeneous by their physical and mechanical properties. Inside the moraine stratum the soils with abnormally low elastic properties are encountered at the depths from 10 - 20 to 80 - 90 m below the foundation slab. This exceeds the thickness of the compressible layer of 50 m adopted in the design and analysis of concrete structure deformations.
The velocities of longitudinal waves in some zones inside the moraine stratum are significantly below the average level for the soils of the given type determined during researches and construction. The density model of the moraine soils constructed on the basis of gravimetrical survey data in the power house foundation consists of two layers. The top layer, 10 - 20 m thick, is of higher density compared to natural conditions. The lower layer up to the bedrock roof is of reduced density. Temporary variations of elastic properties and conditions of moraine soils in the power house foundation have seasonal and trend components.

[3] Institute Hydroproject, *Main findings of geophysical explorations in the area of Plavinas HPP*. 1472-T.18. Moscow: August 2000.

Summary

Some conclusions can be drawn from the data relevant to the incidents recorded in the foundation of the Plavinas plant.

- Incidents recorded at an inverted pendulum (no. 1-10) are associated with increased settlement of the right half of the power house. However, the increased settlement caused by later incidents (than 1979) is not great enough to be clearly apparent in settlement data collected using optical survey methods.
- An increase in hydraulic gradient toward the Amata sandstone, caused by a lowered piezometric level in this stratum as a result of increased flow from the group of high yield relief wells, may have played a role in the occurrence of the movements of the inverted pendulum in 1985.
- An incident involving loss of ground may have occurred near the downstream end of the wall along the right side of the river channel in the early 1990s. The increased rate of settlement has continued undiminished since that time.

Furthermore, the following has become clear:

- Stability is critical in case seepage control fails and the original design parameters are valid.
- Piping has occurred but it is unclear whether it has caused the measured settlements of the structure.
- Controlling the seepage control may be sufficient for stabilising the structure. This can only be guaranteed if the seepage path is significantly extended at the upstream side of the structure or by continuously pumping drainage water. However, the dam is founded on moraines overlying a sandstone layer, which feeds water under pressure into the moraines, resulting in uplift forces. Therefore, the extension of the seepage path will only help if the flow from the sandstone into the till material is limited.
- It is not clear whether sufficient soil investigations have been performed to have a satisfactory insight in the soil profile and soil parameters.

Further Explorations at Plavinas[4]

Several geophysical explorations at the site of the Plavinas plant have given additional information on the geological and hydro-geological conditions. A set of geophysical methods was used to gather data:

- The aquifer intercommunication was studied by electrical survey by self potential method.
- Detection of concentrated seepage paths was done by electrical survey (self potential method), gravimetrical and seismic-acoustic investigations.
- Heterogeneity of the moraine deposits was studied by use of multi point cross hole shooting.

The monitoring systems at the Plavinas dam site have recorded several incidents. Recorded movements, although most likely the result of various short- or long-term processes, could be critical to the safety of the Plavinas plant. Figure 22 in the main report shows the location of points of interest in the following descriptions of incidents.

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[4] Daugava hydro cascade, Plavinas HPP, *History of incidents in the power house foundation*: October 2000

Seepage
A list of important characteristics concerning the seepage paths is given below.

- During the construction period, temporary unwatering wells realised the drawdown of the groundwater level at the area of the construction pit for about 1 year. This led to a large amount of sand washed out from the wells. Around the wells and in solid blocks between them, some cavities and pockets could form, increasing the groundwater seepage at separate sections of the dam. This resulted in the necessity to plug the cavities with proper filler.
- A number of relief wells exhibited increased rates of sand discharge during 1985. The high yield of relief wells (named 2139, 5a, 3a) and those near the PN gallery (Figure 22) have been discharging sand at high rates since 1997, the latter reaching a peak in 1999 but decreasing sharply in 2000. This non uniform distribution of discharges of relief wells can be explained by highly non uniform seepage in the soil mass of the Amata and Gauya series and the presence of concentrated seepage paths in the mass.
- The relationship between the yield of the relief wells and the headwater level revealed by results of routine observation is attributed to a hydraulic connection between the relief wells and the reservoir.
- Piping in the Amata horizon rock occurs, not only due to mechanical removal of fines from the near-filter zones of the wells, but also due to continuous suffosional removal of material through the concentrated seepage paths. This can be extremely hazardous for the structure as the seepage channels continuously developing towards the upstream side can approach the structure. This is particularly valid for the section of the high yield wells located in the immediate vicinity of the power house.
- Increased pumping from relief wells may have increased the hydraulic gradient toward fissures in the Amata sandstone and promoted internal erosion. A zone of concentrated seepage is detected on the downstream reach at the right flank area. The zone is traced through the line 12n, 7g, 44r, 15n and the seepage flow is turned into the direction of the high relief wells. In this direction seepage is not uniform and takes place mainly through local narrow zones. An important zone is supposed to run lengthways the line coupling the wells 15n, 44r and the high relief wells 3a, 5a, 2139. In Appendix 11 a dashed line follows these points in a zone with reduced density of the soil, facilitating the flow of the water. This path of concentrated seepage follows the letters A-A and B-B in Appendix 7. These letters are also indicated in the cross sections of Appendix 4.
- The wells 50n and 12n are in a zone of influence of a large crack. The axis of this crack is between wells 50n and 12n. The width of the zone of influence is about 5m. The way of concentrated seepage is dated for the mentioned fracture. Increased fracturing of the rock mass in the zone of influence of the crack has caused increased karstings. Greatest karstings are marked in a zone of the near surface and in a zone of influence of a crack. The permeability in the revealed zone of the concentrated seepage should be related to fracture karsts-type, at which the speeds of filtration can reach high values. The zone of concentrated seepage caused by the cracks is indicated in Figure 22. It extends to downstream direction between the existing relief wells located at elevation 42 m and 52 m and is 20-40 m wide. The exact height of the seepage is not revealed.
- Some anomalies have been reportedly observed about 300 m to the left of the concrete structure in the left embankment dam. It is unclear what anomalies these are, but as no further investigations are mentioned, it is assumed that this is negligible.

Movements

- Inverted pendulums were used to monitor the relative lateral displacement between the power house and the foundation soil. The anchor of inverted pendulum “1-10” showed displacements toward the left bank and downstream. None of the other inverted pendulums exhibited any unusual behaviour, confirming that the movement was one of the subsoil, not of the structure. High rates of sand loss and changes in flow from the relief wells have been observed prior to the discovery of the displacement at the pendulum. The displacements are noted in the below table Here, x increases as the anchor displaces toward the left bank and y increases as the anchor displaces downstream.

Displacements of inverted pendulum

Incident	Displacement x (mm)	Displacement y (mm)	Ratio x to y
1979	42	72	0.58
1984 (minor)	-4	6	-0.67
1985	11	35	0.31
1998	6	13	0.46
Net total	55	126	0.44

- By far the greatest amount of movement has been recorded at the right end of the power house. The direction of the movement at inverted pendulums “1-10” has not always been the same in all incidents. It is doubtful that the location of the ground loss can be deduced from these data. However, it can be said with reasonable certainty that ground is being lost downstream and to the left of the pendulum. Still, the existence of other locations of ground loss is very possible.
- An interesting feature of the inverted pendulum observations is the seasonal variation noted at many of them. The fluctuations are consistent with the expansion of the power house concrete during summer and contraction in the winter. The inverted pendulums have recorded a continuous slow downstream movement of the power house. This movement could be a result of incomplete reversibility of the seasonal movements because of the influence of the reservoir water pressure.

Deformations

- Inside the moraine stratum, soils with abnormally low elastic properties are encountered at the depths from 10-20 to 80-90 m below foundation slab. This thickness of 60-80 m considerably exceeds the thickness of the compressible layer of 50 m adopted in the design and analysis of concrete structure deformations.
- Bulging of moraine soil to sloughing cavities in the underlying Amata-Gauya strata is a probable reason for movements in the moraine. One of the reasons of non-uniform settlement of the structure by area lies likely in non-uniform deformation properties of the moraine.
- Loss of soil from the glacial till overburden into fissures in the rock (observed prior to the discovery of movements of the inverted pendulum as mentioned above, result in the development of zones of very loose soil and the overstress and collapse of “hard spots” of eroded soil. The soil from the “hard spots” could be expected to move laterally toward the zones of loose soil, thereby densifying them and, in the process, displacing any inverted pendulum anchors in the path of the lateral movement. Both, in the service gallery and the upstream toe gallery, monitoring points at the right end of the power house experienced considerable additional settlement, or an increase in the rate of settlement, associated with the observed subsoil displacement. The effect was not as pronounced at points midway between the right end and the joint between the two halves of the power house. Little if any additional settlement is observed at points located adjacent to the central joint or in the left half of the power house.
- Geodetic observations recorded the settlement of the right bank abutment of the power house, the assembly area and the adjoining retaining walls. They had increased sharply in 1979 to a rate of 5,9 mm/year.
- In 1990 or shortly thereafter, a segment of wall (settlement area, Figure 22) started settling at a considerably increased rate and continues to do so. It may be that a

significant incident took place in this area in the early 1990s but went unnoticed. The fact that the rate of settlement of a segment of wall has not decreased since the early 1990s should be considered. In this particular case, it may mean that the conditions required for the development of arching in the soil (such as ratio of width of area of ground loss to height of soil cover, relative density of soil, effective stress) no longer exist. Therefore settlement as a result of loss of soil volume takes place continuously, instead of sporadically as would be the case if soil arches were to develop or collapse.

- Settlement of the structure has caused minor movements between the two blocks of the power plant. The settlement of the power plant blocks are not uniform, the centre and left side have settled about 100 mm, but the right corner has settled about 300 mm. More serious are the differential movements between the power plant blocks and the adjacent wing walls and embankment structures. At the right abutment, the wing wall has settled 300 mm more than the power plant block and the embankment dam even more. All of this can cause damage, but will not directly endanger the overall stability.

OPERATIONAL EXPERIENCE

Power house[5]

The structural condition of the power plant/spillway appears to be generally sound with little evidence of distress or cracking. Each block as a separate structural entity seems to be in good condition. Still, the movement between the individual blocks has caused concern about the ability of the bitumen filled joint to seal off the water pressure. In 1987 one such joint gave away and a gallery and shaft were partially flooded.

Until 1979 the pore pressures in the ground and uplift on the power plant were stable and very much as predicted in design. In 1979 the pore pressures underneath the upstream apron started to increase. In 1997, the increase is about 0,2 m/year and the pressures were about 5 m below the design maxima.

Embankment dams[6]

The general impression of the Plavinas embankment structures is that they are in good condition. The maintenance of the structure has been adequate for the prevailing condition. The upstream slope protection shows clear signs of deterioration. The wooden planks in the expansion joint are more or less rotten above the normal water level. Thus, if a storm occurs during a flood event, the pore pressures may increase significantly with slippage of the concrete slab as a result. Slippage can also occur in case of rapid drawdown.

The Daugava embankment dams have drainage mainly through pipes, embedded in the downstream embankment.

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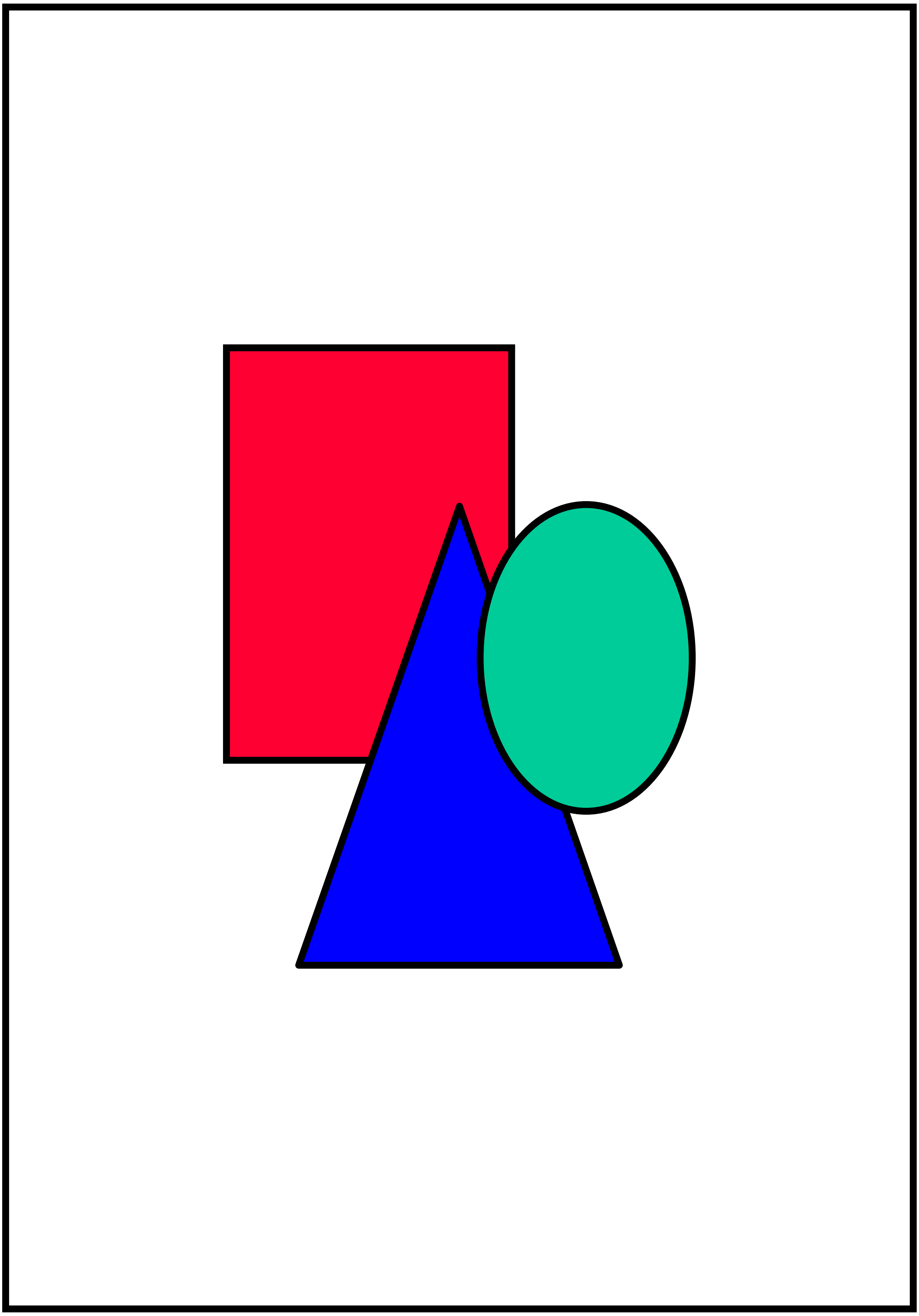
[5] Norwegian Geotechnical Institute, *Daugava River, Latvia Dam Safety Improvement Studies, 970009-3, Stability Evaluation of the Plavinas HPP, Latvia.* Oslo: October 1997

[6] Norplan A.S. of Norway, Latvenergo, European Bank for Reconstruction and Development, *Feasibility study and project preparation for rehabilitation of the Daugava river hydropower schemes, Latvia,* Final report*:* February 1995

Appendix 11: Seepage path in zones of reduced density



Seepage path in zones of reduced density

Appendix 12: Schematic of failure modes

Appendix 13: Uplift water pressures with drainage under power house

Appendix 14: Uplift water pressures without drainage under power house

Appendix 15: Stability analysis power house with working drainage wells

Appendix 16: Stability analysis power house after failure drainage wells

Appendix 17: Stability analysis power house for rising uplift water pressures

Appendix 18: Stability analysis power house for additional weight on power house

Appendix 19: Stability analysis power house for additional weight on downstream apron

Appendix 20: Stability analysis power house for extension upstream apron

Appendix 21: Stability analysis power house for adding vertical curtains

Appendix 22: Stability analysis power house for new drains in upstream apron

Appendix 23: Stability analysis power house for new drains in downstream apron

Appendix 24: Stability analysis power house for walls on downstream apron combined with an extension of the upstream apron

Appendix 25: Uplift water pressure for new drains in the downstream apron

Appendix 26: Water levels reservoir and tailrace

Appendix 27: Background information on dams

Dams[7]

General

A typical reservoir dam is a wall of solid material built across a river, blocking the flow of the river and thus storing water in the lake that will form upstream of the dam as water continues to flow from the river.

The main purpose of most dams is to create a permanent reservoir of water for use at a later date. The dam must be impervious so that only very little water escapes downstream. An essential part of a dam is therefore the "impermeable membrane". It is not necessary that the entire dam is impervious. The dam foundation must also be impervious, as must the river valley, which forms the storage reservoir.

Next to being impervious, it is essential that the dam remains stable. The dam wall must have sufficient strength to firstly, stand permanently under its own weight especially when at least part of the dam wall is saturated with water and secondly, resist the water pressure in the reservoir upstream of the dam. This water pressure exerts forces on the dam tending to push it downstream. The higher the dam, the greater the depth of water stored behind the dam and the greater the water pressure on the dam wall. The dam must also have sufficient strength to resist other forces or displacements to which it may be subjected from time to time such as dynamic movements caused by earthquakes. The threat to dams posed by an earthquake varies widely depending on the region of the world in which the dam is located.

A dam must have an outlet in order to release water in controlled quantities. Depending on the purpose of the dam the water may be released into a pipeline to supply a city with water, or into a hydroelectric power station to generate electricity. The water may also simply be released into the riverbed downstream of the dam and allowed to flow naturally downstream, and used for irrigation of crops further downstream.

When the river discharges a very large volume of floodwater the storage reservoir behind the dam will accumulate the water. This can be far more water than can be released through the outlet. Therefore, a dam must have some means whereby these large volumes of floodwater can be spilled without causing damage to the dam itself. This usually is achieved by a spillway, which in most cases, is an open cut channel large enough to carry the floodwater. If a concrete dam is concerned, the spillway may form part of the dam itself.

Types of dams

Dams can be grouped according to the type of material of which they are constructed. Concrete dams are further grouped according to how they achieve their strength and stability.

Concrete dams

a) Concrete Gravity Dams
   rely on the weight of the concrete of which they are built to resist the forces (gravity, water pressure, earthquake) to which they are subjected.

b) Concrete Arch Dams
   and

c) Concrete Buttress Dams

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[7] http://homepages.tig.com.au/~richardw/

can be built using a smaller amount of concrete than that required for a Gravity Dam and, as a result, are cheaper to build. This is possible because Arch and Buttress Dams are designed to transfer some of the loads to the side of the valley or the end of the buttresses. This gives additional support to the dam with little concrete involved.

Fill (embankment) dams

a) Earth fill dams

- Impervious core type
  The dam is composed of pervious, loose material (sand, gravel) with an impervious core, which retains the water. There are different possible locations for the core within the dam (central core, inclined core) or on the outside. The material can be a thin bitumen zone up to a thick clay core.

- Homogeneous type
  The dam is composed of only one kind of material (excluding the slope protection). The slopes of this kind of dam are very gentle. A good drainage system for the seepage at the downstream part of the dam is needed.

- Combined type (zoned type)
  A central impervious core is surrounded by several different kinds of soil, which are more pervious than the core. These materials protect, support and surround the core. Additional functions: upstream fill remains stable during quick drawdown of reservoir and the downstream fill serves as drain and support for the impervious core.

b) Rock Fill Dams

Most of the dam is constructed of permeable rock fill which, by itself, would be incapable of retaining water. An impermeable membrane has to retain the water. This membrane can be placed inside the dam or be applied on the upstream slope. Again, the membrane can consist of clay, concrete, steel, bitumen, asphalt concrete and for some small dams wood.

In case of the Plavinas dam, the embankment dams are composed of hydraulic fill of sandy loam. Together with a grout curtain, the dams seem to be sufficiently impermeable. The embankment dams are connected to the concrete structure of the power house / spillway.

Purpose of dams

Dams are usually built for one or more of the following purposes:

- To provide a supply of water for towns, mining sites and recreational purposes
- To provide a supply of water for the irrigation of crops
- To generate electricity in hydro-electric power stations
- To help control or mitigate floods
- To regulate river discharge for navigational purposes
- To contain and store waste (tailings) from mines

Many dams are multipurpose and most dams have at least some flood mitigation effect in addition to their primary purpose. Dams built specifically for flood control may have some of their storage capacity kept empty during normal river flow conditions so that space is available to store excess water inflow under flood conditions. The flood mitigation effect of a dam is such that the downstream maximum water level at the peak of the flood is reduced. After the peak has passed, the river levels usually remain high for a longer period than would have been the case if the dam had not been built. This is because excess floodwater is only stored behind the dam temporarily and is slowly released from the dam in the days and weeks after the flood peak has passed.

The purpose of the Plavinas dam is to generate electricity by using the head difference between the reservoir and the tailrace.

Geology

On a large dam construction project the engineering geologist is concerned with:

- the geology of the dam site including the foundation for the dam itself and the sites for other structures such as spillway, diversion tunnel and outlet works. Questions that need an answer include whether the dam foundation has sufficient strength and durability to support the type of dam proposed, whether the dam is stable and the foundation sufficiently impermeable. If not, additional questions could be how much grouting will be required and whether the spillway chute will require concrete lining.

- the geology of the area to be occupied by the reservoir once the dam is completed. Questions often asked here include whether the storage area is impervious or if there are areas of cavernous limestone. In that case, the permeability would be too high for the dam to retain the water. Also landslides into the reservoir are possible which might cause a wave to be pushed over the top of the dam (overtopping), resulting in a dam break.

- the availability of construction materials in the required quality and quantity. This includes for example means of quarrying and borrowing areas.

Appendix 28: Grouting techniques

Grouting Techniques

Below, a summary different of grouting techniques is given.

- Permeation grouting involves the injection of a treatment fluid within the pores of the soil. This method depends on the relation between the coarseness of the pores and the fineness of the injection fluid. Its applicability is generally restricted to coarse soils as sand. Several types of injection fluids are available: cement grouts, clay / bentonite suspensions and chemical injection fluids. As the grain skeleton remains intact, permeation grouting increases the cohesion of the soil. The permeability of the soil is reduced. Heterogeneous soils make staged injections with different injection fluids necessary. Initial injections treat the high permeable zones, followed by other injection fluids to fill the less permeable zones. This type of grouting is sometimes called chemical grouting.

- Jet grouting involves the high-pressure injection of a water-cement mixture in the soil. The high-pressure injection has an erosive action on the soil structure, thereby creating a soil-water-cement mixture in place. Considering the high pressure of the injection, jet grouting is largely independent of the soil type. Generally, the strength of the soil is increased. The decrease in permeability depends on the interlocking of the jet grout columns.

- With fracture grouting, the soil is fractured with a low viscous cement grout fluid, generally with the objective to lift the ground or minimise settlements (compensation grouting). Injections are done by means of Tubes a Manchettes (TAMs), which are installed in the soil mass that needs treatment. By repeatedly fracturing the soil mass with small injection volumes, e.g. adding volume to the soil mass, the stress in the soil mass is increased. Eventually this leads to lift of the overlying structure or reducing the effect of deformation due to underground excavation works. By fracture grouting, the stress state of the soil mass is increased, leading to a higher stiffness of the soil mass. The permeability of the soil is generally not significantly reduced.

- Compaction grouting involves the injection of a high viscous, stiff, cement grout in the soil with the objective to consolidate or increase the stress. The consistency of the grout and the injection process should be such that the soil is not fractured and the pores are not penetrated. Compaction grouting can be best compared with blowing up a balloon in the soil mass. By compaction grouting the stress state of the soil mass is increased, leading to a higher stiffness of the soil mass. The permeability of the soil is generally not significantly reduced.

- Deep mixing involves the mixing of the soil with a water cement mixture by means of a large diameter auger. This way, columns of improved soil are being made. The strength and stiffness of these columns is increased. The reduction in permeability is, similar to jet grouting, dependent on the interlocking of the columns.