Raising of the Ajaure embankment dam by extending the moraine core with a geomembrane

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SYNOPSIS. The Swedish Ajaure embankment dam is a high consequence dam which is 46 m high and was constructed between 1964 and 1966 with commissioning in 1967.

During the 1980's it was noted that the horizontal displacements in the main embankment dam didn't show any sign of diminishing over a time period. From the time of construction up to the year 2001 the total displacement at the crest was in the order of 500 mm and the creep has continued at a rate of approximately 8 mm per year. In order to stabilize the dam, and allow for future raising of the dam, supporting berms were placed on the downstream side of the left embankment dam in 1989 and 1993.

In addition to the deformation problem the Ajaure Dam required to be upgraded to allow for the new design flood. After comprehensive investigations and studies it was decided to raise the crest of the dam to be able to release the design flood at a water level 5 m above the retention level. The owner Vattenfall used a risk analysis as one input in the decision process to raise the dam. The risk analysis is in the subject of a separate paper for this conference (Bartsch, 2004).

Different construction options were considered, and a geomembrane was finally chosen for the extension of the moraine core. A Flexible Polypropylene (FPP) with a thickness of 1.5 mm was selected. Bentonite Enriched Sand (BES) was used to connect the existing core with the geomembrane. The design of the crest raising was started in 2001 by Golder Associates, UK and continued by SwedPower with detailed design and tender documents. The design and the construction, which was completed in 2002, are described.

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BACKGROUND

The Ajaure Dam is situated in the upper part of the River Ume Älv in northern Sweden. The Ajaure embankment dams are classified as high consequence dams, as a dam failure could have disastrous consequences for the downstream hydropower plants all the way to the Baltic Sea.

The dam is similar to that of other Swedish dams built at the same time. However, the downstream slope incline was originally considerably steeper (1V:1.8H and 1V:1.5H near the crest) than that of other Swedish dams. At a late stage of the construction period it was decided to raise the crest of the dam by approximately 1 m. The reason for this was to increase the freeboard along the main part of the dam and to allow for post construction settlements. The raised crest level resulted in steep slopes (1V:1.35 to 1.40H, approx. 35°) in the upper approx. 12 m of the dam. Below this level, the downstream slope has an incline of 1V:1.8H.

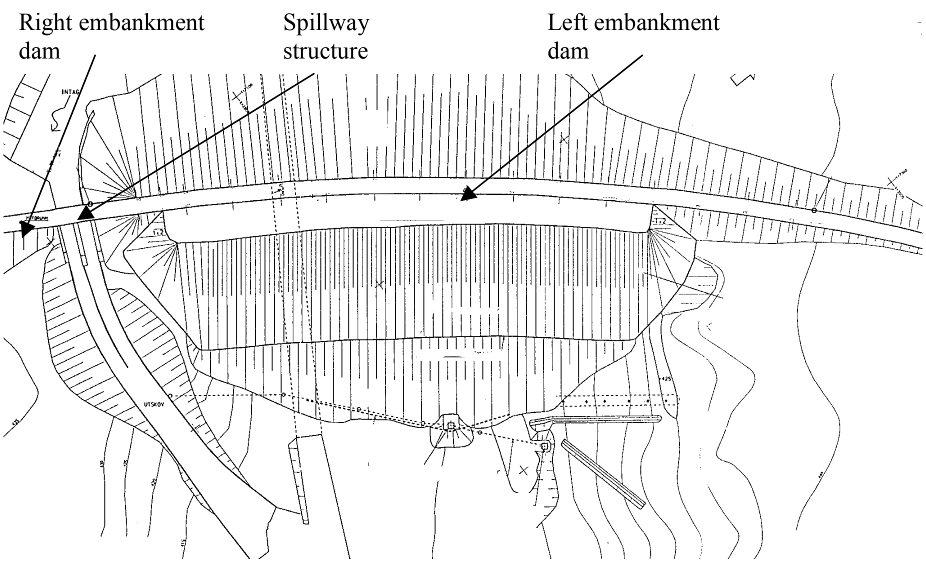


Figure 1. Plan of Ajaure after the placing of the second berm (marked B).

The greatest dam height over bedrock is 46 m. The total length of the dam construction is 522 m. In cross-section the embankment dam has a central impervious core of moraine surrounded by filter zones on the upstream and downstream side. The supporting fill is rock, taken from excavation of the power station, diversion- and tailrace tunnels.

PROBLEM DESCRIPTION

There were two different main problems concerning the safety of the dam. On the one side there was an ongoing exceptional displacement in the left embankment dam. On the other side the new design flood required a rising of the core of dam core by 5 m.

During the 1980s it was noted that the horizontal downstream displacements in the main embankment dam didn't show any sign of diminishing over a time period. The horizontal deformations are significantly influenced by the reservoir as shown in Figure 2.

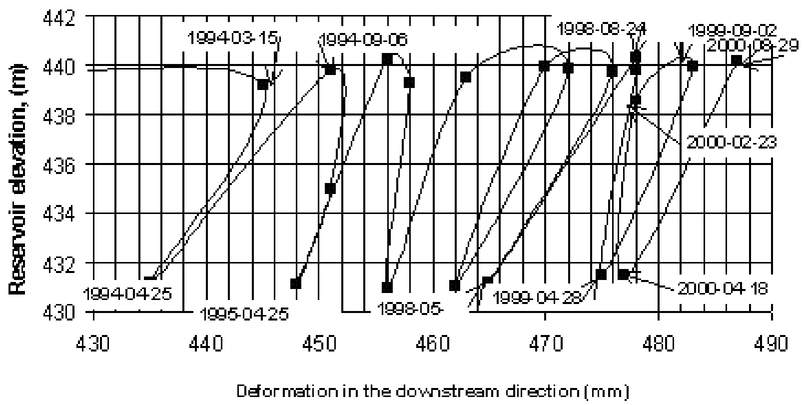


Figure 2. Record of horizontal displacements at the crest

Compared to measurements of deformation in other Swedish dams of similar design, the horizontal deformations were notably large. The present horizontal displacement of approximately 8 mm/year was almost linear and showed no tendency to diminish.

In 1989 test pits were excavated along the downstream toe of the left embankment dam. The rock fill consists of schists and gneiss with a high content of mica. The rock fill had a high content of fines, see Figure 3. It is judged the fines at the lower parts of the fill are partly washed down from higher elevations by the precipitation.

Laboratory shear tests showed that crushing of the material occurred to a large extent. The result of these tests established the weakness of the material at high shear stress, as well as exceptionally low shear strength of the material in the supporting fill. The low safety factor indicated that the shear stresses in situ were close to possible mobilized shear stresses at failure. It was concluded that the high shear stress, in combination with the cyclic loading from the reservoir, results in the progressive crushing of the rock fill and that crushing could be causing the continued horizontal deformations.

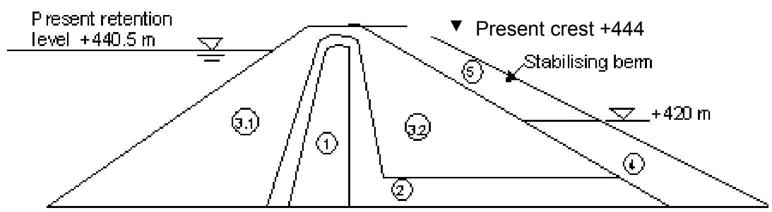
Stability analyses also indicated low stability and a stabilising berm of blasted rock was placed against the lower half of the downstream side of the left embankment dam to increase the stability margin in 1990. The berm was 18 m wide and 20 m high, with a total volume of approximately 50,000 m^3. The fill material was placed with the slope inclination 1 vertical to 2 horizontal, The berm was also intended to increase the erosion resistance at the downstream toe in the dam. No decrease in deformation was however noted after the lower berm had been placed.

Figure 3. Test pits in the downstream toe of the original supporting fill

A second upper berm with a total volume of approximately 100,000 m^3 was placed in 1993. At this time the berm that was placed in 1990 was raised using in the same inclination 1 vertical to 2 horizontal up to a level approximately 1.5 m under the original crest of the supporting fill. The main purpose of the second berm was to further increase the stability margin. Furthermore the second berm was designed to make it possible to raise the crest in the future.

The horizontal downstream movement of the central part of the left dam increased dramatically after the placement of the second berm, and on examination in August 1993, longitudinal fissures on the crest of the dam outside the guardrail were observed.

It has been possible to calibrate reasonably well the deformations that have been recorded while the two supporting berms were placed in 1990 and 1993 with the deformation calculation program PLAXIS. Thereafter the displacement for the raising of the crest from +444 m to +446 together with a 4.8 m higher water level during a design flood was calculated as shown in Figure 4. The sealing element in the crest is not modelled and thus not shown in the figure.



1	Core, moraine
2	Filter, natural gravel
3.1	Upstream shell, rockfill
3.2	Downstream shell, rockfill
4	Stabilising berm of rockfill constructed in 1990
	Stabilising berm of rockfill constructed in 1993

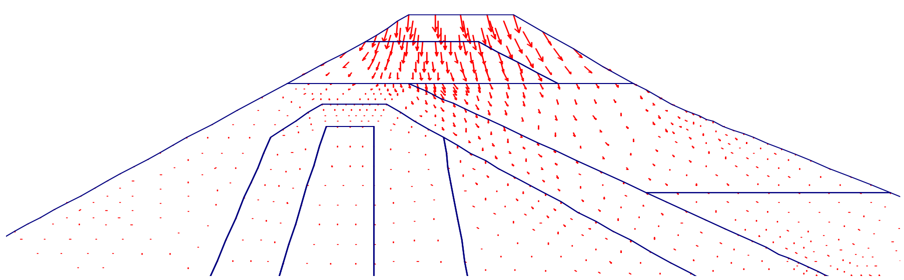


Figure 4. Calculated displacements using the PLAXIS program [ref.1].

The horizontal deformation in the top of the existing left embankment dam core is expected to be minor, in the order of 5 mm, while the vertical displacement in the dam crest is expected to be in the order of 100 – 120 mm. The largest vertical displacements are however expected to occur in the downstream supporting fill. In case of the design flood occurring an additional horizontal displacement of some 130 mm is expected, taking a new suggested crest elevation of +447 into account. The notable displacement will according to the calculations take place outside the moraine core, see Figure 4, which is important since the ongoing horizontal displacement could otherwise cause transversal cracks in the moraine core.

DESIGN

Because of the left dam stability problems and the shortage of suitable moraine material in the area it was decided to raise the core using a geomembrane in a bentonite enriched sand layer set into the existing core crest as the impermeable element. This alternative presented advantages in terms of being less sensitive to displacements, quick construction as well as in cost over other options considered for upgrading the dam. In addition it required a smaller amount of material to be placed on the crest, than if a moraine core had been constructed. The required fill volume was further reduced by introducing an L-shaped concrete wall along the upstream side of the crest to protect against wave run-up. A cross section of the top of the dam is shown in Figure 5.

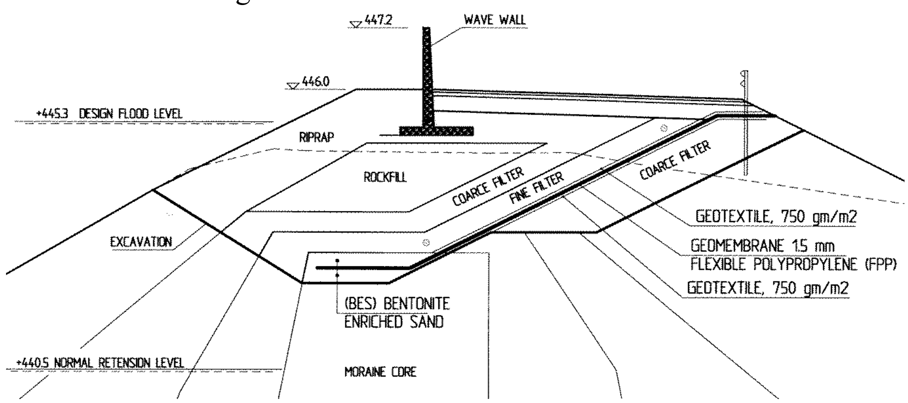


Figure 5. Design of new dam crest on the main embankment dam.

The membrane was embedded in the top of the original moraine core by two layers of bentonite enriched sand. The practicability of constructing the connection to the existing core has determined the geometry of the toe detail. The connection is shown in Figure 5 and is designed to: maintain the integrity of the water retaining structure; seal the seepage path along the surface of the geomembrane; lengthen potential seepage paths to at least half the potential water head; minimize the length of time the excavation exposes the core and to minimize the required depth of excavation.

The slope angle for the membrane was selected for the compaction of the bedding and to avoid a plane of weakness along the membrane and its protective layers. A geotextile was selected as protective layer to achieve a smooth surface for the membrane. The textile maintains the integrity of the membrane against puncture by protruding stone edges. A coarse filter was used as bedding for the textile and membrane.

Assuming that no undetected large-scale damage occurs to the membrane during construction seepages are expected to be small if the sealing in the future will be loaded during a design flood. For seepages that do occur, the coarse filter of gravel provides adequate drainage. The thickness of the coarse filter was determined by practical considerations of traversing compaction equipment. Fine and coarse filters were placed at the upstream side of the membrane.

During construction, settlement of the fine and coarse filters and the rock fill may induce tensile forces in the geomembrane system. In order to minimise the build up of stresses in the membrane the anchoring of the top edge of the geomembrane and protective geotextiles is designed such that the membrane will pull out of the anchorage before tensile forces in the membrane exceed the nominal yield stress. The anchor length is placed on a horizontal surface.

MATERIAL PROPERTIES AND TESTING

Bentonite Enriched Sand, BES

This material is formed by the mixing of sand with bentonite and then adding sufficient water to make the mix suitable for compaction to a high density yet retain some flexibility without cracking. The permeability of the existing core is believed to be in the range 2 x 10^{-7} m/sec to 2 x 10^{-9} m/sec. In order to match properties the target design permeability of the BES is 1 x 10^{-8} m/sec. The sand was single graded with D_{15} = 0,06 – 0,20 mm, D_{85} = 0,20 – 0,60 mm and D_{max} = 10 mm. After laboratory testing it was decided to mix the sand with of 8% (by weight) sodium bentonite with a montmorillonite content > 80 %; 75 % of the bentonite particles should pass the 0,075 mm screen at dry screening; moisture content 8 – 15 % (tested according to BS 1377); liquid limit > 300 %; and swell > 24 ml/2g after 24 hours.

The BES was tested to determine the optimum density, moisture content and bentonite content in order to achieve the required permeability. Pre-testing of the BES was performed using the following methods: wet screening; sedimentation analysis of material < 0,075 mm; and determination of the hydraulic conductivity at 95 % of Standard Proctor density. The mix was then tested in field trials seen in Figure 6.

Figure 6. Field trials with BES. At left the trial surface is compacted in layers of 0.1 m and to the right the material is compacted in two layers of each 0.3 m at water content of 15 %.

Geomembrane

A 1.5 mm thick Flexible Polypropylene (FPP) membrane was selected as this material is more flexible and easy to handle than e.g. HDPE. The FPP is judged to be able to deform around any residual projections in the bedding. A texturing type was available which was necessary in order to develop sufficient friction between the membrane and the geotextiles to maintain stability during construction and operation.

The membrane had the following requirements and was tested according to the following standards: thickness and density (1,5 mm, 900 kg/m^3, ASTM D5994 and D1505A); tensile properties (stress 27 kN/m, elongation 800 %, ASTM D638); tear resistance (90 N/mm, ASTM 1004-90); puncture resistance (300 N, FTMS 101C method 2065); brittle temperature (- 50°C, ASTM D1693); friction angle 29°; carbon black content (ASTM D5994) and carbon black dispersion (ASTM D1603).

Geotextile

The geomembrane was protected from damage by projections and irregularities in the bedding and the coarse filter material, by careful preparation of the bedding surface and a non-woven geotextile with nominal weight of 750 gm/m^2. The same type of geotextile was chosen at the upstream side of the membrane as a protection towards the upstream fine filter.

WORK PROCEDURE

The top of the embankment dam crests was excavated down to 0.6 m below the moraine core and the BES was spread and compacted on top of the moraine, see Figure 7. The excavated material was placed on the upstream side to serve as wave run-up protection during the process of raising the core. The coarse filter was placed, compacted and trimmed prior to the excavation of the existing core. Some blinding of additional coarse filter was performed to fill in voids in the slope face after compaction of the coarse filter. The slope was rolled again after blinding to smooth-face the surface before placing the geotextile on the slope.

The stripped existing core was compacted and then lightly scarified and sprayed with water immediately before placing the BES. A layer of BES was then placed as bedding for the membrane on the excavated core surface, Figure 7. The BES surface was protected against rain and drying using plastic covers while the geotextile was placed on the coarse filter, see Figure 8.

Figure 7. Compaction of the BES bedding layer for the membrane on the existing moraine core. To the right is the compacted coarse filter bedding for the geotextile.

The next step in the construction sequence was to place and weld the membrane on the slope directly on the geotextile, see Figure 9. As the membrane was placed at an inclination of 1V:2H a temporary anchoring was required at the top of the membrane. This was carried out by nailing the membrane in its upper end with 1 m long, ø 20 mm, reinforcement bars. The load of the fill material on the top part of the membrane as described above achieved the permanent anchoring.

Figure 8. Placing of geotextile on the compacted coarse filter bedding. The textile is temporarily held in place by rocks.

Figure 9. Placing of membrane on the geotextile. The BES (to the right in the picture) is temporarily covered by plastic to protect the core against the heavy rainfalls that occurred during construction.

Welding was done with double seams in order to be able to test each seam for water tightness, see Figure 10. The seams are required to have at least 75 % of the geomembrane strength at stress at break yield point. In addition to this destructive testing was carried out on a selected part of each gore of the membrane. These parts were tested for peeling and shear resistance.

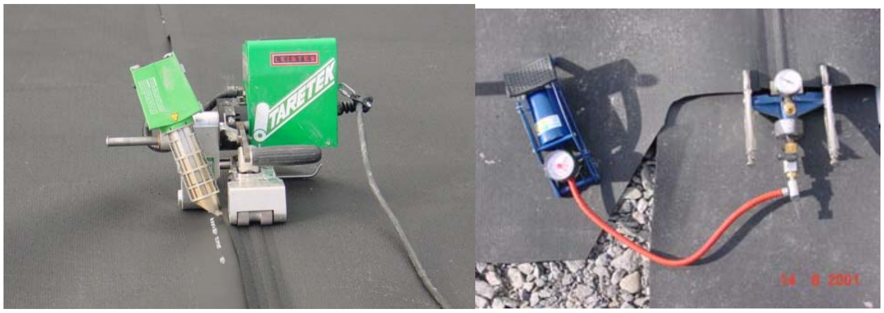


Figure 10. Membrane welding machine for double seams (left) and air pressure pump for air pressure tests of the membrane seams (right)

A non-destructive air pressure test at a minimum pressure of 200 kPa (2 kg/cm^2) was carried out along the entire lengths of all field seams including patches and repairs, see Figure 10. The requirement was that following initial pressure stabilization the pressure should not drop by more than 10 % in 5 minutes.

Figure 11. Protected connections through the membrane for instrumentation in the downstream filter.

The second BES-layer was placed and compacted above the toe of the membrane surface to complete the connection of the membrane to the existing core. The upper surface of the BES was laid with a fall to prevent ponding, from infiltration, that might soften the BES. The covering geotextile placed on the membrane was extended to cover the BES, to prevent stones being driven into the upper surface of the BES during construction.

No direct compaction of the fine filter on the upstream side of the membrane was done, as it would significantly have increased the contact stresses. The geotextile does however provide sufficient protection to the membrane during compaction of the coarse filter. After the completion of the filters the section was raised using rock fill to the new crest level. Finally the upstream end of the new crest was provided with a L-shaped wall to protect the crest against wave run-up, see Figure 5. The construction of the upstream wall allowed a lower crest elevation and thus allowing a shallower and suitable slope angle for the membrane.

CONCLUSIONS AND LESSONS LEARNED

The construction work with the geomembrane was a very quick operation. In spite of sometimes difficult geometry and many welds for pipes, testing etc. the 3,000 m^2 membrane was completed within a week.

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