British Dam Society Conference 2004 – Canterbury, Kent

LONG-TERM BENEFITS AND PERFORMANCE OF DAMS

Chapter 2. The use of new materials

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

Å. NILSSON, SwedPower AB, Stockholm, Sweden
I. EKSTRÖM, SwedPower AB, Stockholm, Sweden

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.

LONG-TERM BENEFITS AND PERFORMANCE OF DAMS

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.

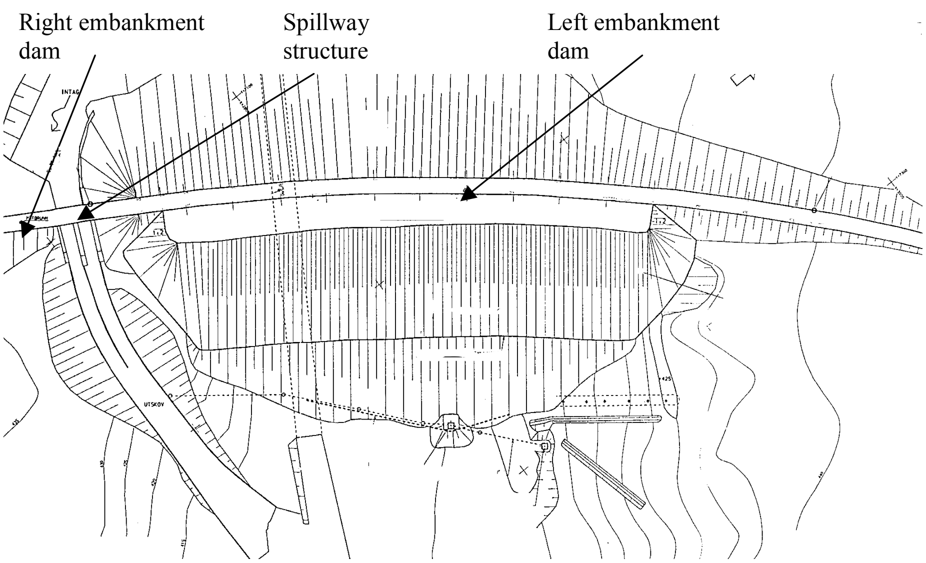


Figure 1. Plan of Ajaure after the placing of the second berm (marked B).

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.

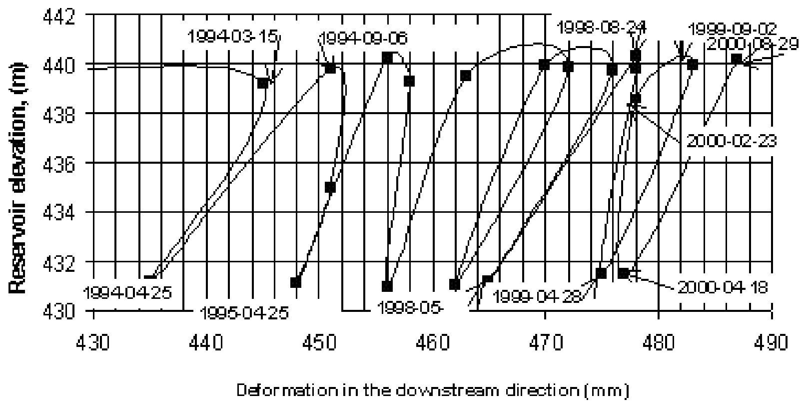


Figure 2. Record of horizontal displacements at the crest

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.

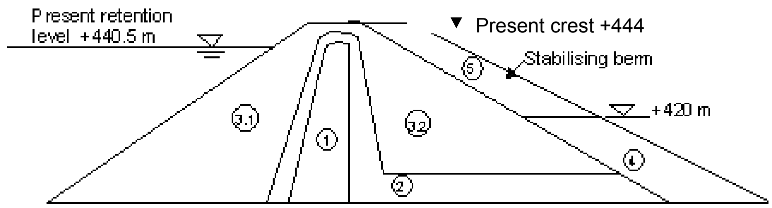
Stability analyses also indicated low stability and a stabilising berm of blasted rock was placed against the lower half of the downstream side of the left embankment dam to increase the stability margin in 1990. The berm was 18 m wide and 20 m high, with a total volume of approximately 50,000 m^3. The 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

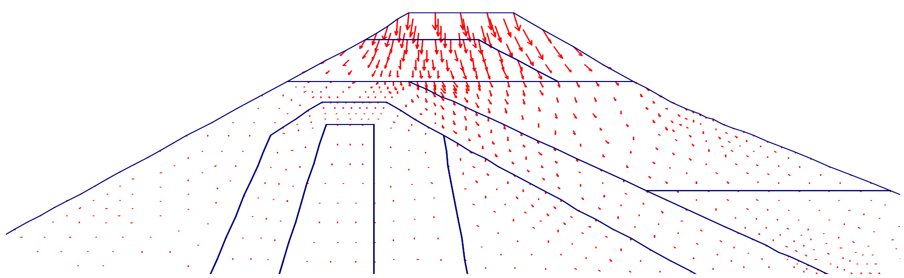


Figure 4. Calculated displacements using the PLAXIS program [ref.1].

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.

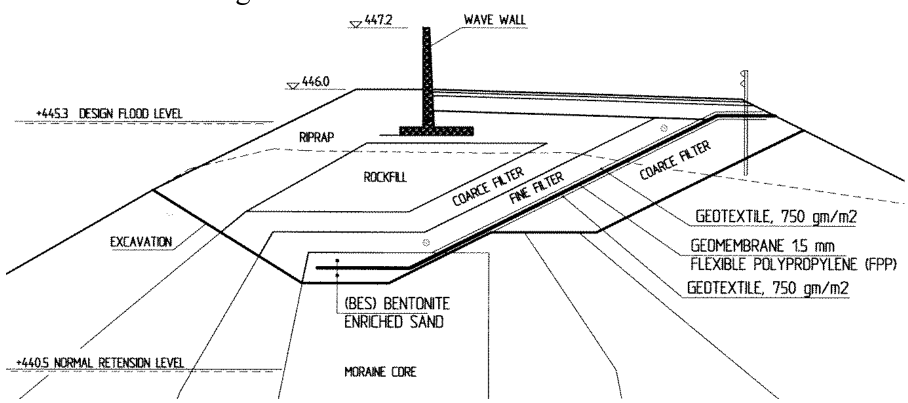


Figure 5. Design of new dam crest on the main embankment dam.

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.

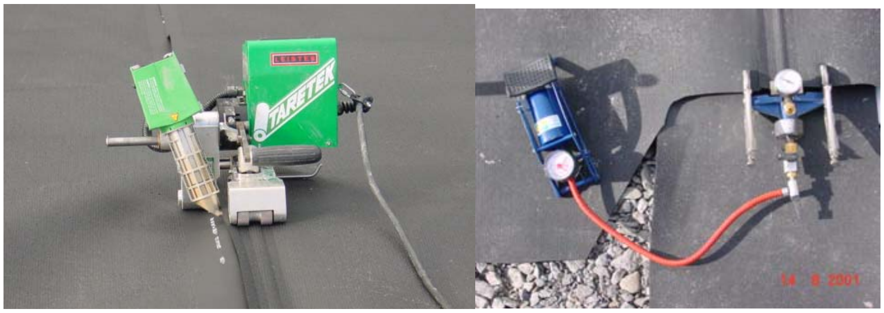


Figure 10. Membrane welding machine for double seams (left) and air pressure pump for air pressure tests of the membrane seams (right)

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.

ACKNOWLEDGEMENTS

Special thanks go to Bill Kearsey from Golder Associates, UK for the pre-design carried out for the raising of the crest. Bill also kindly guided the Swedish design team around in a study tour to different construction sites in UK where geomembranes where used in different applications.

The authors also want to thank Vattenfall the owner of the Ajaure Dam for the permission to publish this paper.

REFERENCES

Bartsch M. (2004). FMECA of the Ajaure Dam. *Long term benefits and performance of dams,* Thomas Telford.

Chang Y. & Nilsson Å. (2000). Deformation analyses of Ajaure Dam in Sweden, Beyond 2000 in computational geotechnics – Ten years of PLAXIS International.

Nilsson Å. & Norstedt U. (1991). Evaluation of ageing processes in two Swedish dams, ICOLD, Vienna, Q65, R2.

Design and performance of Elvington balancing and settling lagoons

A ROBERTSHAW, Yorkshire Water Services Limited, Bradford, UK
A MACDONALD, Babtie Group Limited, Glasgow, UK

SYNOPSIS. The three Elvington Balancing and Settling Reservoirs are each capable of storing 205,000 Ml. of water and were constructed in the period 1992 to 1995 to provide the owner, Yorkshire Water Services Limited, with security of supply to the major treatment works at the site. The reservoirs are of earth embankment construction with the material being won partly from excavation on the site and partly from adjacent borrow areas. A bentonite cement slurry wall was constructed as a cut-off through underlying sand and gravel layers, and the internal face of the lagoons were lined with an HDPE membrane. The total area of liner was around 95,000 m^2.

The reservoirs have been operational for around eight years and the paper will concentrate on the design aspects, in particular the bentonite cement cut-off and geomembrane. A brief description of the overall performance of the reservoirs to date will be given.

INTRODUCTION AND BACKGROUND

Elvington Water Treatment Works is owned by Yorkshire Water Services Limited and is located beside the River Derwent approximately 12km to the south east of York. The works was originally built for Sheffield Corporation in 1964 but are now one of the main source works for the Yorkshire Grid strategic transmission network which is capable of supplying customers throughout most of Yorkshire.

The primary source of water for the treatment works is the River Derwent although since 1996 water can also be brought to the site from the River Ouse at Moor Monkton approximately 20km away. The treatment works has a maximum hydraulic capacity of over 250Ml/day but the River Derwent abstraction license limits the normal average capacity to

Long-term benefits and performance of dams. Thomas Telford, London, 2004

205Ml/day which is approximately one-sixth of the company's daily demand for water.

Due to the strategic importance of the works it was decided in the mid-1980's that the raw water supplies should be protected from short-term pollution of the river and that a storage facility should be constructed which would also have the added water quality advantage of allowing the settlement of solids to take place. The Babtie Group was appointed to design the works in 1990 which were then constructed by Edmund Nuttall Ltd between 1992 and 1995.

DESIGN

General

The site chosen for the reservoirs was on a relatively flat area of agricultural ground immediately adjacent to the existing Works. Three reservoirs, each of 205Ml capacity were required, such that they could be operated in series, with one being filled, one being maintained full for at least 24 hours to allow for quality testing and settlement to take place and one being drawn down into supply.

A number of alternative design options were studied including conventional reinforced concrete tanks, combinations of earthworks and structural solutions, and earth embankment structures using cut and fill techniques to make the best use of the material available on site.

Ground investigations indicated that the sequence of geological strata was generally consistent across the site and comprised:-

Topsoil ; Upper laminated clay; Upper sand and gravel; Clay till; Lower laminated clay; Lower sand gravel; Sandstone (bedrock)

The thickness of the upper sand and gravel layer varied, but in general was no more than 600mm. At some locations it was absent.

Piezometers installed at locations across the site indicated that there was artesian pressure in the upper sand and gravel which responded within a very short space of time to water level changes in the River Derwent, which ran along the eastern boundary of the site. Any design which involved excavation into or through this layer would, therefore, have to accommodate any flows from it or uplift pressures generated within it.

In view of concerns about the suitability of the upper laminated clay for earthwork operations, a trial embankment was constructed, about 30 metres long, 5 metres high, and with side slopes of 1 in 3. Instruments were installed to allow monitoring of the formation during and after construction.

The trial embankment confirmed that the material was capable of being transported and compacted without significant difficulties. Consolidation settlement of the foundation was also found to be fairly rapid, no doubt due to the near horizontal sand lenses within the laminated clay which allowed dissipation of pore pressure into the adjacent excavated area.

Based on the results of the investigation, the decision was taken to proceed on the basis of an earthworks solution with the reservoir basins being formed by excavating down into the clay till, with perimeter and division embankments being formed, founded on the upper laminated clay. A plan showing the general layout of the reservoirs is shown in Figure 1.

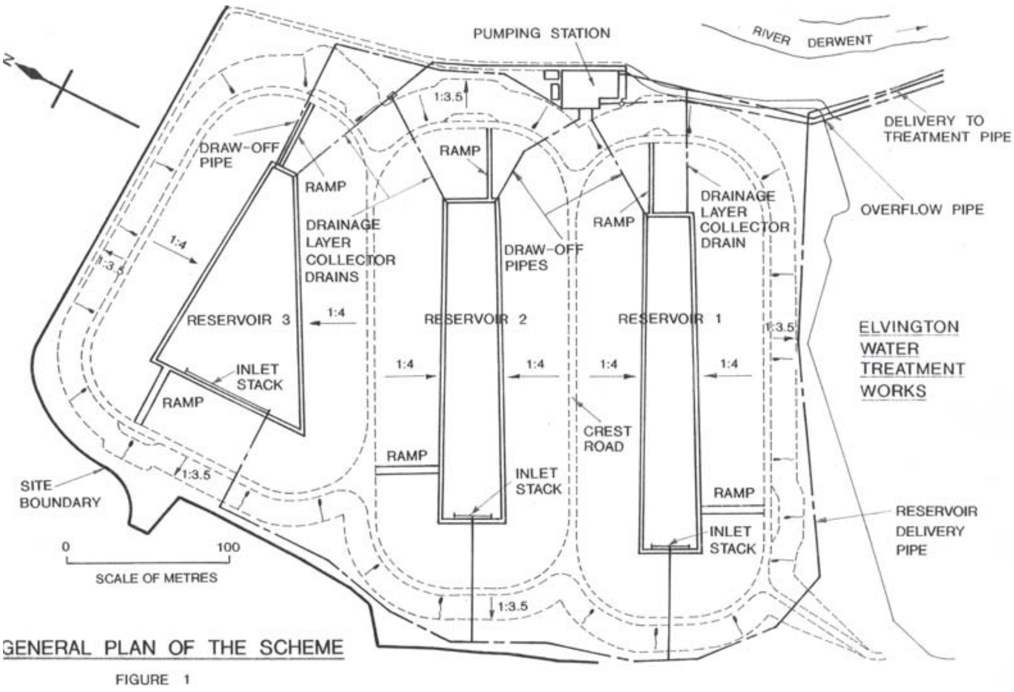


Figure 1: General plan

The decision to adopt an earthworks solution, together with Yorkshire Water’s desire to have lagoons that could be cleaned internally led to the generalised lagoon cross-section shown in Figure 2.

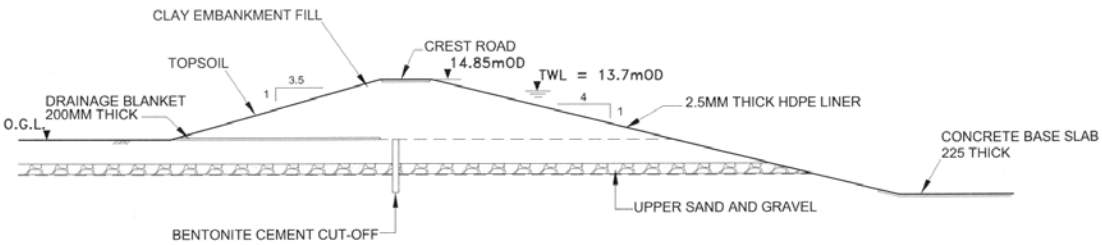


Figure 2: Typical cross-section

Embankments

To simplify construction and to make best use of site won material, it was decided that the embankment fill design parameters would be based on those of the poorest material to be excavated. This was the laminated clay, which had been found to be suitable for earthworking, provided weather conditions were reasonable. The embankments would be of homogeneous construction.

Table 1 summarises the design parameters used for the fill and for the foundation materials.

Table 1: Summary of Design Parameters

Material Type	γ_b **kN/m²**	**Cu kN/m²**	**C′ kN/m²**	**Ø′ Deg.**	**u′**	**M/C %**
1.Embankment Fill	19.3	50	0	25	0.3	26.5
2.Upper laminated clay	19.3	80	0	25	0.3	27
3.Upper sand gravel	18	0	0	32	0.25	-
4.Clay till	21.9	100	0	29	0.25	14
5. Lower laminated clay	19.3	100	0	25	0.3	27
6. Lower sand and gravel (and weathered rock)	19	0	0	36	0.25	-
7. Bedrock sandstone (unweathered)	25	N/A	N/A	N/A	0.20	-

Table 2 summarises the design conditions considered and the corresponding factors of safety for the embankments.

Table 2: Summary of Design Conditions

Design Condition	**Factor of Safety**
1.Reservoir water level at 13.70m OD and liner undamaged	
Upstream slope	>4.63
2.Reservoir empty and liner acting as an impermeable barrier	
Upstream slope	1.46
3.Rapid drawdown, liner badly damaged	
Upstream slope	1.12
4.Reservoir full, liner badly damaged	
Downstream slope	1.39

A 200mm thick drainage blanket was installed at foundation level on the downstream shoulder of the embankments, just downstream of the cut-off. The effect of this has been ignored in design conditions, case 4, for the downstream slope, and the analysis can, therefore, be considered conservative.

While seismic effects were not considered in the original design, the situation was reviewed following the publication of “An Engineering Guide to Seismic Risk to Dams in the United Kingdom” (Ref. 1) and its associated application note (Ref. 2). The embankments fall into Category “II” of this guide and as there are no factors particularly vulnerable to damage by earthquake, a seismic analysis was not considered necessary.

Bentonite Cement Cut-off

A bentonite cement cut-off trench, 0.6m wide, was constructed below formation level of each embankment along the approximate line of the embankment crests of all three reservoirs. The purpose of the cut-off was to reduce seepage beneath the embankments and to isolate the foundation and underdrainage system from groundwater in the surrounding land as well as from neighbouring reservoirs. The cut-off, therefore, prevents the river charging the reservoir and closes a potential leakage path from the reservoirs. It also prevents the liner system being subjected to uplift pressure higher than the design values.

The base of the trench was generally the deeper of 1.5 metre below the top of the clay till and 4 metres below top soil strip level. The minimum depth of 4 metres was a requirement to ensure that local variations in the clay till level were catered for.

The design requirements of the bentonite cement hardened slurry were specified as:

- Permeability to be less than 10^{-8} metres/sec under water head of 12 metres at 28 days;
- A minimum strain of 5% without failure by cracking at 90 days;
- A minimum strength of $80kN/m^2$.

Reservoir Liner

The lining system to the reservoirs had to be capable of being cleaned on an intermittent basis to remove silt, etc. In addition, it had to be capable of dealing with uplift forces in the event that the bentonite cement cut-off failed to operate efficiently.

A number of options were considered for the liner including asphaltic concrete, geosynthetic liners, and reinforced concrete slabbing. As Yorkshire Water were keen to be able to run vehicles on the base of the lagoons for cleaning and maintenance purposes, the design adopted was a combination of reinforced concrete slabs in the base and high density polyethylene (HDPE) membrane on the side slopes.

Each reservoir has a reinforced 225mm thick concrete base slab which continues up the internal slope for 2 metres. The slabs were formed in-situ with C40/20 concrete. Pressure relief valves were cast into the bases to prevent unacceptable uplift pressures developing. The slabs were founded on a 250mm thick drainage layer on top of the clay till, connected to a pumped herringbone underdrainage system.

Around $95,000m^2$ of HDPE liner were required and its use in an exposed location such as at Elvington was most unusual. However, in this instance, it was chosen after careful consideration of a number of factors including its ability to accommodate differential settlement along the embankment fill and cut slopes, durability and cost. The cost benefit analysis undertaken for comparing alternative liners, assumed complete membrane replacement after 15 years, although manufacturers were prepared to guarantee the material for up to 25 years.

A 2.5mm thick HDPE membrane was specified. The liner was laid on top of a 200m thick drainage layer of granular material, connected to the base slab underdrainage system. Some of the key material parameters for the liner are given in Table 3.

Table 3: HDPE Liner Technical Data

Properties	**Units**
Density	0.94 g/cc min
Carbon Black %	2-3%
Tensile Properties:	
- Strength at Yield	16 Mpa/inch width
- Strength at Break	27 Mpa/inch width
- Elongation at Yield	13%
- Elongation at Break	700%
Tear Resistance	289 N
Puncture Resistance	578 N

To counteract uplift forces due to the design wind speed of 45 m/s, the liner was bolted onto concrete anchor beams which run down the internal slopes. At the base, an HDPE connection piece was cast into the concrete base slabs (Ref 3).

Overflow Provisions

The main overflow system for the reservoirs is within the wet well of the reservoir pumping station. Each reservoir has its own double sided weir, separated from adjacent weirs by concrete dividing walls. There are no valves on the pipelines between the reservoirs and the control structure. The water in the wet well rises with the reservoir level up to the weir level of 13.7m AOD. Any discharge over the weir goes into an overflow channel and then into a 1650mm diameter overflow pipe. The overflow pipe discharges into the River Derwent downstream of the supply abstraction point.

The inflow pumps have variable speed drives and will normally be operated at 205 Ml/d i.e. equivalent to the capacity of each lagoon. However, they can be stepped up to a maximum inflow of 324 Ml/d.

Shortly after reservoir construction was complete, it was decided by Yorkshire Water that a second supply source should be added to the system, from their Moor Monkton intake on the River Ouse. The result of this was to increase the normal service maximum inflow from the Derwent and Moor Monkton sources to 355Ml/d, with an absolute maximum of 474Ml/d.

Under the various inflow conditions, the freeboard to embankment crest over stillwater level is shown in Table 4.

Table 4: Reservoir Freeboard

Inflow (Ml/d)	**Freeboard (min)**
0	1.15m
205	0.62m
355	0.35m
474	0.24m

To provide additional security, high level overflow weirs were constructed between adjoining reservoirs as part of the Moor Monkton contract. Each weir is 7m long and discharges freely down the HDPE slope of the adjoining reservoir. These weirs are set at a level of 14.3m AOD, 600mm above normal top water level.

CONSTRUCTION

General

The contract for the lagoons was awarded to Edmund Nuttall Limited and work started on site in November 1992. The completion certificate for the works was issued in August 1995. The Final Certificate for the lagoons under the Reservoirs Act 1975 (Ref. 4) was issued on 13th June 2000.

Embankment Construction

The earth embankments were constructed over two seasons in 1993 and 1994. A method specification was included for compaction.

The material was generally excavated and placed using tractors and scraper boxes. The embankments were built up in layers 225mm thick. At the start of construction the placed material was compacted by 6 passes of a towed tamping roller but was subsequently reduced to 4 passes. The Contractor also elected to change to using a self-propelled CAT 815 wedge foot roller (dead weight 20 tonnes); the layer thickness remained the same.

Laboratory tests were carried out regularly on samples from the placed embankment fill. The results are given in Table 5. Results showed a surprisingly high, 23%, number of undrained shear strength results which were below the value assumed in the design. The majority of these low results were from areas where laminated clay had been placed and it was thought that the presence of laminations within the samples was causing premature failure under test and did not truly represent the behaviour of the mass fill. The material from each sample was mixed to eliminate these laminations, compacted and retested and results similar or better than the design assumptions for undrained shear strength were obtained.

Table 5: Summary of Earthwork Test Results

	Cu (kN/m^2)	**γb (Kg/m^3)**	**M/C (%)**	**γd (Kg/m^3)**
Mean	78	2015	21.1	1671
Maximum	221	2086	25.1	2014
Minimum	31	1953	6.0	1388

Bentonite Cement Cut-Off

The construction of the cut-off was sub-contracted to AMEC Civil Engineering. Although the design mix had to be approved by the Engineer it was the responsibility of the sub-contractor to design a slurry satisfying the specified requirements.

The design mix changed several times in the early stages of construction because the permeability design criteria were not being achieved. A second and sometimes third wall was constructed parallel to the first sections. The accepted cut-off was approximately 1950m long. Mix-specific characteristics are given in Table 6.

Table 6: Mix-specific Characteristics

Reference	**Mix Characteristics**
Blue	4.5% bentonite Oil Companies Materials
	Association grade (OCMA)
	90 second mix
	Single Hany mixer
Orange	5.4% bentonite (OCMA)
	180 second mix
	Single Hany mixer
Green	5.4% bentonite (OCMA)
	300 second mix
	Double Hany mix
Pink	4.5% bentonite (OCMA)
	300 second mix
	Double Hany mixer
Yellow	4.5% bentonite Civil Engineering (CE) Grade
	300 second mix
	Double Hany mixer

Constituents common to all mixes were:

- 112kg of ordinary Portland cement to BS12
- 336kg of ground granulated blast surface slag to BS6699: 1986
- 2799 litres of potable mains water

The consistency of the slurry was checked daily for compliance with the specification using a Marsh cone and a mud density balance.

The following tests on the hardened slurry were carried out on a regular basis:

- Unconfined compressive strength to BS1377 Part 7 Test 7 with 2% strain rate at a minimum of 28 days
- Permeability in a triaxial cell to BS1377 Part 6 1990 Test 6 constant head test at approximately 28 days
- Consolidated drained triaxial compression test to BS1377 Part 8 Test 8 for 5% strain at 90 days

Tests carried out on the pink and yellow mixes indicated compliance with the specification. The other three mixes generally failed to meet specified requirements for permeability. The Contractor had used short hydration periods for the bentonite in the blue, orange and green mixes, whereas the final two mixes adopted a minimum 24 hour hydration period prior to mixing. Microscopic examination of the mixes indicated a "balling" effect in the first three mixes which it was felt was caused by a lack of hydration of the bentonite. In addition, the OCMA grade bentonite has a more angular grain shape than the CE grade, which made adequate hydration and mixing times even more important.

<u>HDPE Liner</u>
During the manufacture of the membrane, samples were taken and tested in accordance with ASTMD638. A quality control certificate was issued with the material. The liner was manufactured by the Gundle Lining Construction Corporation. Installation started in September 1994 and was completed in May 1995 with work being suspended between November 1994 and mid-March 1995.

Two types of welds were used on site (Ref. 3). Where panels overalapped, a hot shoe double fusion weld was formed, and where the liner attached to anchors, and elsewhere where the sheets did not overlap, extrusion welds were used. Destructive and non-destructive tests were carried out on both types of weld. The non-destructive tests consisted of air pressure testing the double fusion welds and spark testing the extrusion welds. The destructive tests were "peel" and "shear" tests. In addition to checking the integrity of the site welds, these tests were also used to check the welding equipment on a daily basis prior to it being used on site. A non-destructive test was carried out on every weld.

Instrumentation

Sensors to measure underdrainage flows and geotechnical instruments were installed to provide information throughout the service life of the reservoirs. The inflow into the chambers is measured by ultrasonic sensors upstream of v-notch weirs and the sensor values are relayed to the main control room. There are also level probes in the chambers which set off an alarm if the water level rises above a certain level.

Geotechnical instrumentation consisted of hydraulic, pneumatic and standpipe piezometers to measure pore and uplift pressures and vertical extensometers to monitor formation settlement

Survey pins were installed along the crest to allow the settlement of the embankment to be monitored. The spacing of the pins is generally 20m.

PERFORMANCE

General

One of the recommendations contained in the Final Certificate stated that annual performance assessment reports should be prepared to provide guidance on the significance of the behaviour of the reservoir, its foundations and the surrounding ground as revealed by instrumentation monitoring and visual inspections. Such reports have been prepared for Yorkshire Water Services by TEAM (an amalgamation of MWH and Arup) under the supervison of a Panel AR Engineer. In addition the reservoir was inspected under Section 10 of the Reservoirs Act 1975 for the first time in June 2002. These reports have all confirmed that the reservoir is behaving in a satisfactory manner.

Settlement

In general the settlement of the embankments has been minimal. However, soon after construction was complete, a depression appeared around the top of the magnetic extensometer on the main embankment of lagoon 1. This extensometer had also become blocked soon after installation. The depression covered an area of approximately 2 metres by 1 metre and had a maximum depth of approximately 0.2m causing cracking of the crest road surface and extending under the top of the liner. After a period of close observation it appeared that the settlement had ceased so the area was reinstated in 2000 and no movement has been observed since. The precise cause of the settlement is unknown but it was concluded that the depression must have been caused by local irregularities during installation of the extensometer.

Interior of the lagoons

The design brief for the lagoons stated that the sides and bases of the reservoirs should have smooth surfaces to facilitate the removal of accumulated sludge and other debris. However the geomembrane liner as constructed has a much folded and wrinkled appearance which has proved difficult to clean and detracts from the appearance of the structure. In addition a number of small splits in the HDPE liner have been found which have been easily repaired and have not caused any concern.

All three reservoirs have now been emptied for cleaning. Each time that a reservoir has been refilled after cleaning a rapid increase in underdrain flow has occurred. For example, Reservoir 1 has a normal base flow of less than 0.3 litres / min but after refilling in 1999 the flow suddenly increased to 3.5 litres / min before reducing to its previous value over a period of approximately 3 months. This action has been attributed to self-seating of the pressure-relief valves in the base slabs and the subsequent re-deposition of silt.

CONCLUSIONS

The lagoons at Elvington were innovative in their use of exposed HDPE liner on such a large scale. However, it was an economical material to use, easy to install, and has performed well in service.

The problems that were encountered with the bentonite cement cut-off highlighted the need for adequate hydration times for bentonite and also the differences between OCMA and CE grade materials. Despite this the cut-off appears to be performing well in minimising uplift pressures on the liner system.

The reservoirs are now forming an important element in Yorkshire Water's supply network and are giving increased security of supply to over 4.5 million customers. Long-term liner performance will remain an interest but with three lagoons, remedial or replacement work should be able to be carried out sequentially without a material effect on supply.

ACKNOWLEDGEMENTS
The authors gratefully acknowledge the permission of Yorkshire Water Services to publish this paper.

REFERENCES:

1. Building Research Establishment, *An Engineering Guide to Seismic Risk to Dams in the UK,* 1991
2. The Institution of Civil Engineers, *An Application Note to An Engineering Guide to Seismic Risk to Dams in the UK,* 1998
3. Murphy, (1996*). The Design and Installation of an Exposed High Density Polyethylene Reservoir Liner*
4. Reservoirs Act 1975, HMSO, London

Twenty five years experience using bituminous geomembranes as upstream waterproofing for structures

M. TURLEY, Colas Limited UK
J-L. GAUTIER, Colas SA, France,

SYNOPSIS.
Since 1978, with minor variations in the structure details, more than twenty rockfill or earthfill dams have been constructed over the last 25 years using a thin bituminous geomembrane for the upstream waterproof facing. This paper illustrates, using the examples of the last two dams built in France, the 38 metre high “L’Ortolo” dam in 1996 in Corsica and the 43 metre high “La Galaube” dam in 2000 near Carcassonne, the details of this construction technique which proves to be an efficient and economical alternative to dense asphalt or clay waterproofing structures.

INTRODUCTION
The modern use of bitumen as a geomembrane waterproofing layer for dam facings commenced in the early 1970’s with the *in-situ* impregnation of a synthetic geotextile placed onto a prepared substrate, and impregnated with bitumen sprayed onto the material at a high temperature – typically in excess of 180°C. The first application of this form of sprayed geomembrane was in lining ponds in 1973-74 and around the same time, on small dams such as the dam of “Les Bimes” (9 metres high) or the dam of “Pierrefeu” (8 metres high), both of them located in the south east of France.
This, although an effective method of producing a waterproof and seamless geomembrane layer, had major inherent drawbacks both in terms of operator safety and quality control. The process was sensitive to moisture, and the bitumen usage was difficult to control leading to variations in thickness. The introduction of prefabricated geomembranes manufactured using bitumen impregnated geotextile under quality-controlled factory conditions removed these failings, leading to greater confidence in their use. There was a transition to the use of the prefabricated geomembranes in the mid 1970’s.

Long-term benefits and performance of dams. Thomas Telford, London, 2004

THE PREFABRICATED GEOMEMBRANE

The factory production of a prefabricated geomembrane of bitumen impregnated geotextile, reinforced with glass fleece, addressed the safety and quality problems. Initially, rolls of material 4.0m wide were produced on a small factory-production plant and laid on site in strips. These strips were overlapped at the edges and joined by forming seams in the material using propane gas torches to heat-weld the overlap. A few small dams were waterproofed according to this technique such as “Locmine” in Brittany, (7 metres high) in 1977 and “Gardel” in Guadeloupe, (14 metres high) in 1978.

The geomembrane today

The bituminous geomembrane liner in use by Colas today, Coletanche, is manufactured in a state-of-the-art factory in Galway, Eire, commissioned by the company in 2000. The facility enables a more versatile approach to production of wider rolls (5.15m) available in several thickness grades impregnated with either Oxidised (NTP grade) or Elastomeric (ES Grade) modified bitumen; typically the “NTP” 3 grade is used for dam lining.

Table 1: Physical characteristics of Coletanche NTP3

Characteristic:	Property:
Roll length	65m
Roll Width	5.15m
Material thickness	4.8
Mass per unit area	5.5kg/m^2

The structure of the geomembrane

The base structure of the geomembrane is illustrated in Figure 1 and comprises:

- A non-woven polyester geotextile, whose weight per m^2 determines the ultimate thickness of the geomembrane – between 3.5 to 5.6mm. Coletanche NTP3 is 4.8mm thick with a mass of 5.5kg/m^2.
- A glass fleece reinforcement (which contributes to the strength of the geomembrane and stability during fabrication).
- Total impregnation with a compound including a blown bitumen of 100/40 pen plus filler (NTP grade), or an elastomeric modified bitumen (ES grade)
- The underside is coated with a Terphane film bonded when the membrane is hot, and designed to give resistance to penetration from tree roots.
- Finally the upper surface is coated with a fine sand to a) provide greater traction on a slope, giving greater operator safety and security from slipping, and b) to give protection from the degrading effects of UV radiation.

Among the properties of this geomembrane the significant mechanical characteristics are shown below in table 2.
The material is now used for a wide variety of environmental and hydraulic applications from lining canals and watercourses, groundwater protection, landfill lining and capping as well as for the waterproofing of dams.

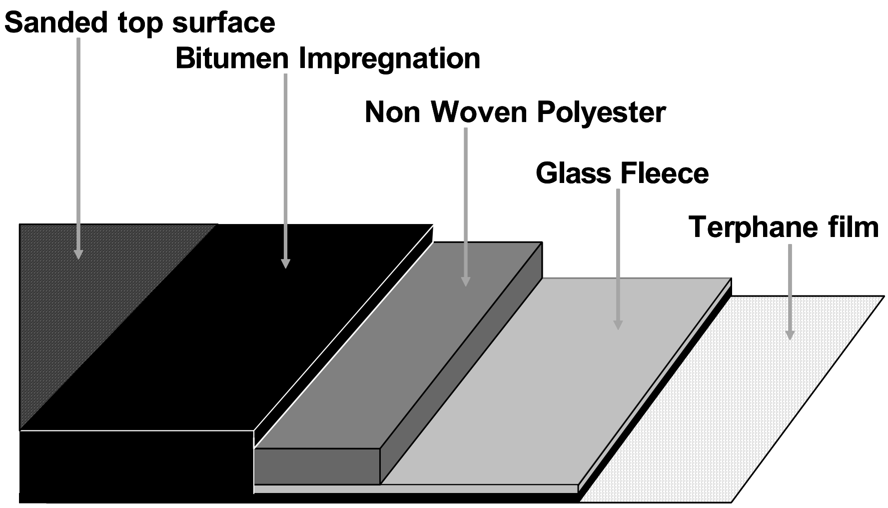


Figure 1: Typical structure of the geomembrane

Table 2: Mechanical characteristics of Coletanche NTP3

Characteristic:	Property:	Test Method:
Tensile strength at break	25,000kN/m	ISO 10319
Strain at break	70%	ISO 10319
Tear resistance	90N	NF G 07-112
Puncture resistance (Static)	500N	NF P 84-507
Puncture resistance (Dynamic)	22J	NF-p 84 502
Permeability (at 0.1 MPa)	7×10^{-14}	Darcy's Coefficient
Conventional watertightness level	$<10^{-4}$ $m^3/m^2/j$	NF-P 84515
Flexibility when cold	0°C	NF P 84-350
Maximum friction angle	35°	

Application of the geomembrane

The rolls of geomembrane are lifted and unrolled using a purpose-designed hydraulically controlled beam, carried by a tracked excavator. Lining the face of a dam, the material is unrolled down the face of the structure with the excavator remaining at the top.
The geomembrane is laid with a 20 cm overlap which is used to form a welded seam, utilising a propane gas torch to liquefy the bitumen prior to pressing and fusing the two liner sections together to form a watertight seam. This method is also used when fixing to structures, and forming round pipe-work, etc after first coating the surface with a primer.

Testing the seams

Quality control of the seams is undertaken non-destructively using ultrasound to check the integrity of the welded area, either using a single hand-held transducer for small areas or a machine known as "CAC 94", which automatically tests the full seam width using 24 ultrasound sensors arranged in a staggered row to completely cover the seam width, downloading the data onto a computer. Software designed for the machine allows a printout to be produced showing a global view of the weld and any defects: a 20cm weld is required to have a minimum 75% width continuously welded for acceptance.

PROJECTS USING REINFORCED BITUMINOUS GEOMEMBRANE:

<u>Ospedale dam</u>

The first reference project using a prefabricated bituminous geomembrane took place in 1978, with the "Ospedale" dam, located in a remote area of southern Corsica at an altitude of 1,000 metres. This was constructed as a rock fill dam with a length of 135 metres and a height of 25 metres, a 70,000 m^3 embankment volume, with slopes 1.7 to 1 upstream, 1.5 to 1 downstream, and designed to have a capacity of 3 million m^3.

An alternative design proposal

The initial design for the dam was based upon an upstream waterproof facing of a 3 layer hot bituminous-mix, making a total thickness of 24 cm. This would have proved difficult to build, due to the remoteness of the construction project and the subsequent long transportation distances for the hot mix. As an alternative to this design, a proposal was put forward based upon the following structure:

- Two regulating layers of a gravel material impregnated with bitumen emulsion to provide stability, in order to improve the profile of the rock fill embankment,
- A 5 cm thick 3/6 mm open graded cold asphalt mix with a permeability of 10^{-4} m/s, to ensure an efficient under-drainage layer and prevent back-pressure under the waterproof geomembrane in the unlikely event of damage to the membrane layer leading to leakage. This cold-mix material was manufactured on site in a purpose-designed mobile plant, laid using small hoppers lowered on hawsers.
- A 4.8mm thick bituminous geomembrane, Coletanche NTP 3, laid between two geotextiles.
- Finally a mechanical protection made of unbound interlocking pre-cast concrete blocks to reduce risk of under-pressure

There were, additionally, other considerations; the membrane must not slip under its own weight down the face of the dam, and the concrete blocks were to be able to move independent of the membrane.

In order to meet these requirements, the membrane was laid with its smooth Terphane-faced side uppermost, as the friction angle on this side varies from 20 to 28°, compared with 30 to 42° on the sanded side. This proved effective in 1983 when during a huge storm, a large section of the concrete paving slipped away, without the slightest damage to the membrane.
The geomembrane was bonded to a concrete plinth at the foot of the dam using a bituminous primer painted onto the concrete to enable the membrane to be heat-welded. A thin stainless steel strip 5 to 6cm wide was fixed into the concrete through the membrane and covered with one additional thickness of membrane for additional protection.
This alternate design was accepted by the "French Committee on Large Dams", and a paper presented during the 13th International Congress on Large Dams in New-Delhi [Bianchi et al, 1979].

<u>Projects carried out during the '80s and '90s</u>
Following the success of the Ospedale dam project, over twenty further lining projects were undertaken with bituminous geomembranes, based upon either oxidised or polymer modified bitumen, with only minor variations in the design, among which can be noted: "La Riberole" dam, for a reservoir to feed a hydroelectric power plant, with a small height of only 8 metres, but at an altitude of 1,625 metres in the Pyrenees; at "Verney" in the French Alps where the membrane is used for a different purpose, to improve the water tightness of the upstream drainage bed; the dam of "Gachet", on the island of Guadeloupe, 14metres high, where the membrane is protected by unbound pre-cast concrete block protection, and still showing very good behaviour despite the tropical weather conditions; the dam of "Mauriac", in the centre of France, which is a constant level reservoir where therefore only the upper part of the upstream facing is protected with precast concrete slabs, anchored by stainless steel cables in the upper part of the dam [Clérin et al, 1991]. Finally the last two large dams built in France, which show the use of a bituminous geomembrane on a quite different scale: "L'Ortolo" dam, in Corsica in 1996, and La Galaube" dam, near Carcassonne in Southern France in 2000, which are described in more detail below.

"L'ORTOLO" DAM
A rock fill dam of 155,000 m^3, 157 metres long, a width at the foot of 120 metres, and 38 metres in height from the base. With slopes of 1.7 to 1 upstream and 1.5 to 1 downstream it has created a reservoir of 3 million m^3. [Tisserand et al, 1997]

<u>The design:</u>

The upstream waterproofing

- 10 cm 25/50 ballast impregnated with bitumen emulsion at 3 kg/m^2

- 10 cm cold asphalt mix 4/10, with a 7 to 8 % void content and a permeability between 10^{-4} and 10^{-5} m/s, manufactured on site
- Coletanche NTP 3, covered by a 400 g/m^2 geotextile fixed at the base with a 6 mm thick and 60 mm wide stainless steel plate, bolted every 15 cm into the plinth, covered with a double thickness of membrane. The geomembrane was fixed at the crest of the dam.

The protection layer

- Fibrous in-situ cast concrete slabs 14 cm thick, 3 metres wide

Comments

Two major events occurred during the construction of this project, which delayed the completion for several months, and which led to changes in the design of the next dam. Storms and strong winds on January 10th 1996 lifted 1,500 m^2 of membrane during its installation phase. The effect of the wind was amplified by the venturi effect above the crest, and under pressure generated through the dam which was permeable to downstream winds.
This storm was followed, on February 2nd, by a 100 year flood of 270m^3/s that led to a 0.5 m high flow of water through the spillway. The dam filled up in one night, and began to leak, with a flow of 5 to 6 m^3/s through the rock fill (i.e. 2litres/s/m^2), and then emptied within 48 hours through the dam and outlet. No damage was noticed to the embankment, with only slight movement of a few rocks, and a minor settlement of 6 cm on top. However, several weeks were then needed to clean the whole area, and remove the mud and all the debris carried by the flood.
The main conclusion from this project was that the permeability of the asphalt layer below the geomembrane needed to be reduced, both to increase the suction effect in case of strong winds, and to decrease flow rates in case of flood during construction before the membrane is completely installed [Tisserand et al, 1998].

"LA GALAUBE" DAM

This rock fill dam is constructed of 800,000 m^3 of mica schists excavated from the site, on a foundation of granite and a reinforced concrete plinth upstream. With a length of 380 metres, a height of 43 metres above the foundations and slopes of 2 to 1, (26°) it is the highest dam in the world with a bituminous geomembrane as an upstream impervious face and was built in 2000 on the Alzeau River in the south of France. The reservoir created is at an altitude of 700 metres with a 68 ha surface, and a capacity of 8 million m^3. [Gautier et al, 2002]

The design

The upstream waterproofing

- 10 cm layer of non-bound material, with a 0/20 mm grading, impregnated with bitumen emulsion, to regulate the slope. This required 5,000 tonnes of limestone from a nearby quarry, deposited from the crest and leveled using 2 laser-guided bulldozers.
- 10 cm layer of cold asphalt mix, with a 0/10 mm grading, to ensure the final regulating of the upstream face before laying the geomembrane. This was to be a semi-impervious layer designed to a) reduce leakage flow through the waterproofing structure in case of accidental damage to the geomembrane, and b) to overcome the problems experienced in Corsica, of premature flooding prior to completion. Laboratory studies were carried out to determine the recipe of the cold mix asphalt required in order to achieve permeability around 10^{-6} m/s, as well as ensuring that a workable mix could be achieved.

Figure 2: Work in progress at La Galaube

- as with the previous dam at L'Ortolo, 4.8mm thick Coletanche NTP3 geomembrane was used, protected by a geotextile above to reduce the effect of underpressure in case of rapid emptying of the reservoir. As it was important to have no transverse seams in the geomembrane on the slope of the structure each individual roll was manufactured to the required length to match its final position on the dam face, with a unique reference number allocated to its position. A roll of this grade of membrane is normally 65m; some rolls, required to be in excess of 100 metres, had an ensuing weight of over 3 tonnes.

The protection layer

- Fibrous concrete manufactured on site was cast into aluminium formwork to produce 10cm thick slabs 5 m x 10 m with open joints.

The completed structure was delivered in November 2000, after less than a five month period for the waterproofing phase, and a cost of €1,500,000 (£1,050,000) for an impervious surface of more than 22,000 m^2. This allowed the owner to start the filling of the dam before winter, leading to the first outflow through the spillway 18months later.

CONCLUSION

The continuous monitoring of dams with the upstream face waterproofed using a prefabricated bituminous geomembrane, some of them for more than 25 years, has demonstrated a good performance over this period. The behaviour of these structures over the period since lining has shown no reduction in watertightness with for example constant flow rates of 2.4 l/hour/m^2 at l'Ospedale dam, or 0.9 l/hour/m^2 at Mauriac dam through the liner into to the drainage layer beneath, results that are very acceptable to he clients. [Tisserand et al, 2002].

Through this knowledge and the experience gained along all these projects, the use of a bituminous geomembrane as an upstream waterproofing facing on earth fill or rock fill dams has now proved an efficient alternative solution to inner clay barriers or dense bituminous mix facings, with installation costs that can be comparable and even more interesting in remote locations.

REFERENCES

Bianchi, C. Rocca-Serra, C. Girollet, J. (1979) *Utilisation d'un revêtement mince pour l'étanchéité d'un barrage de plus 20 mètres de hauteur.* 13ème Congrès des Grands Barrages – New Delhi

Clérin, J. Gilbert, C. Bienaimé, C. Herment, R. (1991) *Massif Central : a rockfill dam with a bituminous geomembrane upstream facing,* Travaux no. 665

Tisserand, C. Breul, B. Herment, R. (1997) *Feedback from Ortolo Dam and i's forerunners.* Geotextiles-geomembranes, Rencontres 97

Tisserand, C. Breul, B. Gimenes, Y. Antomarchi, E. Herment, R. (1998) *Le Barrage de l'Ortolo,* Travaux no. 746

Gautier, J-L. Lino, M. Carlier, D. (2002) *A record height in dam waterproofing with bituminous geomembrane: La Galaube dam on Alzeau river,* 7th International Conference on Geosynthetics, Nice

Tisserand, C. Poulain, D. Royer, P. (2002) *Durability of geomembranes in embankment dams.* Colloque CFG – CFGB – Saint Etienne

Watertightness and safety of dams using geomembranes

A M SCUERO, CARPI TECH S.A., Switzerland
G L VASCHETTI, CARPI TECH S.A., Switzerland

SYNOPSIS. Impervious prefabricated geomembranes are a well-established technology to provide or restore watertightness in dams. Since 1959, they have been installed on all types of dams worldwide. The paper gives an overview of how the geomembrane systems have been applied and perform on different types of dams. Some significant case histories on concrete gravity dams, masonry dams, fill dams, RCC dams are presented.

INTRODUCTION

Synthetic impervious geomembranes were first used on a dam in 1959. In pioneer installations, all types of geomembranes available in the market were adopted. Selection was based on local availability, on aggressiveness of marketing, on personal knowledge and information. As years went by and field results became available, selection of the geomembrane could rely also on ascertained performance. Materials that performed poorly were abandoned in favour of more flexible, robust, and durable ones. According to the ICOLD 2003 database listing 232 large dams incorporating a geomembrane for watertightness, PVC, mostly coupled to an anti-puncture geotextile, is most frequently adopted. Some other geomembranes are practically no longer used in modern applications: the last reported installation of in-situ fabricated geomembranes on a dam occurred in 1988, and of HDPE geomembranes in 1994.

Pioneer installations were performed on embankment dams. In a few years, improvement in technology allowed installation on more demanding sub-vertical facings (Scuero & Vaschetti, 1998).

Today, all types of dams have a geomembrane as an impervious element, either as a repair method or incorporated in the dam since original construction: these are summarised in Table 1.

Long-term benefits and performance of dams. Thomas Telford, London, 2004.

Table 1: Total dams by type of works

Type of dam	New construction	Rehabilitation	Unknown
Fill	75	51	39
Concrete	1	34	5
RCC	21	1	5
Total	97	86	49
Percentage	47	53	-

Table 2: Total dams by type of position of geomembrane

Geomembrane is	Total dams	Percentage
Exposed	87	39.2
Covered	135	60.8
Unknown	10	-
Total	232	100

Europe, with 111 dams out of 232, is still the leader, as already reported in 1998.

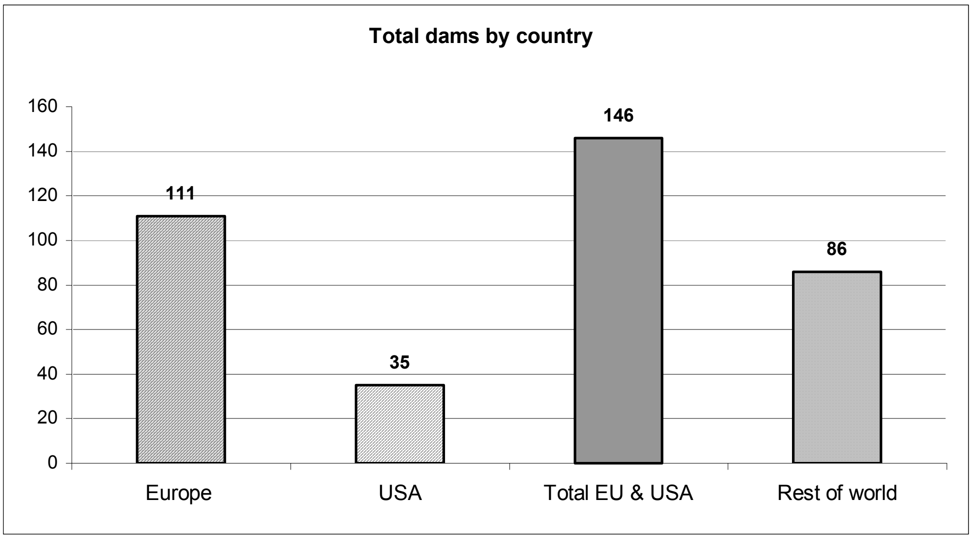


Figure 1: Total dams by country

RECENT TRENDS

The state-of-the-art systems are now widely covered in available literature and will not be discussed in this paper. One recent trend is a further increase in the adoption of drained geomembrane systems as compared to undrained systems. Among the advantages that the drained system can provide, there is the possibility of progressively dehydrating the dam, as further reported below, and of monitoring efficiency.

Another marked trend is the increase in RCC dams that incorporate a geomembrane: 27 (Scuero & Vaschetti, 2003) out of approximately 260 RCC dams, accounting for more than 10% of the total.

A recent development is also the adoption of leak detection systems based on fibre-optic cables. One of the earliest applications of this system to monitor the efficiency of geomembrane liners was made in UK, at Winscar dam (Carter et al., 2002).

The following case histories aim to give an overview of the most remarkable recent features of the geomembrane system on the various types of dams.

REHABILITATION OF CONCRETE AND MASONRY DAMS

The deterioration of concrete dams is caused by the environment (temperature changes, wetting-dehydrating and freeze-thaw cycles, impact by ice, debris, transported materials, chemical action of water) or by abnormal behaviour of the structure (expansive phenomena of concrete, problems with foundations and differential settlements). Concrete cracks and loses imperviousness, water infiltrates the dam body, and subsequent washing of fines may cause carbonation and clogging of the drains. As the drains cannot efficiently perform their function, seepage extends to the whole body of the dam and saturation of concrete occurs. Increase in pore pressure causes deviation from the initial design conditions, and stability of the structure may be at stake. In dams subject to alkali-aggregate reaction, increase in the water content aggravates the reaction. Rehabilitation generally aims to stop water infiltration and further deterioration of the structure.

On concrete dams, the liner adopted is generally a PVC geomembrane coupled during fabrication to an anti-puncture and drainage geotextile. The liner, supplied in flexible sheets, is generally installed directly over the concrete. Sometimes an additional drainage geonet is installed behind the geocomposite to enhance drainage transmission.

On masonry faced dams, the geocomposite system must meet the demanding requirements of the exceptional roughness of the substrate and its different consistency (stone and mortar in the joints). On these dams, the system is implemented by a transition anti-puncture layer, usually a thick geotextile to achieve a smoother surface without extensive civil works.

Rehabilitation of a concrete dam affected by AAR: behaviour of Pracana after 11 years

Pracana is a 65 m high buttress dam in a seismic region of Portugal, built between 1948 and 1951, and owned by EDP, Electricidade de Portugal. In 1980, the dam was taken out of operation to undertake a thorough investigation of the deterioration related to the continuous concrete expansion phenomena. Investigations ascertained the presence of a secondary alkali-aggregate reaction (AAR), which would be further activated by infiltration of water from the reservoir, creating a critical scenario in respect to sliding conditions along horizontal cracks. Installation of a drained waterproofing liner on the upstream face was deemed necessary to stop water infiltration feeding the AAR, and to avoid the possibility of water exerting uplift in the horizontal cracks, especially in the case of a seismic event.

The exposed drained PVC geomembrane system was installed in the dry season of 1992, in 5 months, concurrent with major rehabilitation works including construction of a new foundation beam and grout curtain, two sets of concrete struts on the downstream face, local grouting of larger cracks and mass grouting of smaller cracks, construction of a new spillway and a new water intake. Since 1992, the behaviour of the geocomposite waterproofing system has been monitored in respect of leakage and its capability of dehydrating the dam, reducing the water content feeding the AAR. At ICOLD's 21st Congress, the owner reported that the dam waterproofing might be assumed to contribute for the reduction of the swelling process (Liberal et al., 2003).

Figures 2 & 3: Pracana 65 m high buttress dam in Portugal, affected by AAR, was waterproofed with an exposed PVC liner in 1992. In 2003 (at right), the owner reports "the concrete dam waterproofing may be assumed to contribute for the reduction of the swelling process".

Rehabilitation of a masonry dam: Beli Iskar, 2002

Beli Iskar is a 50.7 m high rough rock masonry dam located in the highest part of Rila Mountain, in Bulgaria, at 1878.70 m a.s.l. In 1950, the upstream face was coated with “Inertol”, between 1976 and 1978 the leaking joints were repeatedly repaired with “Soral”, and the face with resin coating. These repairs were washed away. The exposed drained PVC geomembrane system was installed in 2002.

Preparation works included removal of debris and sediments to expose the rock masonry. A 2000 g/m^2 geotextile was installed on the masonry as anti-puncture protection to the PVC geocomposite. The waterproofing liner is SIBELON CNT 3750, a 2.5 mm thick PVC geomembrane coupled during fabrication to a 500 g/m^2 geotextile, and supplied in rolls as long as the section they cover.

The geocomposite is anchored along vertical lines to the masonry by the same patented tensioning assemblies used for concrete dams. The masonry surface was regularised with a layer of mortar under the anchorage assemblies, to provide an even surface for welding of the PVC sheets. Adjoining geocomposite rolls were watertight heat-welded with manual one-track method.

Figures 4 & 5: Beli Iskar 49.7 m high masonry dam, Bulgaria 2002. The exposed PVC geocomposite is installed on an anti-puncture geotextile. The dam is at 1878 m of altitude, ice thickness up to 0.60 m.

The PVC geocomposite is anchored at the perimeter with the same watertight seal, made by compression, adopted on concrete dams. In order to provide an even surface which is crucial to achieve watertightness, the masonry was regularized in the areas of the seals with a gunite layer.

A new plinth was constructed where additional grouting was required. The new grouting plinth was waterproofed with the same PVC geocomposite waterproofing the upstream face. The bottom watertight perimeter seal connects the geocomposite lining to the upstream face with the geocomposite lining the plinth.

LONG-TERM BENEFITS AND PERFORMANCE OF DAMS

EMBANKMENT DAMS

In embankment dams, watertightness must be provided by a material different from the materials constituting the dam body. Traditionally, the barrier to water infiltration had been made with either natural or man-made materials, by constructing either an impervious core using materials such as asphaltic concrete or clay, or an impervious upstream face as in Concrete Faced Rockfill Dams (CFRD) and embankment dams with an asphalt concrete upstream layer.

The use of traditional materials such as clay or asphaltic concrete requires adequate materials selection, placing and compaction. Sometimes appropriate materials are not available at reasonable costs, or available materials do not have the required quality, or construction is difficult, time consuming and expensive. Sometimes inadequate construction, or inadequate weathering resistance, for example with some asphaltic concrete facings, leads to increasing leakage over time. The final result may end up with general poor performance of the dam.

CFRDs in turn involve complicated design, and construction of the face slab and of its complex waterstop system can significantly affect the overall construction time. Time schedules have often been extended considerably beyond what was initially foreseen, especially when contractors have had no previous experience with this type of construction. From a performance perspective, although installation of the concrete layer occurs when placing of the fill has been completed and the anticipated major settlements have taken place, in many CFRD's already built the settlement of the fill continued after the filling of the reservoir. There are dams where the deformation due to the settlement, combined with the deformation of the fill due to the hydraulic head, provoked cracks in the concrete face and/or failure of the waterstop systems. This problem, related to the placement of a rigid element (the waterproofing concrete face) over the deformable fill dam body, is aggravated in dams constructed in seismic areas.

In new construction, the use of synthetic impervious geomembranes avoids the problems connected with design and installation of multiple defence lines of waterstops, with deterioration of waterstops, with connections of the core or of the asphalt concrete to concrete structures. In case suitable traditional materials are not available at reasonable cost, the geomembrane option can make the project feasible. In new construction as well as rehabilitation, the geomembrane can maintain watertightness in the presence of relative movements of the dam and of differential settlements of the fill.

With very few exceptions, mostly in Chinese dams, the PVC geomembrane is installed in the upstream position. This is technically preferred as it minimises uncontrolled water presence in the dam body, improving safety. Exposed upstream membrane systems also allow easy inspection, and have lower construction times and costs.

Specific aspects to be addressed for rehabilitation of embankment dams are the face and perimeter anchorage systems, which are designed depending on the type and strength of the existing facing (asphaltic concrete or concrete). An outstanding example of rehabilitation of asphaltic concrete facing has been Winscar dam in UK. The following case history addresses a recent repair of CFRD.

Rehabilitation of embankment dams: Strawberry CFRD, 2002

Strawberry, the second oldest concrete faced rockfill dam in the world, is located in the USA. Owned by Pacific Gas and Electric Co. (PG&E), the dam is 43 m high and about 220 m long, and has 9 vertical joints that did not have any waterstops installed. Deterioration of joints caused increasing leakage that over the years became unacceptable to California dam safety officials.

To permanently reduce leakage, PG&E selected for Strawberry an external waterstop system that was a development of the concept adopted for exposed geomembrane systems on entire face dams since 1976. The exposed waterstop system, which is patented, consists of a PVC geocomposite installed at the joints over a support layer. The waterproofing liner intrudes in the active joint at maximum opening of joint under the maximum water head. The geocomposite is watertight anchored along the perimeter and is left exposed to the water of the reservoir.

Differently from conventional embedded waterstops, which allow deformation only of few millimetres in the central portion of the bulb, the external waterstop can deform along the entire width of the PVC geocomposite (typically 40 to 60 cm), hence it is capable of accommodating significant movements that may occur in the joint. The system is the same installed to waterproof the contraction joints of RCC dams (see below).

At Strawberry the external waterstop, patented, consisted of four layers. The first two layers, Layers 1 and 2, anchored along edges on both sides by impact anchors into the existing sound concrete or in new shotcrete which replaced excessively damaged concrete, were a geocomposite made by a polyvinyl chloride (PVC) membrane 2.5 mm thick with a 500 g/m^2 geotextile laminated to it. These first two layers acted as anti-intrusion support on the joints, and were anchored so that they could move independently of each other. Layer No. 3 was a non-woven geotextile anchored along its vertical edges. This layer had an anti-friction purpose, to avoid layer No. 4 being affected by the movements of the bottom layers. Layer 4, the waterproofing liner, was the same PVC geocomposite as used in the first two layers, and was anchored at about 15 cm centres with a stainless steel batten strip along the perimeter edges. Watertightness of this perimeter seal was obtained by a compression system of the same type successfully used and tested up to 2.5 MPa of head. The completed joint liner system was about 1 m wide.

The works were staged in three phases: phase 1, improving the access road, phase 2, installation of the geomembrane system on 6 joints, and phase 3, installation of the geomembrane system on the remaining 3 joints that were not replaced in Phase 2.

Although the Phase 3 work is yet to be performed and the reservoir was filled in May 2002, and the recorded leakage has been about 85 percent below the 1998 leakage rates, more than adequately meeting the acceptance criteria (75 % requested for the all joints) and well below the historic levels.


Figures 6 & 7: On the left, Strawberry 43 m high CFRD, USA 2002. The exposed PVC geocomposite was installed as external waterstop on the failing joints. On the right, Salt Springs 101 m high CFRD in USA. The exposed PVC geocomposite will be installed in 2004 to stop seepage across the upper 2/3 of the dam face.

RCC DAMS

The geomembrane system has been installed in new constructions to provide watertightness to the entire upstream face, or to waterproof contraction joints as an external waterstop, and in rehabilitation, to waterproof failing joints/ cracks.

In new construction, the geomembrane system has been typically adopted in RCC dams of the low cementitious content type, where it allows separation of the static function, provided by the RCC, from the waterproofing function. One main advantage of the geomembrane is that it avoids water seepage at lift joints, reducing design uplift, and the risk that water can hydro-jack the lifts. Also some design constraints can be significantly reduced, such as the need of a conventional concrete layer on the upstream face, and the need for bedding mixes or special paste treatment of the joints. This leads to the possibility of placing one RCC mix over the entire cross section of the dam without the interference caused by bedding mix and conventional concrete. Inferior quantities of cement, pozzolan, fly ash, less stringent properties for aggregates, deletion of provisional sum for cooling, wider weather placement period, can constitute additional benefits.

Two conceptual systems are available: the covered system and the exposed system. The covered system was developed and patented in USA, where the first installation was made in 1984 (Winchester dam, now Carroll E. Ecton dam, Kentucky). The exposed system, an evolution of the exposed geomembrane system developed and used since the 1950s for repair of concrete and embankment dams, was developed and patented in Italy and was first adopted in 1990 (Riou RCC dam, France).

Table 3: Synthetic geomembrane systems on RCC dams

Country	Total dams	Geomembrane on	Position
Angola	1	EF*	Covered
Argentina	1	EF	Covered
Australia	1	EF	Covered
Brazil	1	J**, C***	Exposed
China	3	EF	Exposed
Colombia	2	EF, J	Exposed
France	1	EF	Exposed
Greece	1	J, C	Exposed
Honduras	2	EF	Exposed
Indonesia	1	EF	Exposed
Jordan	1	BF****	Exposed
Mexico	1	EF	Covered
Turkey	1	EF	Covered
USA	10	EF	Covered 9 Exposed 1
Total	27		Covered 14 Exposed 13

* EF: entire upstream face
** J: induced joints
*** C: repair of cracks
**** BF: bottom section of upstream face.

LONG-TERM BENEFITS AND PERFORMANCE OF DAMS

New construction: Miel I, Colombia 2002, and Olivenhain, 2003
Miel I is a straight gravity dam constructed in a narrow gorge in Colombia. At 188 m, it is the world's highest RCC dam. The dam crest, at an elevation of 454 m, is 354 m long and the entire upstream face is 31,500 m^2.

To meet the contractual schedule, the original design of an upstream face made of slip formed reinforced concrete was changed to a drained exposed PVC geomembrane system, placed on a 0.4 m wide zone of grout enriched vibrated RCC. Due to the height of the exposed dam face, this double protection was considered necessary (Marulanda et al. 2002). The use of grout enriched RCC allowed applying good compaction of RCC mix at the dam face, assuring a good finishing of the upstream concrete surface. The RCC mix has a cement content of 85 to 160 kg/m^3; total RCC volume is 1,745,000 m^3. Contraction joints are placed every 18.5 m.

The waterproofing liner is a composite geomembrane, consisting of PVC geomembrane laminated to a 500 g/m^2 polypropylene nonwoven geotextile. In the deepest 62 m the PVC geomembrane is 3 mm thick, in the top 120 m it is 2.5 mm thick. The PVC geocomposite was installed over the upstream surface, after removal of the formwork. At the contraction joints, two layers of sacrificial geocomposite, of the same type used for the waterproofing liner, provide support-avoiding intrusion of the liner in the active joint.

The attachment system for the PVC geocomposite on the dam face is made by parallel lines of tensioning profiles assemblies, placed at 3.70 m spacing. Where the water head is higher, from elevation 268 m to 358 m, the stainless steel profiles have a central reinforcement. A seal watertight against water in pressure up to 240 m fastens the PVC geocomposite along all peripheries.

The drainage system, consisting of the geotextile attached to the PVC geomembrane, of the vertical conduits made by the tensioning assemblies, of longitudinal collectors and of transverse discharge pipes, is divided into 4 horizontal sections (compartments). Each horizontal compartment is in turn divided into vertical sections with a separate discharge. Each compartment discharges in the gallery located at its bottom. In total there are 45 separate compartments, achieving very accurate monitoring of the behaviour of the waterproofing system.

To allow installation of the PVC waterproofing system to take place independently of the RCC activities, a railing system was attached to the dam face at a first level of 360 m, approximately 90 m above foundation, and then moved to a second level of 407 m, some 140 m above foundation.

The change in design allowed the schedule to be met. Construction of the dam started in April 2000 and ended in June 2002, for a total of 26 months.

Olivenhain is an RCC gravity dam 788 m long and 97 m high completed in summer 2003 in California. It is the highest and largest RCC dam in USA. The dam is a key element of the Emergency Storage Project (ESP) of the San Diego County Water Authority, owner of the dam. About 90% of water is brought to San Diego from hundreds of miles away, and the aqueducts cross several large active faults, including the San Andreas fault. The ESP is a multi year program that will provide water to the San Diego region in case of an interruption in water delivery deriving from an earthquake or drought, thus allowing time to repair the aqueduct.

The geomembrane system was selected because it was deemed that, following a seismic event, it would have been able to prevent seepage losses through any resulting cracks (Kline et al., 2002). Furthermore, the geomembrane liner and its associated face drainage system were considered to be two features that would tend to reduce the uplift pressure.

The waterproofing system is conceptually the same adopted at Miel I. The PVC geomembrane is 2.5 mm thick, and the drainage system differs in that the number of compartments is lower, and there is an additional drainage geonet over the entire upstream face.

Figures 8 & 9: On the left, Miel I in Colombia, 2002, at 188 m the highest RCC dam in the world. Total leakage from the exposed PVC geocomposite at fully impounded reservoir is 2 l/s, mostly coming from abutments. At Olivenhain 97 m high dam on the right, USA 2003, highest and largest RCC dam in USA, the exposed PVC geocomposite was adopted to assure watertightness in case of a seismic event. At 50% impounded reservoir total leakage from 38,880 m^2 upstream face is 0.18 l/s.

Exposed waterstop: Porce II, Colombia 2000

Porce II is a 118 m high RCC dam constructed in 2000 in Colombia. Its upstream face is formed by curb extruder. The contraction joints, at average spacing of 35 m, are waterproofed with a patented external waterstop system.

In detail, at Porce II the system consists of:

- Support structure: proceeding from the dam body towards the reservoir, support is made by two stainless steel plates having a strip of Teflon in-between to decrease friction and allow sliding of the plates, of one 2,000 g/m² polyester geotextile providing anti-puncture protection against the sliding edges of the steel plates, and of one sacrifice/supporting PVC geocomposite. The steel support impedes intrusion of the waterproofing liner into the active joint, and the flexible components provide extra support and a low friction element, so that the movements of the joint occur without affecting the waterproofing geocomposite
- Waterproofing liner: a PVC geocomposite, SIBELON CNT 5050, consisting of a 3.5 mm thick PVC geomembrane heat-coupled during extrusion to a 500 g/m^2 polyester geotextile. The geocomposite is exposed and centred on the joint and watertight anchored at the periphery by flat stainless steel batten strips compressing it against the concrete, regularised by trimming the offsets and by a layer of epoxy resin. Synthetic gaskets distribute stress to achieve even compression. At plinth, the PVC geocomposite is connected directly against the rock.

Rehabilitation: Platanovryssi, 2002

The external waterstop system has been used for the underwater repair of Platanovryssi, a 95 m high RCC dam in Greece. Platanovryssi was completed in 1998 and is at present the highest RCC dam in Europe. The dam, of the high cementitious content type, was designed to be impermeable in its whole RCC mass. The vertical contraction joints were waterproofed with 12 Carpi external waterstop system during construction.

On first filling, after mid-December 1999, seepage started increasing, and reached more than 21 l/s at the end of May, dropped again and then increased to a maximum of 30.56 l/s on 10 October 2000. The cause of the leak was what appeared at first as a hairline crack in the gallery, then on the upstream and downstream face, with a maximum opening of 25 mm. The crack was approximately 20 m long. Repair works were scheduled for Spring 2002. Due to an unusually dry season, the owner could not afford to lose the volume of the water to empty the reservoir in order to work in dry conditions. In addition to that restriction, when Platanovryssi is emptied the pumped storage scheme of Thissavros cannot operate, with serious implications to the production system. It was considerably more cost effective to do the work underwater (Papadopoulos 2002).

The system selected for the underwater installation on the crack in 2002 was the same conceptual system that had been selected and installed on the vertical contraction joints during construction of the dam. The materials that constitute the support layers and the waterproofing liner for the crack have a different thickness to those used for the vertical contraction joints, due to the different hydraulic load they sustain at the joints, and that they will have to sustain at the crack.

The repair system installed on the crack consists of:

- Support structure: 2 layers of sacrificial geocomposite, of the same type used for the waterproofing liner, each anchored independently
- Waterproofing liner: PVC geocomposite SIBELON CNT 3750, installed over the joints and left exposed except for the upper 10 m above pool level where it is covered by an independent steel plate which provides mechanical protection. The PVC geocomposite is anchored along the perimeter by a watertight mechanical seal made by stainless steel batten strips compressing it against the RCC.

The waterproofing system was installed in the dry from crest level at 227.50 m to elevation 225 m, and underwater from elevation 225 m down to elevation 208 m. To facilitate underwater works, a special steel frame was constructed and lowered into the reservoir to serve as a template for placement of the perimeter seal.

Works started on April 22, 2002 and were completed on May 23, 2002. The leak through the dam has been fully stopped and the downstream face in the vicinity of the previous leaking joint is now dry.

Figures 10 & 11: On the left, the external waterstops on the contraction joints of Porce II 118 m high RCC dam (Colombia 2000). The same external waterstops were installed in 1998 on Platanovryssi 95 m high RCC dam in Greece. In 2002, the external waterstop was used again at Platanovryssi, to repair a new crack that formed during first impoundment. Installation was performed underwater (on right).

PERFORMANCE

Durability of geomembrane systems is estimated in several decades. Among others ENEL, the Italian National Power Board, is monitoring behaviour in harsh climates of PVC drained geomembrane systems installed since 1976 on 6 of its dams at high elevation (max. 2,378 m). ENEL reports that behaviour is satisfactory, and that the impermeability coefficient has remained quite constant versus time (Cazzuffi 1998).

The leakage rates from the system described are typically very low: at Miel I, in 2003 the recorded leakage at fully impounded reservoir level from the whole geomembrane system is 2 l/s, at Olivenhain, at 50% impounded reservoir total leakage from 38,880 m^2 upstream face is 0.18 l/s.

CONCLUSIONS

The techniques for waterproofing and protecting the upstream face and the joints of all types of dams with drained PVC geomembranes, has reached a high degree of sophistication and reliability. The state-of-the-art system, by constructing a continuous flexible waterproofing liner on the whole upstream face, capable of bridging construction joints and cracks, allows resisting opening of even large cracks in case of differential settlements, of seismic events, of concrete swelling. It can dehydrate the dam from saturation water, and relieve uplift pressures. Installation is quick and can be executed also underwater. With more than 25 years of maintenance free history, it has proven to be durable and reliable, and to have a standard quality not dependent on weather or on a large amount of skilled labour.

REFERENCES

Carter, I. C., Claydon, J. R., Hill, M. J. (2002). *Improving the watertightness of Winscar Reservoir.* Proc. British Dam Society Conference, Dublin, pp 415-430.

Cazzuffi D (1998*). Long Term Performance of Exposed Geomembranes on Dams in Italian Alps.* Proc. Sixth International Conference on Geosynthetics, Singapore, pp 1107-1109.

Klein, R. A. et al. (2002). *Design of Roller-Compacted concrete features for Olivenhain dam;* Proc. 22ndUSSD confer., S. Diego, pp 23-34.

Liberal, O., Silva Matos, A., Camelo, D., Soares de Pinho, A., Tavares de Castro, A., Machado Vale, J. (2003). *Observed behaviour and deterioration assessment of Pracana dam.* Proc. ICOLD 21st International Congress, Montréal, pp 185-205.

Marulanda, A. Castro, A., Rubiano, N. R. (2002). Miel I: *a 188 m high RCC dam in Colombia.* The International Journal on Hydropower & Dams vol. 9, Issue 3, pp 76-81.

Papadopoulos, D. (2002). *Seepage evolution and underwater repairs at Platanovryssi.* The International Journal on Hydropower & Dams vol. 9, Issue 6, pp 88-90.

Scuero A M, Vaschetti G L (1998). *A drained synthetic geomembrane system for rehabilitation and construction of dams.* Proc. British Dam Society Conference, Bangor, pp 359-372.

Scuero A M, Vaschetti G L (2003). *Synthetic geomembranes in RCC dams: since 1984, a reliable cost effective way to stop leakage.* Proc. 4th International Symposium on Roller Compacted Concrete (RCC) Dams, Madrid, pp 519-529.

Downstream Slope Protection with Open Stone Asphalt

A.BIEBERSTEIN, University of Karlsruhe, Karlsruhe, Germany.
N.LEGUIT, Bitumarin bv, Opijnen, The Netherlands.
J.QUEISSER, University of Karlsruhe, Karlsruhe, Germany.
R.SMITH, Hesselberg Hydro Ltd, Sheffield, UK.

SYNOPSIS
Recent investigations and results from tests on the performance of erosion protection systems in Open Stone Asphalt (OSA) on man-made embankments during overtopping conditions are presented. The paper also includes state of the art mix-designs / compositions, installation routines and experience generated from projects in Northern Europe. An analytical statical concept for revetments is presented, which was developed in a research project at University of Karlsruhe, Germany.

1. INTRODUCTION

A major problem during the construction of a dam that can be overtopped is the ability of the slopes covered with revetments to withstand hydraulic loads (e.g. Rathgeb 2001 and Dornack 2001). Therefore an engineering or geotechnical concept is necessary to ensure the stability (LfU 1997).

It is necessary to protect the downstream slope of such embankments against erosion. Four possible cases of overtopping of an embankment are considered:

<u>Overtopping arrangements</u>

1) Overflow of a man-made reservoir during / after rainstorms.
2) Overflow of flood storage reservoirs, whenever the collected rainfall of a catchment area exceeds the total drainage capacity of rivers plus the flood storage arrangements.
3) Overflow of river embankments, when the drainage capacity of the tidal basin (estuary) of a river has been reduced by a storm surge.
4) Overflow sections of river embankments, when the flood exceeds the design level.

2. CONSTRUCTION REQUIREMENTS

These overtopping cases are all emergency situations. Since the managed flood control system is only used during an overtopping event, which occurs with a low frequency, the construction can often be hidden in the landscape. The requirement to choose a long lasting and permeable construction material with plastic properties to compensate underground settlements is often combined with the request to restore the existing appearance of the embankment and to support the establishment of vegetation. Excellent hydraulic performance under extreme conditions, together with low or no maintenance cost to the materials are desirable for the erosion protection in stand-by mode.

Open Stone Asphalt

To fulfil the function of a durable, maintenance-free erosion resistant material together with a green appearance, Open Stone Asphalt is often selected. Based on experience gained from seafronts, river and canal revetments and reservoir erosion protection on the upstream faces of dams the material is now being used on the downstream slope of embankments. The bituminous mix consists of single sized crushed stone coated with sand mastic. The design of the mastic coating is based on a sand/filler mixture, overfilled with penetration grade bitumen and with or without added fibres (Smith 1998).

After installation and a light compaction sufficient voids remain for it to be permeable to water and air and to accommodate roots of the surface vegetation.

The porosity of Open Stone Asphalt is almost equal to the original used stone (without mastic coating), so a filter layer is required to avoid loss of subsoil particles when seepage occurs. A variety of filters can be applied: woven or non-woven geotextiles, lean sand asphalt or loose granular material.

Pioneers

After an exceptional storm surge along the Belgian coast, in early 1976, several controlled flood areas were constructed as overflows to the river Scheldt.

The hydraulic models were studied at Flanders Hydraulics in Belgium. Bitumar, the Belgian partner of Bitumarin, was contracted to install the Open Stone Asphalt and specific lab-investigations and studies were carried out at the facilities of Bitumarin in Opijnen, Holland (Leguit 1984).

Erosion resistance of Open Stone Asphalt
The properties of resistance to flowing water with high currents were available from test regimes used to design revetments under wave attack or propeller scour from navigation (TAW 1985).
The main conclusions were:
- (Lith, The Netherlands) - Stationary and quasi-stationary flow: limited, only surface, damage was observed after 34 hours with current velocities of 6m/s. (the maximum possible generated flow of the test facilities)
- (Main-Danube-Canal, Germany) - Turbulent flow: no damage occurred after direct attack by an 800HP cargo ship at full strength for 5 minutes (Kuhn 1971).
- (Opijnen, The Netherlands) - Duration flow by pump surcharge to test construction joints.

Performance of geotextile filters to support Open Stone Asphalt
Various woven and non-woven types of geotextile were tested on filter stability in combination with soil samples extracted from the planned location (Leguit 1984).
The main conclusions were:
- After duration testing, the permeability of most filters was reduced by trapped soil particles.
- A thin blanket of sand to catch silt particles would extend the life performance of the filter construction and would reduce the possibility of erosion under the revetment.

<u>Developments in OSA mix design</u>
Open Stone Asphalt is a gap-graded, underfilled mixture of mastic asphalt and aggregate. The mastic comprises sand, filler and bitumen, and coats the aggregate particles with a layer 1 to 1.2mm thick. The mastic film is resistant to weathering and fixes the open aggregate skeleton together to withstand hydraulic loading. Ongoing research and development has led to several improvements in the mix design procedure (TAW 2002).

Aggregate/bitumen adhesion
The adhesion between the aggregate and the bitumen is important for the durability of OSA as it is the integrity of this bond which gives the material its strength. In the past it was generally accepted that carboniferous limestone had good adhesion to bitumen and this stone was usually used. After problems with the durability of a coastal revetment, the adhesion of different aggregate was investigated, and it was concluded that the chemical composition and the surface texture of the aggregate should also be considered, and that other types of stone could be acceptable.

The current standard is to always check the adhesive properties of the stone with the Queensland stripping tests, using a known control stone.

Mastic Viscosity

The asphaltic mastic coats the aggregate to give the material strength. To coat the aggregate correctly the viscosity of the mastic is very important. At the mixing temperature (typically 140-160^{o}C) the mastic must be sufficiently fluid to coat the stones, but also sufficiently viscous not to drip off the stones causing segregation. The target viscosity for the mastic is 30-80 Pascal seconds (Pa.S) at 140^{o}C, which is tested in the Kerkoven apparatus.

Cellulose fibres

The viscosity of mastic is temperature dependent, so the effects of variations in the mixing temperatures are investigated for the designed mastic. Mastics with a relatively high bitumen content will be more temperature-sensitive than those with a lower amount of bitumen, and so the risk of segregation during mixing due to increased temperature will be greater.

A recent development in the UK is the addition of cellulose fibres to the more temperature-sensitive mastics. The fibres combine with the bitumen to decrease its viscosity and to make it more stable. Typically 0.4-0.5% fibres are added to the mastic, and the bitumen content is increased by 1-2%.

Coating thickness

The proportion of mastic in OSA is generally between 19% and 21% depending on the aggregate grading (40mm or 28mm).

The durability of the OSA depends on a sufficient layer of mastic coating each stone so it is therefore an advantage to calculate the coating thickness rather than rely on a 'rule of thumb'.

For current mix designs the aggregate grading and the flakiness index are used to calculate the surface area per unit weight for the stone. This is then used to calculate the amount of mastic required for each aggregate.

For standard OSA mixes a coating thickness of 1.0mm is used, but this can be increased to 1.2mm for mixes containing cellulose fibres, as the increased stability of the mastic means that a greater coating thickness is achievable without increasing the risk of segregation.

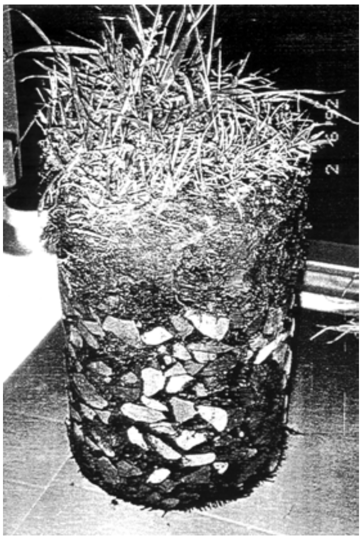


Figure 1: Drilling core extracted from a revetment of Open Stone Asphalt at the project Tielrode (B) with a layer of humus for planting vegetation (Source: ELSKENS 1995)

Performance of Open Stone Asphalt revetment, after overtopping.

Minor cracks were registered in the embankments after overtopping had occurred. A study was carried out to explain the mechanism of failure (Mulders 1983).

The main conclusions are:

Most of the embankments were built like a typical "Dutch Embankment": A sand core, capped with good quality clay. The sand is more permeable to air than the clay, so during a rapid rise in water level the overpressure of the trapped air in the sand core reduces the soil stability and even tries to lift the clay capping during overtopping.

3. INVESTIGATIONS AT UNIVERSITY OF KARLSRUHE

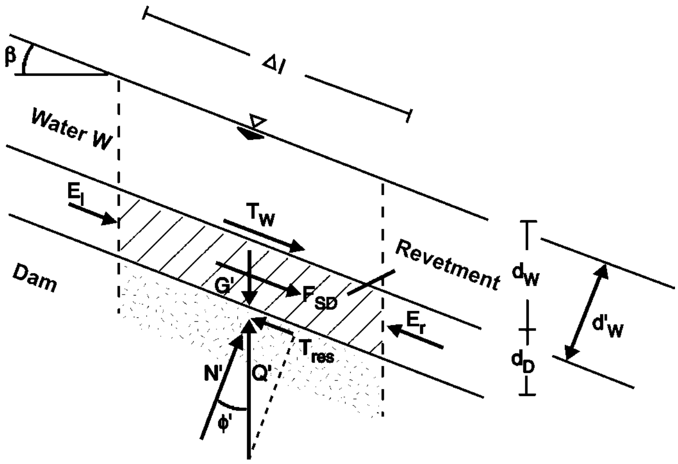
A collaborative research project “dams and embankments (levees) designed for overtopping” was carried out by the Institute of Soil Mechanics and Rock Mechanics and the Institute of Water Resources Management, Hydraulics and Rural Engineering of the University of Karlsruhe. The main question was how to build dams and embankments of a few metres in height in order to withstand intentional overtopping during a flood. In order to do this~~,~~ the downstream slope must be adequately protected against erosion. Up to now there have been no design rules, which made it possible to carry out the necessary stability checks in order to obtain technical solutions which are both economical and easy to build. During this project a statical approach was developed to determine the dimensions of a coherent, self-supporting revetment made of Open Stone Asphalt.

Suitability of a coherent revetment

Various steps were examined to prove the suitability of the selected revetment concept:

- Geotechnical aspects: Dimensions of the revetment, determination of the shear parameters as well as proof of the load capacity of the revetment in a tilting flume.
- Hydraulics: Numerical investigations to determine the dimensions and optimisation of the discharge conditions, in particular to guarantee a reliable dissipation of energy at the toe of the embankment.
- Verification of the results by means of investigations on a half dam model on a technical scale.

The proof of the stability of an overtopped revetment can take place on an embankment element for the given conditions (cf. Larsen et al. 1986). Here the stability can be analysed by comparing all relevant forces and resistances (see Fig. 2).



N', T_{res}, Q'	resulting forces in the shear plane		
Φ'	Angle of friction between the subsoil and the revetment	T_W	Shear stress resulting when overtopped
G'	Weight of revetment under uplift	d_W	Average thickness of the water layer
$E_l \cong E_r$	Earth pressure	d_D	Thickness of the revetment
F_{SD}	Strength of flow	β	Slope angle

Figure 2: Individual element of an embankment which is overtopped and percolated (Larsen et al. 1986)

An evaluation of the analytical correlation is shown in Fig. 3. In this case the slope is covered with a 12 cm thick revetment layer, and the maximum permitted load for an Open Stone Asphalt revetment is given; the results are shown for different angles of friction in the shear plane.

For practical application, the following becomes clear from the correlations shown, as was to be expected:

- The angle of friction in the shear plane has a significant influence on the permitted load capacity of the embankment.
- On flat embankments (small slope angle) higher hydraulic loads are always permissible.
- On the other hand, steep slopes (e.g. $\beta > 15°$) permit only a very low hydraulic load – almost regardless of the size of the angle of friction.
- The correlation shown applies for the limiting state ($F_S = \eta = 1.0$).

The friction conditions in the shear planes are fundamental for the design and the static proof of the self-supporting revetment system. Thus the shear parameters for the system were quantified without any hydraulic load – an angle of friction of approx. 31° was determined.

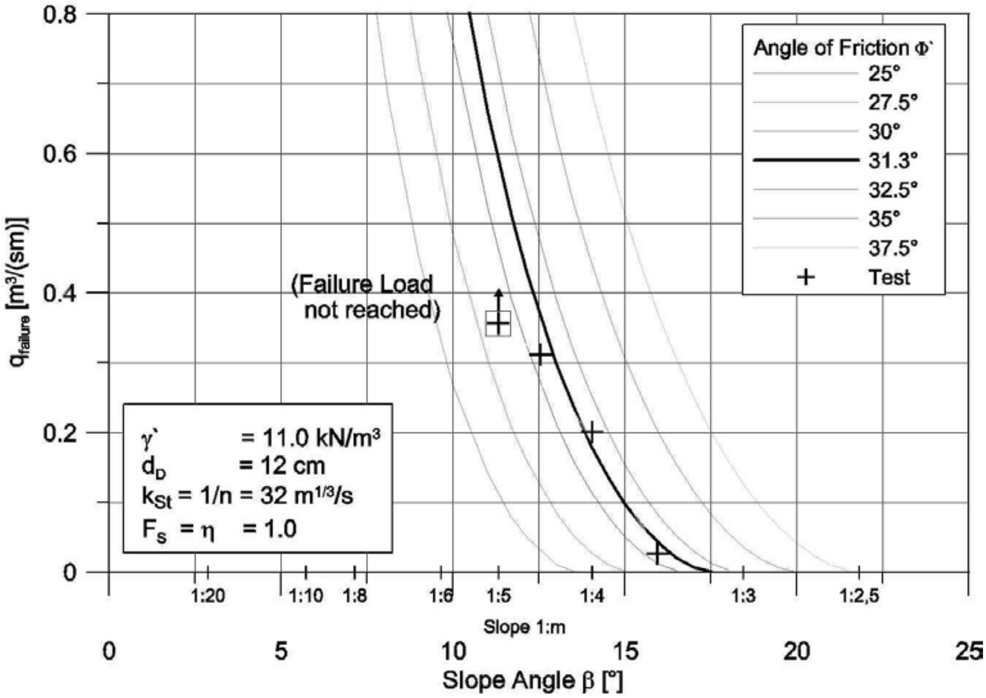


Figure 3: Self-supporting, coherent permeable Open Stone Asphalt revetment – theoretical maximum loads $q_{failure}$ depending on the slope angle β and the angle of friction Φ' (lines) as well as results of experiments from the tilting flume at $\Phi' = 31°$(crosses)

In order to ascertain the maximum hydraulic load of the selected revetment, investigations were carried out on a slope element in a tilting flume, which was infinitely adjustable between an angle of 0° to 35° (length: 4 m, width: 1.31 m). The results confirmed the analytical calculation approach in the investigated load range; the maximum loads for varying slope angles are entered as crosses in Fig. 3. In the future, the dimensions of revetments can be calculated on this basis for the relevant discharge.

From a hydraulic point of view, it was important to look more closely at the three runoff regimes which arise when a dam is overtopped. These differ from one another characteristically with regard to the development of the flow speeds and the water levels as well as the Froude numbers. Here the dam crest with the transition from a subcritical to a supercritical flow was examined and the slope was evaluated with a supercritical steady flow. Above all, the toe of the slope, where at the transition from a supercritical to a subcritical flow a significant portion of the energy dissipation takes place, was the centre of attention. In preparation for the tests on the physical model one-dimensional numerical calculations were carried out. With the help of these calculations, estimates could be made of the position of the hydraulic jump to be expected and on the quality of the energy dissipation.

Finally all the results were used to design and build a half dam model on a technical scale (see Fig. 5), in which the indeed load situation could be realistically reconstructed. This was done up to a hydraulic load of q = 300 l/(sm), in addition the flow conditions in the dambody (sand) were observed and analysed.

An important aspect of the investigations was to optimise the intended energy dissipation at the transition from the slope to the horizontal area, in order to protect the downstream area from erosion. This was achieved by creating a hollow secured with Open Stone Asphalt acting as a scour protection in the transition area between the slope and downstream the toe. The forces and loads arising during energy dissipation are absorbed by the safety element and discharged to the subsoil. Due to the form of the hollow the hydraulic jump cannot move into the unsecured tailwater (see Fig. 6).

Fig 4: Tilting flume – Self supporting Open Stone Asphalt revetment under a load of q = 360 l/(sm) at a slope of 1: 5

Fig 5: Half dam model for overtopping tests at a slope of 1 : 6 during operation

Figure 6: Energy dissipation at the toe of the slope – View from upstream (cf. Bieberstein et al. 2002)

Suggestion for practical application

Based on the statical concept, it is possible to calculate the dimensions of overtopping stretches up to a specific discharge of approx q = 500 l/(sm). As a result of the investigations, an alternative design suggestion for the flood retention basin at Mönchzell, as shown in fig. 7, could be constructively derived: The downstream slope of the dam with an angle of 1 : 8 is covered with a layer of geotextile, on which the revetment of Open Stone Asphalt with a thickness of 20 cm is placed hot.

The Open Stone Asphalt revetment is finally completely covered with topsoil and seeded with grass. However, the grass is not included in the statical considerations. In the case of flooding, the fact cannot be ignored that it could become damaged – and in fact could be lost completely, since the topsoil is not necessarily secure. The revetment underneath takes over the securing of the dam or levee as planned during the period of overtopping.

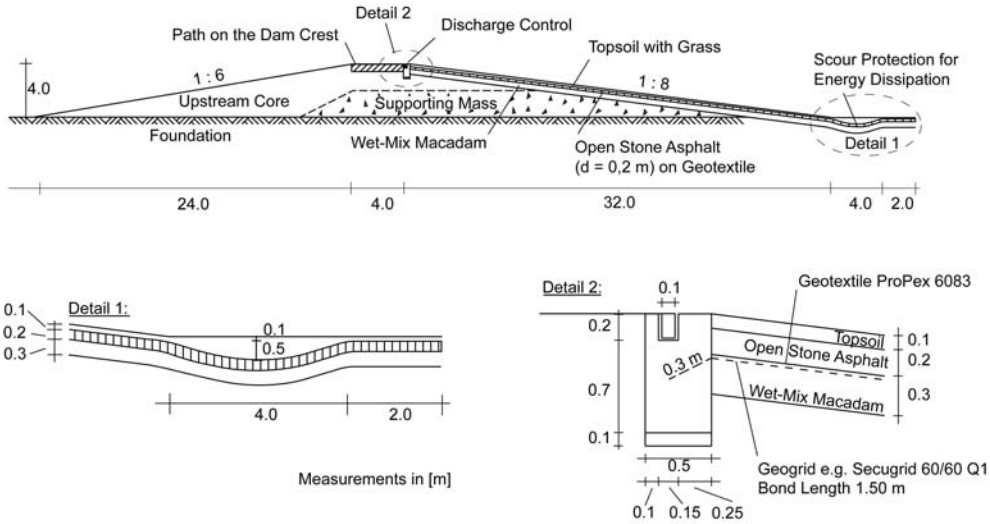


Figure 7: Design for the flood retention basin at Mönchzell with a revetment of Open Stone Asphalt (spec. hydr. load q = 405 l/(sm))

4. DOWNSTREAM SLOPE PROTECTION IN THE UK

Scheme

The Bodmin town leat flood alleviation scheme was designed by Halcrow for the Environment Agency and the main contractor was TJ Brent. The scheme involved the construction of a storage pond above the Cornish town.

The storage pond was created by the construction of a 4 metre high earth embankment with a culvert beneath it to take normal flows. A control structure at the intake limited the flow through the culvert and into the leat through the town, so during flood conditions water would back up and be stored behind the embankment.

In the event of a flood exceeding the design criteria, the embankment was designed to be overtopped over a lowered 60 metre section of the crest, with the water flowing over the downstream slope onto sports pitches below.

The erosion protection required to protect the downstream slope was specified as Dycel 100 blocks placed as pre-fabricated mattresses and secured with in-situ reinforce concrete edge beams.

Open Stone Asphalt (OSA) erosion protection
Hesselberg Hydro proposed an alternative to the concrete blocks comprising a 125mm thick layer of OSA, 250 kg/m^2, placed on a geotextile filter layer.

The OSA layer was thickened by 100mm around the outer edge to increase stability during high flows and at the crest the geotextile was placed in a soil-filled trench to give additional support to the revetment (see Fig. 8).

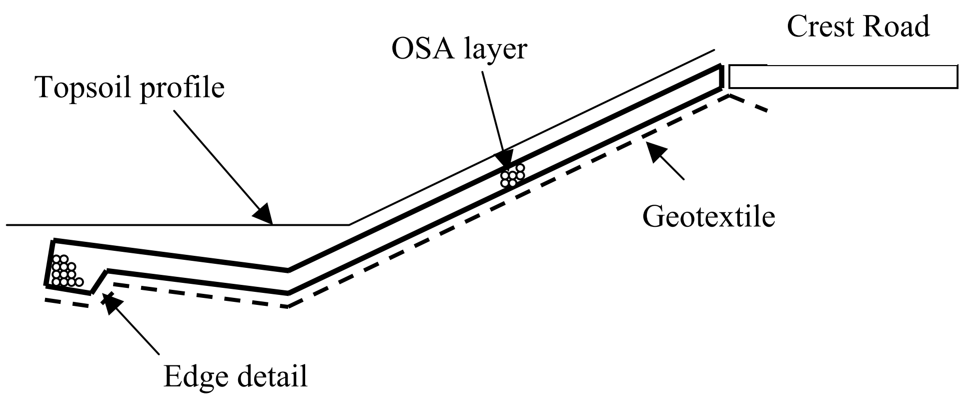


Figure 8: Scheme of the OSA erosion protection

Once the OSA was installed, a layer of topsoil was placed over it and seeded to give the revetment an acceptable appearance.

Overall, 1,000 m^2 was installed complete with crest and edge detail in less than 3 days.

Advantages of OSA

- The in-situ material follows all contours of the stilling basin and is easily placed around manholes and other structures. No cutting of blocks is required.
- A flexible revetment will follow the settlements associated with new earthworks.
- The material is durable and resistant to vandalism and unexpected impacts. In the event of damage to the revetment, the bound nature of the material ensures that damage is not progressive as with a revetment consisting of individual elements.
- The irregular surface voids can support a wider variety of plants than the regular voids in blockwork. The uniformly sized 'pots' in the blocks may suit particular plants which then become dominant.
- A layer of OSA will retain moisture more efficiently than a layer of open concrete blocks where moisture can escape through the holes through the whole block.
- OSA is quick to install and competitive on cost.

Fig 9: OSA erosion protection as installed

Fig 10: Downstream slope after topsoil/seeding

5. CONCLUSIONS

Overflow sections of dams and also of levees are used for flood prevention purposes all over Europe. Open Stone Asphalt has been used in hydraulic engineering for more than 35 years in many countries. In this paper examples of overflowable dam embankments protected by Open Stone Asphalt have been described. The first such attempts were performed in 1976 in Belgium. The experiences have been presented in short form as well as optimized mix design procedures existing at the present time.

In a research project the University of Karlsruhe, Germany, an analytical statical concept was developed for such coherent, self-supporting and permeable revetments. The results were verified in models on a technical scale and are being transferred into practice right now.

LITERATURE

Bieberstein, A., Brauns, J., Queisser, J., Bernhart, H. (2002): *Überströmbare Dämme – Landschaftsverträgliche Ausführungsvarianten für den dezentralen Hochwasserschutz.* Zwischenbericht: BW Plus

CUR, VBW Asfalt, Benelux Bitume (1995): *Toepassing van asfalt bij. Binnenwateren,* (No.1791995) (Dutch).

Dornack, S. (2001): *Überströmbare Dämme – Beitrag zur Bemessung von Deckwerken aus Bruchsteinen.* Dissertation, Institut für Wasserbau und Technische Hydromechanik, Universität Dresden.

Elskens, F. (1995): *Protecting Overflow Dikes for Controlled Flood Areas in Belgium.* PIANC Conference on Inland Waterways and Flood Control, Brussels, Belgium.

Kuhn, R. (1971): *Erprobung von Deckwerken durch Schiffahrtsversuche.* Wasserwirtschaft 3.

Larsen, P., Blinde, A., Brauns, J. (1986): *Überströmbare Dämme - Hochwasserentlastung über Dammscharten.* Versuchsbericht der Versuchsanstalt für Wasserbau und Kulturtechnik und der Abteilung Erddammbau und Deponiebau am Institut für Bodenmechanik und Felsmechanik, Universität Karlsruhe, unveröffentlicht.

Leguit, N. (1984): *Filter Investigations for Paardewijde, Berlare-Wichelen (B).* Report Bitumarin division R&D.

LfU - *Landesanstalt für Umweltschutz Baden-Württemberg (1997): Dammscharten in Lockerbauweise bei Hochwasserrückhaltebecken.* Handbuch Wasser 2.

Mulders, G. (1983): *Technical Report Failmachanism of Overflow Embankments.* Tielrode - Bovenzande, Bitumarin division R&D.

Rathgeb, A. (2001): *Hydrodynamische Bemessungsgrundlagen für Lockerdeckwerke an überströmbaren Erddämmen.* Dissertation, Institut für Wasserbau, Universität Stuttgart.

Schönian, E. (1999): *The Shell Bitumen Hydraulic Engineering Handbook.* ISBN 0953588505.

Smith, R. (1998): *The use of asphalt in dam maintenance work.* Dams & Reservoirs Vol. 8 No. 3, Dec. 1998.

TAW - Technical Advisory committee on Waterdefences (1985): *The use of asphalt in hydraulic engineering,* (No.37/1985).

TAW - Technical Advisory committee on Waterdefences (2002): *"Technisch Rapport Asfalt voor Waterkeringen"* (Dutch).

SESSION 2
THE USE OF NEW MATERIALS

Chairman Jon Green
Technical Reporter Bryn Philpott

Papers Presented

1. Raising of Ajaure embankment dam by extending the moraine core with a geomembrane
A Nilsson/L Ekstrom

2. Twenty five years experience using bituminous geomembranes as upstream waterproofing for structures
M Turley & J.L Gautier

3. Watertightness and safety of dams by geomembrane
A.M Scuero & G.L Vaschetti

4. Design and performance of Elvington balancing and settling lagoons
A Robertshaw & A MacDonald

Papers not presented

1. Downstream slope protection with open stone asphalt
A Bieberstein/N Leguit/J Queisser & R Smith

Jon Green (Chairman)
General question to all. For the products that you're promoting, what protection is there against vandals?

Andrew Robertshaw (Yorkshire Water Services Ltd)
The sort of protection that we tend to use depends upon the situation. It could just be a layer of rip-rap or large stones on top of the geomembrane. It could be, as in the case of the dams that I've spoken about, an unreinforced fibrous concrete slab cast in situ. There have also been interlocking blocks or blocks suspended on steel hawsers down the face of the dam just below the water level.

I think each situation needs to be treated on its merits. We have used Coletanche with no protection layer at all, under situations where it's been secure for example at a fenced-off lagoon. We're quite confident that the materials are perfectly UV stable and do not suffer from atmospheric damage.

Gabriella Vaschetti (Carpi Tech SA)
Carpi does not like to protect the membrane, we prefer to leave it exposed so that it can be inspected and if any repair is needed we can do it under water. However where vandalism is really a concern, besides the type of protection that has already been cited, it is also possible to use a very light protection systems that are made with special geo-synthetics that can be

filled up with shotcrete. This is something that is very light and can be secured at the crest of the dam so it does not need a particular care in the designing of a support structure because of it's lightweight properties. It is supported by the crest itself or by the wave wall and it can be extended down to the minimum water level. Even if it cracks, it will still protect the membrane from vandalism so this is an additional protection.

In Germany for example, in a canal, we used the Macaferri gabions in a double layer without the stone infill. This was done to allow animals to get out of the canal but also provided additional protection against vandalism.

Chris Binnie (Independent Consultant)
I was involved some years ago in a lined reservoir of some 300,000 sq metres using H.D.P.E. We did not use concrete anchor strips. They do seem to add to construction complexity and cost. Why were they used here and would he use them again?

Andrew Robertshaw (Yorkshire Water)
This is referred to in the paper. The concrete strip design was built purely for wind resistance. You're quite right, it does add significantly to the costs and to the detail of construction. I can say that the designer felt that that was necessary.

Ian Carter (M.W.H. Ltd)
I've just got a quick question for Ake Nilsson on his Ajaure Dam in Sweden. You had quite considerable horizontal displacements, could you explain the mechanism for that briefly? The second part of the questions is have those horizontal displacements ceased now since you've done the remedial works?

Ake Nilsson (SwedPower AB)
At the moment we think these horizontal displacements are caused by the shoulder material, which is a rock fill with a high mica content. This is crushing down when the reservoir level goes up and down (approximately 15 metres every year). We have had a horizontal displacement of half a metre since the start of construction.

The embankment berms on the downstream side are there to improve erosion resistance and also improve the sliding resistance for large shear surfaces, however they do not stop deformation, so it continues at the moment with displacements of 8-10mm per year.

We have not done anything about this. The leakage at the moment is very small and the berm will allow for large leakages if they occur.

Chris Binnie
I notice that your wave wall was on the upstream side, whereas your membrane had terminated near the downstream side. As I understood it the wave wall is actually resisting the variation due to the waves but underneath the wave wall you've got semi-permeable fill. I wondered how you calculated the height that you had to take your membrane up in relationship to the wave height.

Ake Nilsson
The height of the membrane is just above the stable water level at very extreme floods. The wave wall is just there to avoid overtopping of waves for winds, which could occur at the same time as the extreme flood. This is calculated for smaller wind velocities than at normal

reservoir levels. They are calculated for wind velocities of 20 m/s whereas we usually use around 30 m/s wind velocities at the normal retention level.

John Sammons (Independent Consulting)
A question for Michael Turley. We are used to concrete slabs being employed, with varying degrees of success, to protect the upstream face of dams from the action of waves.

The picture you showed of cast in-situ concrete slabs gave the impression they were rather thin particularly when considering the dam's steep slope. Could you clarify the purpose of the concrete slab facing? Is its primary job to prevent damage by objects impacting on the membrane and is there no actual need to defend against wave action?

Michael Turley (Colas.UK Ltd)
Well, I wasn't intimately involved in that particular dam. I did go to visit it and it was very nice but it did rain. As far as I'm aware the slabs are there for physical protection because the region was up in the mountains. There were lot of trees around the reservoir and dead trees were falling into the water and floating downstream. It was really there for protection of the geomembrane against impact of things like floating trees, dead animals etc.

Colin Hunt (Bristol Water Plc)
It's a question for Andrew Robertshaw. It's an operational problem really. How do you clean the reservoirs in question and what precautions do you need to take to prevent damage to the membrane?

Andrew Robertshaw
As I said, the reservoirs have to date been cleaned on a 3-year cycle. They were constructed with a concrete base slab, principally with the aim of assisting cleaning. We can get plant into the lagoons, down concrete access ways and onto the base slab. The reservoir is then cleaned from that slab using high pressure hoses. The problem with the wrinkles is that you get a lot more accumulation of material in those wrinkles. You've therefore got to give them more attention which takes more time and effort.

Cassio Viotti (President ICOLD)
This question is about the Swedish dam. I would like to know how was the rock fill compacted and what depth were the layers? I know several cases where rock fill has kind of a creep behaviour and keeps settling with time. Also there are cases where the rainfall can increase settlement.

Ake Nilsson
As far as I know it was not compacted by vibrating rollers in layers. It was only put in layers and it was sluiced with water. With regards to the depth of the layers I cannot say that but I would guess it was a couple of metres.

Harry Doherty (ESB)
My question is for Michael Turley. You mentioned placing your geomembranes under water. How do you form a welded seam underwater?

Michael Turley
You obviously can't use propane torches to weld the Coletanche, but there is a material produced by Shell called Tixafelt which can be applied as a paste under water. You clearly

need to weigh the seam down and it has a curing period of several days. It’s quite simple to carry out repairs under water repairs if necessary and you can also use this for putting over things like waterstop bars etc before you weld.

George Hallows (Independent Consultant)
It is important to remember that some plastic liners can be lethally slippery. At a small farm reservoir, for which I am Construction Engineer under the Reservoirs Act 1975, a man slipped and fell down the lined 1 on 2 slope while trying to extricate his dog which had fallen in previously. The dog later managed to get out somehow but the man did not and he drowned.

The water in that reservoir is now being drawn down to allow remedial works unconnected with that incident, and when I was on site last week the moderate breeze which was blowing was sufficient to lift the liner, now not restrained by any significant weight of water. On the face of one bank it was rising and falling by about one metre. An effective method often used to prevent uplift by wind is to lay old car tyres on the liner tied to ropes anchored at the top of the bank. In the case mentioned this had been done previously but it appeared that ropes had frayed and many of the tyres had come loose and fallen into the water. “Chains” of tyres fixed to ropes in this way can also act as ladders to allow people who fall in to climb out, and may be used as such in desperation even if that was not the original intent. Therefore the ropes should not only be strong enough to prevent fraying and loss of the tyres, but they should also be strong enough, and the anchors should be strong enough, not to fail under the weight of a person climbing out.